1994

Stability of roadways and intersections

Jamal Hematian
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STABILITY OF ROADWAYS AND INTERSECTIONS

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

Doctor of Philosophy

from

UNIVERSITY OF WOLLONGONG

by

JAMAL HEMATIAN
(BSc, MSc Mining Engineering)

Department of Civil and Mining Engineering

June 1994
IN THE NAME OF GOD

This thesis is dedicated to my dear parents;

My mother, Mansoreh Namazi
My father, Ahmad Hematian

for their love and support
AFFIRMATION

I hereby certify that the work which is being presented in the thesis entitled "Stability of Roadways and Intersections" in fulfilment of the requirements for the award of the degree of Doctor of Philosophy, submitted in the Department of Civil and Mining Engineering, University of Wollongong, is an authentic record of my own work carried out during the period from July 1991 to June 1994 under the supervision of Prof. R. N. Singh and Dr. Ian Porter. To the best of my knowledge and belief this thesis contains no material previously published or written by another person except where due reference is made in the body of the text.

JAMAL HEMATIAN

The following publication has been based on this research:

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ABSTRACT

A major factor determining the efficiency and economy of underground coal mining is stability of the openings. Gate roadways and intersections are particularly susceptible to ground control problems due to the inherent wide roof span, excessive stress and variable intersection shape. Adverse conditions such as high horizontal stress and abutment pressures from longwalls also have a deleterious effect on ground control.

As many past studies which dealt with site specific problems failed to address the fundamental mechanisms of failure, there are still many stability problems in underground coal mines. This research was carried out to determine the actual failure mechanism of roadways and intersections and includes case studies from Australian underground coal mines. The present work provides an in depth study of the behaviour of the roof, pillar and floor in roadways and at intersections leading to the design of appropriate support systems under various conditions.

The research was conducted in three major parts to achieve the objectives of the project. Laboratory testing was carried out on various rocks to determine the mechanical properties of rocks and prepare input data for numerical modelling. Numerical modelling was conducted using the Finite Element Method to consider the effect which different variables have on the behaviour of roadways and intersections. Finally, a field investigation program was undertaken to obtain in-situ data for validation of numerical results. Each part of the research was performed in such a way that they were interactive with one another.

Theoretical, empirical and numerical research relating to the stability of roadways and intersections were reviewed and also general features of the Finite Element Method (FEM) were described. The 3-D FE code, NASTRAN, was introduced and its limitations were discussed. A number of new modelling techniques were developed and applied in the program in order to gain the most realistic stability assessment of underground structures. These techniques enabled consideration of the post-failure behaviour, empirical failure criteria and complete stress-strain characteristics of rocks as well as allowing the assessment of bedding plane effects and appropriate loading techniques for underground structures.

Two underground coal mines in New South Wales, Australia were chosen for field investigations and sample collection. Laboratory testing was designed for acquisition of complete information on the mechanical properties of coal measures rocks such as coal,
shale, mudstone and three types of sandstone. More than 250 standard samples were prepared and tested using uniaxial and triaxial compressive, point load, direct shear and Brazilian tests. The results from the triaxial tests were examined against the Mohr-Coulomb, Bieniawski and Hoek and Brown failure criteria. In addition to the above criteria a new failure criterion was proposed. Moreover, rock mass classification was used to determine the in-situ properties of strata units based on the classifications index and laboratory results. The conventional and proposed failure criteria were employed in the finite element analysis (FEA) of a roadway model to determine the limitations of these failure criteria in stability analysis of underground structures.

The stability of roadways (2-D models) was investigated by means of FEA of general and site-specific models. The mechanism of interaction between the roof, pillar and floor under various conditions was comprehensively studied. The significance of high horizontal stress, post-failure behaviour of rocks and abutment pressure from longwall faces on the stability of main and gate roadways was addressed. A general approach was suggested for the design of an optimum (safe and economic) support system by evaluating three potential modes of failure (structural failure criteria); arch failure over the roof line, shear failure on the bedding planes and guttering over the rib-lines. The stability of pillars and the floor was considered by determining the extent and degree of yield in the elements surrounding the roadway. The results obtained from FEA of roadways employing the new techniques were in good agreement with field data.

Further application of the finite element technique was extended to determine the influence of certain parameters on the behaviour of four-way and three-way intersections. Individual parameters such as depth of cover, the ratio of horizontal to vertical stress and the width of opening were varied during a parametric analysis using 3-D models of intersections. The influence of the above factors on the stress and displacement patterns around the intersections was determined. In addition, site-specific models of intersections based on field data were constructed and analysed. Results from this research were very promising and were validated by empirical investigations carried out both during this project and in the past by other investigators. On the basis of the results of the investigation, a new procedure was suggested for the design of an optimum support system at intersections.
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<td>two-dimensional</td>
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<tr>
<td>3-D</td>
<td>three-dimensional</td>
</tr>
<tr>
<td>B</td>
<td>shape function</td>
</tr>
<tr>
<td>B'</td>
<td>width of the panel</td>
</tr>
<tr>
<td>B''</td>
<td>safe unsupported span of the opening (p. 25)</td>
</tr>
<tr>
<td>b</td>
<td>diagonal span of the intersection (p. 26)</td>
</tr>
<tr>
<td>B</td>
<td>stress factor in the Coulomb shear strength equation</td>
</tr>
<tr>
<td>C</td>
<td>half of the beam thickness (p. 17)</td>
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<tr>
<td>C''</td>
<td>in-situ uniaxial compressive strength of the pillar (p. 36)</td>
</tr>
<tr>
<td>C''''</td>
<td>uniaxial compressive strength</td>
</tr>
<tr>
<td>c. g.</td>
<td>coarse grained</td>
</tr>
<tr>
<td>D</td>
<td>bolt diameter</td>
</tr>
<tr>
<td>D</td>
<td>deflection rigidity factor (= E . I) (p. 19)</td>
</tr>
<tr>
<td>D</td>
<td>diameter of specimen in Brazilian test (p. 135)</td>
</tr>
<tr>
<td>D_e</td>
<td>elasticity matrix for plane stress</td>
</tr>
<tr>
<td>D_eP</td>
<td>elastic-plastic matrix for plane stress</td>
</tr>
<tr>
<td>D_l</td>
<td>Internal Model displacement</td>
</tr>
<tr>
<td>D max</td>
<td>maximum bottom dimension of the roof fall at the intersection</td>
</tr>
<tr>
<td>D min</td>
<td>minimum bottom dimension of the roof fall at the intersection</td>
</tr>
<tr>
<td>D n</td>
<td>Null Model displacement</td>
</tr>
<tr>
<td>D n</td>
<td>nominal dimension of the roof fall bottom at the intersection (p. 41)</td>
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<td>D o</td>
<td>External Model displacement</td>
</tr>
<tr>
<td>d c</td>
<td>height of the roof fall at the intersection</td>
</tr>
<tr>
<td>d c 0.1</td>
<td>half of the bottom diameter of the stress contour line 0.1σv</td>
</tr>
<tr>
<td>d h</td>
<td>maximum heave at the centre of the intersection</td>
</tr>
<tr>
<td>d max</td>
<td>maximum displacement</td>
</tr>
<tr>
<td>d s</td>
<td>maximum sag at the centre of the intersection</td>
</tr>
<tr>
<td>d x</td>
<td>height of the roof fall in the X-roadway</td>
</tr>
<tr>
<td>d y</td>
<td>height of the roof fall in the Y-roadway</td>
</tr>
<tr>
<td>E</td>
<td>elastic modulus</td>
</tr>
<tr>
<td>E m</td>
<td>rock mass elastic modulus</td>
</tr>
<tr>
<td>ESR</td>
<td>the excavation support ratio</td>
</tr>
<tr>
<td>F</td>
<td>force vector</td>
</tr>
<tr>
<td>F</td>
<td>force</td>
</tr>
<tr>
<td>FE</td>
<td>finite element</td>
</tr>
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</table>
FEA: finite element analysis
FEM: finite element method
f. g.: fine grained
{F}: column matrix of force
{f}: column matrix of force
GAP: one-dimensional element
H: depth of cover
He: entry height
Hf: height of the roof fall at the intersection
Hm: height of the model
Ho: height of the opening
H': slope of the stress strain curve in the plastic region
h: pillar height
h: load height (p. 26)
hc: height of the stress contour line 0.1σv
I: second moment of area
Is: point load index
ISRM: International Society of Rock Mechanics
J2: second invariant of deviatoric stress
Jr: joint roughness
K: constant in the Experimental criterion (p. 120)
K: stiffness
K: the ratio of horizontal to vertical stress
[K]: stiffness matrix
L: bolt length
L: length of a beam or a plate (p. 17)
L: length of the specimen in Brazilian test (p. 135)
Lp: length of a pillar
LVDT: linear variable differential transducer
M: moment
m: constant in the Bieniawski and Hoek and Brown criteria
m: height of the extraction (p. 35)
m. g.: medium grained
N: normal stress on the bedding plane
NASTRAN: a finite element program
NGI: Norwegian Geotechnical Institute
Nφ: friction number
n: constant in theBieniawski criterion
<table>
<thead>
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<th>Symbol</th>
<th>Description</th>
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<td>n</td>
<td>number of chain pillars across a gate entry (p. 33)</td>
</tr>
<tr>
<td>P</td>
<td>constant in the Experimental criterion (p. 120)</td>
</tr>
<tr>
<td>P</td>
<td>failure load in Brazilian test (p. 135)</td>
</tr>
<tr>
<td>P</td>
<td>pressure</td>
</tr>
<tr>
<td>P_Y</td>
<td>bolt yield strength</td>
</tr>
<tr>
<td>P_u</td>
<td>bolt ultimate strength</td>
</tr>
<tr>
<td>PID#</td>
<td>property identification number</td>
</tr>
<tr>
<td>Q</td>
<td>rock mass quality index</td>
</tr>
<tr>
<td>{Q}</td>
<td>unknown vector of constant forces due to single and multi-point constraints</td>
</tr>
<tr>
<td>Q_l</td>
<td>load distribution across the length of a plate</td>
</tr>
<tr>
<td>Q_w</td>
<td>load distribution across the width of a plate</td>
</tr>
<tr>
<td>QUAD4</td>
<td>two-dimensional element</td>
</tr>
<tr>
<td>q</td>
<td>nodal force vector</td>
</tr>
<tr>
<td>q_u</td>
<td>bearing capacity of the floor</td>
</tr>
<tr>
<td>R</td>
<td>tensile load induced in each bolt</td>
</tr>
<tr>
<td>RMR</td>
<td>rock mass rating</td>
</tr>
<tr>
<td>RQD</td>
<td>rock quality designation</td>
</tr>
<tr>
<td>r</td>
<td>stress concentration ratio (the ratio of induced to virgin stress)</td>
</tr>
<tr>
<td>S</td>
<td>stress concentration ratio (the ratio of induced to virgin stress)</td>
</tr>
<tr>
<td>SAFETY</td>
<td>the combination of SFCONT and SFDRAW</td>
</tr>
<tr>
<td>SF</td>
<td>safety factor</td>
</tr>
<tr>
<td>SFCONT</td>
<td>a fortran program for consideration of the post-failure behaviour of rocks</td>
</tr>
<tr>
<td>SFDRAW</td>
<td>an Auto-list program as a graphical post-processor for SFCONT</td>
</tr>
<tr>
<td>SOLID</td>
<td>three-dimensional element</td>
</tr>
<tr>
<td>s</td>
<td>constant in the Hoek and Brown criterion</td>
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<tr>
<td>T</td>
<td>constant in the Experimental criterion</td>
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<tr>
<td>T</td>
<td>tensile stress</td>
</tr>
<tr>
<td>t</td>
<td>beam thickness</td>
</tr>
<tr>
<td>U</td>
<td>displacement vector</td>
</tr>
<tr>
<td>UCS</td>
<td>uniaxial compressive strength</td>
</tr>
<tr>
<td>UTS</td>
<td>uniaxial tensile strength</td>
</tr>
<tr>
<td>{u}</td>
<td>column matrix of displacement</td>
</tr>
<tr>
<td>W</td>
<td>load per unit length</td>
</tr>
<tr>
<td>W</td>
<td>plate width (p. 21)</td>
</tr>
<tr>
<td>WC</td>
<td>weakening coefficient</td>
</tr>
<tr>
<td>W_e</td>
<td>total width of the gate entry</td>
</tr>
<tr>
<td>W_m</td>
<td>width of the model</td>
</tr>
<tr>
<td>W_o</td>
<td>width of the entry</td>
</tr>
</tbody>
</table>
\( W_p \) : width of the pillar
\( X_d \) : width of the yield zone
\( Z \) : depth
\( \beta \) : angle of shear
\( \delta \) : deflection
\{\( \delta \)\} : error vector
\( \varepsilon \) : strain vector
\( \varepsilon \) : strain
\( \varepsilon_0 \) : initial strain vector
\( \phi \) : internal angle of friction
\( \gamma \) : density
\( \lambda \) : elastic stiffness before peak strength
\( \lambda' \) : elastic stiffness after peak strength
\( \mu \) : friction coefficient
\( \mu_Y \) : friction coefficient of gap element in \( Y \) direction
\( \mu_Z \) : friction coefficient of gap element in \( Z \) direction
\( \sigma \) : stress
\( \sigma \) : stress vector
\( \sigma_1 \) : uniaxial compressive strength of pillar at the critical size (p. 37)
\( \sigma_1 \) : maximum principal stress
\( \sigma_3 \) : minimum principal stress
\( \sigma_{1n} \) : normalised maximum principal stress
\( \sigma_{3n} \) : normalised minimum principal stress
\( \sigma_c \) : uniaxial compressive strength of the laboratory sample
\( \sigma_h \) : horizontal stress
\( \sigma_i \) : Internal Model stress (p. 77)
\( \sigma_i \) : vertical induced stress
\( \sigma_m \) : first invariant of stress
\( \sigma_o \) : External Model stress
\( \sigma_P \) : average pillar load
\( \sigma_t \) : indirect tensile strength
\( \sigma_v \) : vertical stress (virgin stress)
\( \sigma_Y \) : peak abutment pressure, yield stress
\( \sigma_z \) : vertical stress (induced stress)
\( \tau \) : shear stress
CHAPTER ONE

STABILITY OF ROADWAYS AND INTERSECTIONS
CHAPTER 1

STABILITY OF ROADWAYS AND INTERSECTIONS

1.1 Introduction

A major factor determining the efficiency and economy of underground coal mining is the stability of the openings. Although several systematic attempts have been made to understand the mechanisms of ground movement due to extraction of coal seams, there are still many roof fall occurrences in underground coal mines. These falls occur particularly in gate entries and at intersections, and tend to damage the underground structures resulting in economic loss and in certain cases fatalities.

1.2 Roadways in underground coal mines

The first step in the development stage of any underground mine is limited to driving access roadways and intersections (where the main roadway meets the cut-throughs) to divide the coal seam into panels and pillars. These roadways can be classified under two main categories; main roadways (main entries) and gate roadways (head and tail entries).

The main roadway will have a long life and forms the main artery for coal removal and provides access for men, equipment, materials and ventilation in the underground network. During its life, the main roadway will service many coal producing panels. The stress distribution around the main entries is similar to that in room-and-pillar mining. Gate roadways on the other hand have a limited life span varying from a few months up to twice that of a panel. With the increased rate of face advance arising from the need for greater concentration of production, the average gate life will reduce. The stress pattern around the gate entries is highly influenced by the side abutment pressure resulting from adjacent longwalls. Figure 1.1 shows typical roadways in underground coal mines.

Intersections are formed when the pillars between two roadways or between one roadway and an adjacent panel are cut through. Roadway intersections are grouped into two types: four-way (+ type) and three-way (T type) intersections. In both types of the intersections the diagonal roof span is wider than either individual roadways (Figure 1.2).
1.3 Problems associated with the stability of roadways and intersections

Despite many investigations into the stability of roadways, adverse conditions such as high horizontal stress and the unsteady state of abutment pressures from longwalls cause serious stability problems; especially in gate entries.

Roadway intersections in underground coal mines are particularly susceptible to ground control problems. This is due to inherent wide roof spans, excessive stress and variable intersection shapes. The region surrounding a roadway intersection is characterised by different failure criteria from the region surrounding a single entry. Stresses induced during intersection development may result in a high incidence of roof and rib failure.
Beam theory which has been used in the past to describe roof deflection and failure of individual entries is not applicable to analyse intersection structures. It is therefore, necessary to use plate or slab models to evaluate stresses and displacements in the strata overlying intersections.

In contrast to two-dimensional analysis of roadways, evaluation of failure mechanism at intersections is very complex because stress and deflection must be analysed as a three-dimensional problem. Also, the difference between the mechanical properties of the floor, coal and roof strata makes this analysis more complicated. Furthermore, in the presence of high horizontal stresses as is the case in underground coal mines in Australia, a realistic solution can only be obtained using a three-dimensional model.

1.4 Australian underground coal mines

The coal mining industry has made a significant contribution to Australian economic growth over the last decade. Since 1984, Australia has been the world's largest exporter of coal. Mines in Australia produced a total of approximately 180 million tonnes of black coal and made about 7.2 billion dollars revenue for 126 million tonnes of coal exports in 1992 (Brush, 1993).

Coal seams in Australia are mostly flat and the workable thickness varies from 1.5 to 10.0 metres. Black coal deposits occur at depths down to 1300 metres. Presently the deepest mining operations are carried out at depths down to 550 metres in strata which are relatively strong and exhibit brittle fracture behaviour when compared to conditions prevailing in Europe and the U.K. Most excavations in these strata are subject to high horizontal stresses, which lead to the development of an envelope of fractured ground (Schaller and Savidis 1985). During almost two centuries of coal mining, a variety of mining methods have been practiced and developed to suit local mining conditions. Of these methods, longwall mining is currently the key element in a cost-competitive and export based underground coal industry in Australia (McKensey 1990). In 1992, there were 24 operating longwall faces in 23 mines (NSW Coal Industry Profile, 1993).

Although there have been many attempts to improve the stability of roadways, there are still some problems with keeping open the individual development units as well as with poor development rates restricting longwall productivity. For example, in 1985, major strata control problems in the main gate of West Cliff Longwall panel No. 6 resulted in a fall which stopped production for six weeks. Similarly, in 1987, failure of a take-off roadway at Pacific Colliery caused the longwall equipment to be buried, stopping
longwall operation for approximately three months (Ruston, Lama and Cutafani 1988). Incidents such as those mentioned above have a significant effect on overall longwall output with the result that the benefits, which should be derived from high productivity systems, can be eliminated over a short period.

1.5 Significance of the research

From an Australian viewpoint, coal as a natural resource offers opportunity for economic, social and political benefits. However, the Australian coal industry is facing a challenging decade. Demand for Australian black coal for export continues to grow strongly, but there are a number of competitors abroad with high potential capacity. Therefore, it is expected that all mines have to operate in a very competitive environment. In these circumstances, economic and safe operation is a key factor to establish the coal industry at the competitive end of the cost curve.

Reviewing the production and safety of Australian underground coal mines in recent years reveals that while the productivity of longwall faces has remained unchanged, about 6300 t/d since 1989, the number of accidents resulting in fatalities rose sharply in 1991 as shown in Figure 1.3. It has been indicated that a considerable portion of these problems were related to the stability of roadways and intersections, and gateroad development is still behind what is required to satisfy the new productive longwalls (Price 1994). To achieve high recovery of coal from underground coal mines at minimum cost and in a safe condition, mining engineers must find solutions to difficulties and problems that restrict the efficiency of coal mining methods; one of the most important areas is the stability of roadways and intersections. This research is an attempt in this direction.

![Figure 1.3 Fatalities in NSW underground coal mines (Lost-Time Injuries, 1993).](image-url)
1.6 Research objectives

At the outset the objective of this research was to analyse the structural behaviour of roadways and intersections with respect to particular Australian coal mines where roof strata are thick and competent and the maximum principal stress is horizontal. After calculating the stress redistribution around the structure and determining corresponding displacement of the roof, pillar and floor, this study was extended to define the actual failure mechanism for these locations. Consequently, the influence of various adverse parameters such as high horizontal stresses was identified. The final objective was to assess and evaluate support requirements in relation to the stability and safety of roadways and intersections with special reference to Australian coal mining conditions.

1.7 Research methodology

Three-dimensional (3-D) analysis of such a complex problem is almost impossible by theoretical methods. Therefore, Numerical Modelling was chosen as the main method for this project. The 3-D finite element program, NASTRAN, was used to solve the problem for induced stresses and displacement of surrounding strata.

Three major points were applied in the numerical analysis of the problem, which have not been considered in previous researches. These are: accurate stress-strain relationship, representative failure criterion and acceptable post-failure behaviour of each type of rock (roof, coal and floor) contributing to the structure of the roadways and intersections. Also it was necessary to develop new techniques which could be applied to construct and analyse more realistic models of the structures, and to optimise the finite element (FE) parameters.

Along with numerical modelling, the research program included comprehensive laboratory testing and field investigation to acquire the input data to run the FE programs as well as to validate the results achieved by numerical modelling.

The procedure of numerical modelling for stability analysis of roadways and intersections as illustrated in Figure 1.4, consists of four basic steps as follows:

(a) define the mechanical property of materials, geometry of structure and virgin stresses,
(b) determine stresses and displacement induced around the structure,
(c) assess the stability of the structure, and
(d) if unstable conditions exist, design an appropriate support system.
1.8 Schedule of research

This research consisted of six distinct phases, shown in Figure 1.5, and it was aimed to complete the work in three years (Table 1.1). The first phase was limited to a literature survey of the fundamentals of numerical modelling, strata mechanics and related investigations.

The second phase was to assess and modify the FE program to consider non-linear properties, empirical failure criteria as well as post failure behaviour of materials. The third phase was laboratory testing on samples along with site investigations for acquisition of input data for computer modelling.
CHAPTER 1: Stability of Roadways and Intersections

PHASE 1: Literature survey on fundamentals of:
- Numerical methods
- Strata mechanics
- Related investigations

PHASE 2:
- Evaluating FE programs
- Choosing a 3-D FE program

PHASE 3:
- Laboratory testing
- Mechanical properties of rocks
- Site investigations
- Geometry of structures

PHASE 4:
- Making changes in FE model

PHASE 5:
- In-situ instrumentation
- Induced stresses & displacements

PHASE 6:
- Comparing the results
- Agreement
- Stability assessment
- Stable: END
- Unstable: Evaluate support requirements

Figure 1.5 Stages of the research.
CHAPTER 1: Stability of Roadways and Intersections

The fourth phase was to run the selected FE program using data gained through phase three. Patterns of stress distribution and displacement around the structure are the result of this phase. These results were classified and interpreted to clarify the structural behaviour of roadways and intersections. The fifth phase included field investigation. This was conducted to detect actual in-situ behaviour of the strata surrounding the structures. Comparing roof sag, floor heave, pillar displacement and roof fall height predicted by numerical modelling with field results reveals any possible defects in the numerical model. Consequently, if any appreciable discrepancies were observed, modifications to the model were necessary. The sixth and final phase was to assess the stability of roadways and intersections. If the outcome showed unstable condition, then support requirements were evaluated.

Table 1.1 Research time table.

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1.9 Scope of the thesis

This thesis consists of 8 chapters. The first Chapter presents the general purpose of the research, and also highlights the importance and impact of the stability of roadways and intersections on the safety and economy of underground coal mines in New South Wales, Australia.

In Chapter 2 strata control theories and previous research techniques are reviewed. Theoretical, empirical and numerical works are classified and discussed according to whether they relate to roof, pillar or floor behaviour. This Chapter also presents a summary of various techniques and solutions.
Chapter 3 describes the general features of numerical methods, with particular reference to the fundamental of the finite element method (FEM). The 3-D FE code, NASTRAN, is introduced and its limitations are discussed later in the Chapter.

In Chapter 4, the new techniques which are developed by the author are described. These techniques are for considering the most realistic stability assessment of underground structures including post-failure behaviour of rocks, empirical failure criteria, bedding plane effect, appropriate techniques of loading and so on. To carry out this investigation, more than 100 models were constructed and tested.

Chapter 5 covers a comprehensive study of the site observations and laboratory investigations. Two Collieries were chosen as sites for field investigations and sample collection. Laboratory tests were designed for acquisition of complete information on mechanical properties of rocks in six major strata units. In this part of the research more than 250 standard samples were prepared and tested for Uniaxial, Triaxial, Direct Shear, Point Load and Brazilian tests.

Chapters 6 and 7 describe the specific results achieved from FE modelling of roadways and intersections, respectively. These models were constructed based on the real data obtained from laboratory and field investigations for this particular research. 2-D models were analysed to reveal the structural failure mechanisms of roof, pillar and floor and also to determine the effect of different factors on the stability of the roadways. These models enabled the simulation of the actual strain softening of rocks and accounted for the residual strength of rocks after their failure. Furthermore, a number of 3-D models of three-way (T) and four-way (+) intersections were built and analysed to reach a deep understanding of the stress and displacement patterns around the intersections. Three major factors, width of the opening, vertical stress values and various combinations of horizontal stress were simulated in these models. Consequently, the most practical and appropriate bolting system is suggested for general situations.

A summary and discussion of the results obtained from various stages of the research are given in Chapter 8. The most important factors which may significantly influence the stability of roadways and intersections are also reviewed. Finally, some possible measures have been suggested to govern the safety and stability of the roadways and intersections in particular conditions. Figure 1.6 illustrates the structure of the eight chapters in this thesis.
CHAPTER ONE
Stability of Roadways and Intersections

CHAPTER TWO
Theoretical Aspects of Stability of Roadways and Intersections

CHAPTER THREE
Fundamentals of Numerical Modelling

CHAPTER FOUR
New Optimisation Techniques for Finite Element Modelling of Roadways and Intersections

CHAPTER FIVE
Site Investigation and Data Acquisition for Numerical Analysis

CHAPTER SIX
Stability Evaluation of Roadways (2-D FE Models)

CHAPTER SEVEN
Stability Evaluation of Intersections (3-D FE Models)

CHAPTER EIGHT
Conclusions and Recommendations for Future Research

Figure 1.6 Structure of chapters in the thesis.
CHAPTER TWO

THEORETICAL ASPECTS OF STABILITY OF ROADWAYS AND INTERSECTIONS
CHAPTER 2

THEORETICAL ASPECTS OF STABILITY OF ROADWAYS AND INTERSECTIONS

2.1 Introduction

The ability to deal with structural stability problems in roadways and intersections is highly dependent on understanding stress redistribution and the behaviour of strata around these locations. Major factors contributing to stress concentration and pattern of strains in strata surrounding the structures are:

- In situ stress state (virgin stresses)
- Geometry of structure
- Physical and mechanical properties of strata
- Sequence of adjacent mining operations
- Interaction of structural elements; roof, pillar, floor and support system

Since the above factors may vary from mine to mine, it would be difficult to determine a unique pattern of stress and displacement at roadways and intersections which would be applicable to analyse the stability of these structures in all coal mines. This is why the prediction and identification of unstable conditions for these areas can not be achieved with complete confidence.

As many case studies, dealing with site specific problems, fail to address the fundamentals of the failure mechanism, particularly at roadway intersections for all conditions (Peng, 1989; Haramy and Kneisley, 1989; and Beer, Meek and Lucia 1982); further studies on new cases encompassing different conditions seems necessary.

In the past, various approaches have been used to evaluate the stability of roadways and intersections by a number of investigators. Investigations on this subject can be divided into four categories as follows:

- Theoretical or analytical methods
- Experimental methods (Physical Modelling)
- Empirical methods
- Numerical methods
Theoretical methods seek to simplify the problem by making appropriate assumptions as to the statics of the structure and the physical and mechanical properties of the strata associated with the problem. Research using these methods resulted in development of theories such as beam, voussoir beam, plate, and to some extent, arching theories (Beer, Meek and Lucia, 1982).

Experimental methods or physical modelling have been based on laboratory or scaled model studies. Particular difficulties encountered in this method are due to preparing representative materials and large, expensive and complex rig requirements. Equivalent material modelling to predict the stability of underground opening is one approach (Roko and Daemen, 1983).

Empirical methods are basically related to field investigations and experiences encountered in previous projects (Terzaghi, 1946; Barton et al, 1974; and Bieniawski, 1976). Rock mass classification systems are the most important and relevant examples of this method.

Numerical methods which can be subdivided into: the finite element method (FEM), the finite difference method (FDM), the boundary element method (BEM) and the discrete element method (DEM) have been used increasingly in the past 15 years. These methods are used to analyse different structures for stress and displacement and to evaluate the stability of them in various conditions. Considering the number of technical papers published and variety of programs developed, it seems that the FEM is more commonly used in the field of strata mechanics. By using this method, many complex features can be simulated e.g. shape, geological conditions, support system, non-linear geometry and strata behaviour, and the interaction of different elements of the structure (roof, pillar, floor and support system).

This Chapter is concerned with a review of investigations related to stability analysis of roadways and intersections carried out using different methods. The sequence of the subjects is based on the design procedure in strata mechanics, as presented in Chapter 1. Each part will present different theories and results achieved by researchers contributing to the subject.
2.2 In-situ stress (virgin stress)

In order to design an appropriate mine opening with respect to its geometry and orientation, it is essential to have an accurate estimation of the magnitude and direction of the in-situ state of stress. There are several sources of in-situ stress, the most important ones are gravitational and tectonic sources.

It is generally assumed that the vertical compressive stress increases approximately in proportion to depth and density of overlaying strata as expressed by Equation 2.1.

\[
\sigma_v = \gamma \cdot z \quad (2.1)
\]

Where:
- \( \sigma_v \) = vertical compressive stress (MPa)
- \( \gamma \) = a factor depending on the density of overlying strata (MPa/m)
- \( z \) = depth below the surface (m)

Sedimentary strata, such as shales and sandstones of the coal measures, are normally considered to have a specific gravity of approximately 2.3 which gives a vertical pressure of about 0.023 MPa/m depth.

The horizontal stresses \( \sigma_{h1} \) and \( \sigma_{h2} \) generated by vertical stress, disregarding the tectonic stresses, depends on the state of stress of the material. The relationship between \( \sigma_v \), \( \sigma_{h1} \) and \( \sigma_{h2} \) illustrated in Figure 2.1 has been comprehensively discussed by Woodruff (1966). He defined the ratio of horizontal to vertical stress as a coefficient factor which is dependent on the Poisson's ratio, internal angle of friction and cohesion of the material. Accordingly, the ratio of horizontal to vertical stress in various materials is always less than or equal 1.

Figure 2.1 Virgin Stresses.
Horizontal stresses of several times the vertical stress have been recorded, particularly at shallow depths, by many investigators in different locations and geological environments around the world (Worotnicki and Denham, 1976; Brown and Hoek, 1978; Denham et al, 1979; Jeremic, 1981; and Stephanson and Brown, 1988). This evidence proves the existence of other sources of stress in the ground, such as:

- Tectonic stress
- Glacially related stress
- Thermal stress
- Residual stress

As might be expected, despite many attempts, no mathematical theory is available to correlate the in situ stress pattern with depth in the Earth’s crust. However, a number of investigators, such as Kropotkin (1972), Herget (1988), Worotnicki and Denham (1976), Haimson (1978), and Denham et al (1979) established empirical relations that hold good on a regional basis. Among them, Denham and Worotnicki conducted a number of measurements at various sites in Australia (over 1500 measurements of in situ stress at more than 50 locations). They suggested that the average vertical and horizontal stress can be presented by Equations 2.2 and 2.3 as follows:

\[
\sigma_V = 0.027Z \tag{2.2}
\]

\[
\sigma_H = 7.26 + Z (0.0215 \pm 0.0028) \tag{2.3}
\]

Where \( \sigma_V \) and \( \sigma_H \) are the average vertical and horizontal stresses in MPa, respectively, and \( Z \) is the depth at the site of measurement in metres (Brown and Hoek, 1978).

Data collected by Worotnicki and Denham (1976) from different locations in Australia are plotted in Figures 2.2 and 2.3. In Figure 2.2 the vertical stress, \( \sigma_V \), is plotted against depth, \( Z \), and in Figure 2.3 the ratio of horizontal to vertical stress, \( K \), is plotted against depth. The ratio of average horizontal to vertical stress generally lies within limits \( 100/Z + 0.30 \leq K \leq 1500/Z + 0.50 \) and is defined by Equation 2.4 (where \( Z > 400 \) m).

\[
K = (269/Z) + 0.80 \tag{2.4}
\]
CHAPTER 2: Theoretical Aspects of Stability of Roadways and Intersections

Figure 2.2 Vertical stress at different depth in Australia (Data after Worotnicki and Denham, 1976).

Figure 2.3 The ratio of horizontal to vertical stress in Australia (Data after Worotnicki and Denham, 1976).
2.3 Structural analysis of roadways and intersections

Excavations such as roadways and roadway intersections in horizontally bedded strata typically produce rectangular shaped structures with flat backs. In this situation, the formation and structure of the strata dictates the specific stress and displacement fields around the opening. Therefore, appropriate approaches based on structural analysis would be more practical to clarify the behaviour of strata surrounding the opening.

Structurally, roadways and roadway intersections consist of three major elements; i.e. roof, pillars and floor. While the roof stability may have immediate implications on productivity and safety, instability of the pillars and the floor will ultimately affect the stability of the entire structure. In analytical approaches each element is treated independently (Adler and Sun, 1968). For instance, beam or plate theory is used to explain the deflection and failure of roof, pressure arch theory and yield zone theories are used to describe the transference of roof load to pillars, and foundation theory is used to determine the floor reaction to pillar loading.

2.3.1 Structural analysis of the roof

Roof structure theories explain the mechanism of failure as: the immediate roof deflects downward in a beam action, and if it is fractured, a voussoir arch action may take place. If the span exceeds a certain value, the maximum pressure arch width, the voussoir arch will break down; if no artificial support were set before this action occurs, the immediate roof may fail and an elliptic shape of cavity will form in the roof (Adler and Sun, 1968).

(a) Beam and plate theories

Stresses in a single gravity-loaded roof slab overlying an underground opening may be approximated by the stresses developed in a uniformly loaded beam clamped at both ends. However, it is assumed that the thickness is small compared with its lateral dimensions and the length of the opening is more than twice its width.

Equations for deflection, shear and tensile stresses in a clamped beam are given by Equations 2.5 to 2.9.
CHAPTER 2: Theoretical Aspects of Stability of Roadways and Intersections

\[ \delta_{\text{max}} = \frac{WLL^4}{384EI} \quad (2.5) \]

\[ \tau_{\text{max}} = \frac{3WL}{4tcb} \quad (2.6) \]

\[ T_{\text{max}} = \frac{MC}{I} \quad (2.7) \]

\[ M = -\frac{WLL^2}{12} \quad \text{(at the ends of the beam)} \quad (2.8) \]

\[ M = \frac{WLL^2}{24} \quad \text{(at the centre of the beam)} \quad (2.9) \]

Where:

- \( \delta_{\text{max}} \) = maximum deflection (at the centre of the beam) (m)
- \( \tau_{\text{max}} \) = maximum shear stress (at the ends of the beam) (MPa)
- \( T_{\text{max}} \) = maximum tensile stress (at the ends of the beam) (MPa)
- \( C \) = half of the thickness of the beam (0.5 t) (m)
- \( I \) = second moment of the cross section about the neutral axis (m^4)
- \( t \) = thickness of the beam (m)
- \( b \) = breadth of the beam (m)
- \( E \) = elastic modulus (MPa)
- \( W \) = load per unit length of the beam (MN/m)
- \( L \) = length of the beam (m)
- \( M \) = moment (MN.m)

Distribution of moments and shear stress are shown in Figure 2.4. It can be seen that at the centre of the beam, the shear stress is zero and the tensile stress is one-half the maximum value. Thus, the point of initial failure would be expected to occur at the ends of the span, rather than at the centre. This has been proved by field observations at Ellalong Colliery (Hematian and Porter 1993b) and also reported by other investigators (Hanna et al, 1985 and 1986a).
When the immediate roof consists of two or more members and the lowest member is loaded by the upper member, the increase in load on the lowest member is considered to be an increase in the density of the lowest member. This "apparent" density is given by Equation 2.10 (Woodruff, 1966).

\[
\gamma_a = \frac{E_i t_i \sum_{i=1}^{n} \gamma_i t_i}{\sum_{i=1}^{n} E_i t_i^3}
\]  

(2.10)

Where:

\( \gamma_a \) = apparent density of the lowest member (t/m^3)
\( \gamma_i \) = density of each member (t/m^3)
\( E_i \) = elastic modulus of each member (MPa)
\( t_i \) = thickness of each member (m)

If \( W \) is calculated by \( W = \gamma_a \cdot t_i \), then the equations previously presented could be used to determine the maximum tensile stress and maximum deflection of the lowest member of the multi-member immediate roof.
The apparent density theory calculates the additional load of the upper member onto the lowest beam based on the elastic modulus and thickness of the beams. Hematian (1989) proposed the "active roof" theory which is a more realistic approach to evaluate the load transference between overlying strata (beams).

Active roof is an equivalent stratum for the immediate roof whose thickness and elastic modulus are the same as immediate roof but whose load is calculated by Equation 2.11.

\[ W = W_{im} + \Delta W \quad (2.11) \]

Where \( W_{im} \) is the uniform weight of the immediate roof and \( \Delta W \) is the uniform load transferred to the immediate roof through the deflection of overlying strata. The amount of additional load, \( \Delta W \), depends on the structural properties of the strata. The additional load can be calculated by Equation 2.12 (when deflection of the upper stratum is more than that of the lower one).

\[ \Delta W = \frac{D_L W_U - D_U W_L}{D_L + D_U} \quad (2.12) \]

Where:

- \( W_U, W_L \) = uniform weight of upper and lower beam, respectively
- \( D_U, D_L \) = deflection rigidity factor (= E.I) of upper and lower beam, respectively

For a multi beam (strata) formation, the above equation must be solved in an iterative procedure beginning from the lowest strata and extending to the upper strata.

When considering support techniques, the movement and displacement of the roof strata have the same degree of importance as that of load distribution; the movement of the roof strata may be appear in the form of sagging, slipping or a combination of both. The former happens when the tensile stress perpendicular to the strata exceeds the tensile strength of the bedding plane. On the other hand, the latter occurs when the shear stress between two strata is greater than the shear strength of the bedding plane (Figure 2.5).
Investigations (Hematian and Porter, 1993b and Stimpson, 1983) into roof movement modes have been carried out by theoretical and experimental methods. Both approaches showed that the maximum amount of slip between layers takes place at a distance of approximately 0.13 to 0.21 times the span from the side walls (Hematian and Porter, 1993b). These results suggested that installing fully grouted bolts at the location of maximum slip will reduce the required number of bolts.

Results from sensitivity analysis (Stimpson, 1983), which considered the influence that different parameters had on the location of maximum slip can be summarised as follows:

1- Location of maximum slip moves away from the abutment as:
   (a) Roof span increases
   (b) Modulus of elasticity of the abutment increases
   (c) Poisson's ratio of the abutment increases

2- Location of maximum slip moves towards the abutment as:
   (a) Modulus of elasticity of roof beam increases
   (b) Poisson's ratio of roof beam increases

3- As the thickness of the abutment increases the location of maximum slip, at first, moves away from the abutment but then moves back towards the abutment.

4- Location of maximum slip is unaffected by the density of the roof beam.

5- Location of maximum slip is over the abutment as roof thickness increases beyond 1 meter (Figure 2.6).
When the length of opening is less than twice the width, the roof must be considered as a plate or slab. Theoretical studies by Hematian (1989) showed that in a gravity-loaded plate, when length / width ratio, $L / W$, decreases the load distribution across the length of the span, $Q_L$, gets close to the load distribution across the width, $Q_W$. Figure 2.7 shows the variation of $Q_W / Q_L$ with $L / W$ ratio.
If the roof is considered as a gravity-loaded plate (with length / width ratio less than 2.0) the maximum tensile stress may be calculated by Equation 2.13 (Woodruff, 1966).

$$T_{max} = \frac{\gamma L_p^2 B}{t}$$  \hspace{1cm} (2.13)

Where:

- $T_{max}$ = tensile stress (MPa)
- $L_p$ = shorter span of the plate (m)
- $\gamma$ = density of the plate (MN/m$^3$)
- $t$ = thickness of the plate (m)
- $B$ = a constant whose value depends on the length / width ratio and also on Poisson’s ratio. If Poisson’s ratio is taken 0.33 then $B$ varies from 0.0513 to 0.0829 for length / width ratios from 1/1 to 2/1, respectively.

(b) Vousoir beam theory

In 1941, the concept of the vousoir beam was proposed to explain the stability of mine roofs containing many transverse fractures. This theory was developed by Evans, and modified and extended by Beer and Meek (1982). Vousoir beam theory recognises the fact that in a confined situation the ultimate strength of a beam is larger than its elastic strength and that pre-existing cross fractures may not allow tensile stresses. This theory, thus, assumes that the beam consists of a material without any tension and carries its own weight by arching effect (Brady and Brown, 1985).

The vousoir beam model for a roof bed is illustrated in Figure 2.8(a), and the forces operating in the system are defined in Figure 2.8(b). The essential idea conveyed in the figures is that the section of the beam transmitting lateral load is assumed to be approximated by the parabolic arch traced on the beam span (Figure 2.8c).

The vousoir roof beam supports its own weight, W, by vertical deflection and induced lateral compression. There are three possible failure modes for vousoir beams as follows:

(a) Shear at the abutment when the limiting shear resistance, $T \tan (\Phi)$, is less than the abutment vertical reaction, $V (= 0.5 W)$,
(b) Crushing at the hinges formed in the beam crown and lower abutment contacts,
(c) Buckling of the roof beam with increasing eccentricity of lateral thrust, and a consequent tendency to form a 'snap-through' mechanism. (Beer, Meek and Lucia, 1982; Roko and Daemen, 1983).

Figure 2.8 Voussoir beam model and forces operating in this system
(After Brady and Brown, 1985).

For cases where the opening has span and length dimensions not compatible with the plane strain conditions (length / width ratio less than 2.0; the same condition as at roadway intersections) it is necessary to consider the roof as a plate and to account for the development of cracks in the roof plate. The cracks are directly analogous to the yield lines postulated in the behaviour of reinforced concrete slabs (Brady and Brown, 1985).

The idea of the voussoir beam can be extended to accommodate the more general geometry of the slabs of the immediate roof. The solution procedure based on the relaxation method and a computer program to solve such problems were presented by Beer and Meek, 1982. They have written that the computer program allows the computation and plotting of curves of limiting span as a function of strata thickness for hanging-walls or roofs. Design curves have been determined for long excavations under plain strain conditions as well as for square and rectangular rooms. The main parameters
considered are the strata thickness, the dip angle, the elastic modulus and compressive strength.

Although it seems that the voussoir plate theory could be applied to analyse the stability of roadway intersections, there are two important differences between the conditions assumed in voussoir plate theory and those that exist at roadway intersections. Firstly, the theory assumes that the plate is clamped on all four sides, but at roadway intersections, the roof stratum is clamped at its four corners. Secondly, the theory assumes that the load on the plate is limited to the weight of plate, but in real conditions the deflection of overlying strata and lateral stresses induce additional load on the immediate roof.

(c) Arching theory

The arching action refers to the natural process by which a fractured material acquires a certain amount of ability to support itself partially through the resolution of the vertical component of its weight into diagonal thrust. As shown in Figure 2.9, the lower element A, has little tendency to support itself, because as it moves downward it creates little deformation along the reaction lines AD and AE. Consequently, little stress is created along these lines and there is almost no vertical component resisting downward movement of this element. When element C attempts to move downward, it creates a great deal of strain (shortening) along reaction lines CD and CE; therefore, a great deal of stress. By this explanation, it is clear that the arching action does not signify the shape of the opening but is a means of spanning an opening by resolving vertical pressure into horizontal or diagonal thrust.

![Figure 2.9 Arching action (After Adler and Sun, 1968).](image)

It is evident that a relatively small force applied upward on the plane DE will stabilise the ground. This force needs only be sufficient to prevent failure (by shearing) of the potential unstable rocks under this arch shape zone. At roadway intersections, field investigations by Peng and Okubo (1978a) revealed that this zone is spatially dome shape (This problem is further discussed in a subsequent section in this chapter). There have
been many attempts to estimate and to predict the shape and height of the arch zone. The height of the arch zone may be used to estimate the load height to design the required support system.

A number of models have been suggested to clarify the geometry and dimensions of the arch zone by many investigators (Unrug and Szwilski 1982; Protodyakonov and Everling, quoted by Biron and Arioglu, 1983; and different rock mass classifications described by Hoek and Brown, 1980). Amongst the many rock mass classification methods, the RMR and Q systems, based on a large number of case histories are the most comprehensive systems. Each of these systems could be used to estimate the pressure on the support system and assess the stability of the openings (Hoek and Brown, 1980).

Two main equations provided by the Norwegian Geotechnical Institute (NGI) classification, Q system, are those for permanent support pressure and safe unsupported excavation span, Equations 2.14 and 2.15 (Sheorey, 1991).

\[
P = \frac{20}{J_r} Q^{0.33} \quad (2.14)
\]

\[
B = 2 ESR . Q^{0.4} \quad (2.15)
\]

Where:

- \( P \) = Pressure (tonnes / m²)
- \( J_r \) = Joint roughness number
- \( Q \) = The rock mass quality
- \( B \) = Safe unsupported span (m)
- \( ESR \) = The excavation support ratio; between 3 and 5 for mine openings

Barton et al. recommended Equations 2.16 and 2.17 to calculate pressure for temporary supports and for safe unsupported span of roadway intersections:

\[
P = \frac{20}{J_r} (5 Q)^{0.33} \quad (2.16)
\]

\[
B_i = 2 ESR 
\left( \frac{Q}{3} \right)^{0.4} \quad (2.17)
\]

Where \( B_i \) is the diagonal distance between pillar corners at roadway intersection.
The application of Equation 2.14 to a few case studies in India by Sheorey (1991) gave very high support pressures. Also, a few trial calculations showed that when ESR was taken between 3 and 5 in Equations 2.15 and 2.17 unrealistic roadway spans were obtained. As a result of this, Sheorey modified the existing NGI classification for coal measures in India. For example, he decided to use a ESR = 1.6 and to double the original value of Q which may then be reduced further for unfavourable joint orientations or horizontal stress direction, or special structures. It is claimed that these adjustments made the performance of the NGI system more realistic as it was seen from the case studies of stable and unstable roadways and roadway intersections.

Another empirical equation was introduced by Unal (1986), which relates the load height, h, to a quantitative rock quality index, RMR, and roof span, B, as following:

\[ h = \frac{100 - RMR}{100} \times (B \text{ or } B') \]  

Where:

- \( h \) = load height (m)
- \( RMR \) = rock mass rating
- \( B \) = span of roadway (m)
- \( B' \) = diagonal distance between pillars at roadway intersection (m)

The plots of the load height as a function of roof span and RMR is presented in Figure 2.10. It is said that this equation had been developed based on a large number of case studies and a comparison of empirically calculated load height from field investigations and numerical modelling (Unal, 1986).

(d) Mechanism and classification of roof failures

Failure of the roof in roadways and at intersections may be divided into bedding plane failure, arch failure in the roof and guttering over the rib edges. Each of these failures may have a tensile or shear failure mechanism.

Bedding plane failures are due to relative movements of strata on both sides of the bedding plane. Sagging and slipping are the most common types of strata movement contributing to failure on the bedding plane. When the sag of the lower stratum is more than that of the upper one, then the two strata tend to separate from each other due to tensile stress created perpendicular to the bedding plane. In bending condition, when the
structural and mechanical properties of two overlying strata are different, and the pattern of deflection is such that separation does not occur, then an appreciable shear stress is created on the bedding plane. In this condition shear failure may occur.

![Figure 2.10 Load height as a function of roof span and RMR (After Unal, 1986).](image)

According to elastic theories, in a rectangular opening, the maximum tensile stress occurs at the mid-span of the roof line (simply supported beam). If the tensile stress at the mid-span of the roof-line exceeds the rock strength, a tensile crack parallel to the direction of the entry will develop leading eventually to sloughing, arch or dome type roof failures. This depends on rock type and method of support. The maximum shear stress at the intersection occurs above the roof and rib-lines of a rectangular roadway. A shear type of roof fall (commonly called the gutter roof) involves a structural failure bound on two sides by failure planes parallel to the rib-lines but extending vertically or nearly so into the roof (Su and Peng 1987). The top side of the fall may either be dome or arch shaped.

Classifying roof falls can help in identifying the mechanisms of roof failure, and thus can be used as a tool in prediction studies. There are many methods of roof fall classification proposed by different investigators (Pothini, 1978; Patrick and Aughenbaugh, 1979; Peng, 1986; Vervoort, 1990 and Chase and Mark, 1990). The shape, size, time interval between active mining and the roof fall, and the effect of geological features are the basis
for the suggested classifications. However, practical and numerical studies (Dougherty 1971, Ellison 1978, Hobbs 1986, McNabb and Wardle 1986 and Krese 1988) have revealed that in addition to the above factors, the high vertical stress, excess horizontal stress, relative stiffness between coal and its immediate roof and gas pressure have significant effect on the roof failure mechanism.

Gale and Blackwood (1987) and Jeremic (1981) conducted studies of roof failure due to high horizontal stresses. In high lateral stress fields roadway stability is significantly influenced by the orientation of the face with respect to the in situ principal stresses. Figure 2.11 illustrates tension fracture locations in roadways with various drivage angle to the maximum principal stress, $\theta_{sr}$, (Gale and Blackwood, 1987).

![Figure 2.11 Tensile fractures in roadways with various drivage angle ($\theta_{sr}$) to $\sigma_{hmax}$](image)

To summarise the results obtained by the above investigators, three conditions for roadways with respect to horizontal stress can be defined as follows:

(a) Roadways driven perpendicular to the direction of major lateral stress are likely to exhibit the worst conditions. These roadways experience the majority of roof falls, extrusion of coal ribs into the openings and guttering along the rib-sides.
(b) Roadways parallel to the major lateral stress are generally in the best conditions. They may exhibit roof sagging and bed separation and sometimes uneven floor heave.
(c) To reduce the problem, it is sometimes useful to drive long headings in the best direction (parallel to the major horizontal stress) and to accept the more difficult conditions in cut-through headings. Alternatively, a direction can be determined such that an acceptable level of roadway deterioration is expected in both directions.
(e) Roof control and support design techniques

The function of different support systems in underground structures can be performed from active or passive techniques. The main purpose of any active support system is to assist the rock mass in supporting itself by building a ground arch and mobilising the optimum shear strength of the rock. Indeed, it is the surrounding rock, not the supporting system, which takes the majority of the excess stress due to mining. The support merely helps the rock to support itself. However, passive support systems are supposed to carry the total weight of an unstable zone as a dead load. This load may be transferred either to the upper stable strata (in the case of using point anchor bolts) or to the floor (in the case of using steel arches or props and caps).

Rock reinforcement, particularly at roadway intersections, by using rock bolts and cable bolts is a common practice to strengthen strata about the intersection to sustain greater stresses and increase the strength of the structure. The function of rock bolts and dowels are mainly as follows:

1- To enable the rock to support itself by either:
   (a) forming a stronger beam from individual laminae; or
   (b) increasing the internal friction of the rock so that competent stratum is less likely to break, and broken strata may more readily form an arch; or
   (c) pre-stressing the rock, working in the same manner as pre-stressing reinforced concrete.

2- To prevent sliding across bedding planes in strongly laminated or slickensided strata.

3- To suspend larger, single blocks from competent strata above.

From structural point of view the functions mentioned above can be classified into four types as frictional effect, arching effect, key effect and suspension effect (Figure 2.12). A number of researchers have conducted theoretical, practical, laboratory and numerical studies to investigate the mechanisms and theories of roof bolting (Roko and Daemen 1983, Jeffrey and Daemen 1983, Snyder 1983, Stimpson 1983, Dhar et al 1983, Richmond et al 1984, Peng 1986, Gale et al 1988, and Indraratna and Kaiser 1990a,b).
2.3.2 Structural analysis of pillars

Coal pillars are widely used in underground coal mines both in room-and-pillar and longwall mining methods. This portion of coal is left in place to support the overlying strata and to maintain the integrity of openings. From the stability point of view, the performance of pillars in underground structures has a significant effect on the overall stability of the whole structure.

Investigations relating to the stability of pillars can be divided into two parts: (i) stress distribution pattern in pillars, (ii) in-situ strength of pillars. Almost, all the investigations have indicated that parameters such as: depth of cover (H), unit weight of overburden (γ), entry width (W₀), pillar length (Lₚ), pillar width (Wₚ), height of the coal seam extracted (h) and the mechanical properties of coal have a great influence on the stress distribution pattern and in-situ strength of the pillar. The process of designing a pillar, therefore, is to determine a proper size of pillar to suit the location and mining conditions and to develop a relationship between design parameters in order to optimise the pillar size.
2.3.2.1 Stress distribution within pillars in longwall developments

In longwall developments, there are two major loading conditions over pillars: (i) uniformly distributed and static loading conditions in main entries, (ii) unevenly distributed and dynamic loading conditions in gate entries.

Loading condition on chain pillars in main entries is more likely to be static and the simplest approach to govern it is the tributary area loading concept (Obert and Duvall, 1967). In this concept, it is assumed that the area supported by a pillar covers the area above it and the neighbouring area tributary to it (Figure 2.13).

\[
\sigma_p = \sigma_v \times \frac{(W_p + W_o) (L_p + W_o)}{(W_p \times L_p)}
\]  

(2.19)

When \( W_p = L_p \) (square pillar), then:

\[
\sigma_p = \sigma_v \times \left(1 + \frac{W_o}{W_p}\right)^2
\]  

(2.20)

Where:

- \( \sigma_p \) = average pillar load (MPa)
- \( \sigma_v \) = virgin vertical stress (MPa)
- \( W_o \) = width of entry (m)
- \( W_p \) = width of pillar (m)
- \( L_p \) = length of pillar (m)
In the second situation the pillars in gate entries are subject to high dynamic abutment loads resulting from adjacent working longwalls. This condition causes many ground control problems, particularly in tail entries where the pillars are located between two gob areas. In this case, an adequate chain and / or rib pillar is necessary to maintain the stability of the opening.

In general, the total load on the pillars in gate entries is the sum of the weight acting directly from the overburden plus the extra load transferred from the adjacent goaf. There have been two major approaches to estimate the load. The first was proposed by King and Whittaker (1970); accordingly, the load is considered uniform and static as shown in Figure 2.14. The magnitude of this load is mainly associated with the depth of the panel and caving characteristics of the strata overlying the working area. The strata are categorised as 'soft' or 'hard'. The soft strata condition means that the sedimentary rocks consist of relatively homogeneous low strength material and the average angle of shear, \( \beta \), is about 30°. In this condition, caving tends to form a triangular goaf area above the panel. The hard strata condition means that the sedimentary rocks are relatively strong and thick (\( \beta \) is about 15°). Caving, in this condition, forms a trapezoidal goaf area above the panel depending on the depth of the panel. From the subsidence point of view, the former condition (small B/H ratio and large angle of shear Figure 2.14 b) induces a sub-critical subsidence which results in a small surface subsidence. The latter condition (large B/L ratio and low angle of shear) induces a super-critical subsidence which results in the maximum possible subsidence.

With respect to Figure 2.14 the average pillar load can be calculated by the following equations:

\[
\sigma_p = \sigma_v \left[ \left( n \cdot W_p + (n - 1) \cdot W_o \right) + H \cdot \tan(\beta) \right] \frac{W_o + L_p}{W_p \times L_p} \]

(2.21)

when \( B \geq 2H \cdot \tan(\beta) \)

\[
\sigma_p = \sigma_v \left[ \left( B + n \cdot W_p + (n - 1) \cdot W_o \right) - 0.25 \frac{B^2}{H \cdot \tan(\beta)} \right] \frac{W_o + L_p}{n \cdot (W_p \times L_p)}
\]

(2.22)

where:

- \( \sigma_p \) = average pillar load (MPa)
- \( \sigma_v \) = virgin vertical stress (MPa)
- \( W_o \) = width of entry (m)
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\[ W_c = \text{total width of gate entry (m)} \]
\[ W_p = \text{width of pillar (m)} \]
\[ L_p = \text{length of pillar (m)} \]
\[ B = \text{width of panel (m)} \]
\[ H = \text{depth of cover (m)} \]
\[ n = \text{number of chain pillars across the gate entry} \]
\[ \beta = \text{average angle of shear (degrees)} \]

Figure 2.14 Load imposed on gate entry pillars according to King and Whittaker, 1970.

Field investigations have shown that abutment pressures on gate entries resulting from longwall faces are dynamic, not uniform and static. Wilson (1977, 1983) developed an analysis of vertical stress around a longwall panel. This analysis uses a stress balance method in which the total vertical force applied over a large plan area must remain equal to that caused by the overburden, even after part of the seam has been removed. It is assumed that, compared to the total vertical force to be redistributed, the vertical forces transmitted by the roadway supports are small and may be ignored.
Figure 2.15 depicts the hypothetical decay of stress around a longwall face (Wilson 1983). The distribution of vertical stress postulated in the caved waste is based on an assumption that the stress reaches the overburden stress at a distance of 0.3 H from the rib-side. Table 2.1 sets out the values calculated by Wilson for the vertical stress, the peak abutment stress, the width of the rib-side yield zone, and the total vertical force carried by the yield zone.

Where:

- \( \sigma_v \) = virgin vertical stress = \( \gamma H \) (MPa)
- \( \sigma_y \) = peak abutment pressure on pillar (MPa)
$\sigma_z$ = vertical stress (MPa)
$C_o = \text{in-situ uniaxial compressive strength of coal seam (MPa)}$
$b = \text{constant in equation, } \sigma_1 = C_o + b.\sigma_3$
$m = \text{height of the extraction (m)}$
$H = \text{depth of cover (m)}$
$x = \text{distance from rib-side (m)}$
$\chi_d = \text{width of yield zone (m)}$
$p = \text{support pressure, } p_i, \text{ plus the unconfined compressive strength of the broken material at the rib side, taken as 0.1 MPa, or } p = p_i + 0.1 \text{ MPa}$

Table 2.1 Equations for calculating vertical stress distributions at rib-sides (roof, seam and floor of similar strength) (Bieniawski, 1992)

<table>
<thead>
<tr>
<th>Equation Type</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical stress</td>
<td>$\sigma_z = bp \left( \frac{2x}{m+1} \right)^{b-1}$</td>
</tr>
<tr>
<td>Peak abutment or yield stress</td>
<td>$\sigma_Y = C_o + b.\sigma_v$</td>
</tr>
<tr>
<td>Width of yield zone</td>
<td>$\chi_d = \frac{m}{2} \left[ \left( \frac{\sigma_v}{p} \right)^{b-1} - 1 \right]$</td>
</tr>
<tr>
<td>Vertical force carried by the yield zone</td>
<td>$A_b = \frac{m}{2} \times p \left[ \left( \frac{\sigma_v}{p} \right)^{b-1} - 1 \right]$</td>
</tr>
</tbody>
</table>

2.3.2.2 Pillar strength

In the past, many investigations have been carried out to develop equations for estimating the in-situ strength of pillars. Each formula specifies its own appropriate factor of safety. Laboratory tests (Obert and Duvall, 1967; and Holland, 1962), large scale in-situ tests (Bieniawski, 1968 and 1983), case studies (Salamon and Munro, 1967) and numerical modelling (Pariseau, 1977; Bensehamdi, 1989; and Wu and Salamon 1992) have been the major techniques to derive these equations.

From all the available formulae for estimating pillar strength, five empirical equations are most commonly used. These formulae plotted in Figure 2.16 are summarised as follows:
Obert-Duvall Formula: This formula is derived from laboratory tests on hard rocks and elasticity considerations, and it is given as:

$$\sigma_p = \sigma_1 \left( 0.778 + 0.222 \frac{w_p}{h} \right)$$

(2.23)

where $\sigma_p$ (in MPa) is pillar strength in and $\sigma_1$ (in MPa) is the uniaxial compressive strength of a cubic specimen of critical size (e.g. 1.0 m, Bieniawski 1992). $w_p$ and $h$ are the width and height of the pillar (in m), respectively. According to Obert and Duvall, this equation is valid for $w_p / h$ ratios of 0.25 to 4.0, assuming gravity-loading conditions. Through back calculation from mining case histories and utilising laboratory rock properties, safety factors of 2 to 4 were derived for short- and long-term pillar stability, respectively (Bieniawski 1992).
**Holland-Gaddy Formula:** This equation is given as:

\[
\sigma_p = \sigma_c \sqrt{D} \times \left( \frac{\sqrt{w_p}}{h} \right)
\]  

(2.24)

where \(\sigma_p\) (in MPa) is pillar strength and \(\sigma_c\) (in MPa) is the uniaxial compressive strength of rock specimen tested in the laboratory having a diameter or cube dimension \(D\) (in inches). \(w_p\) and \(h\) are the width and height of the pillar, respectively. This equation is valid for \(w_p / h\) ratios of 2 to 8. Holland specified a safety factor between 1.8 and 2.2 for design of coal pillars.

**Holland Formula:** Holland provided a different expression for the strength of coal pillars as given by:

\[
\sigma_p = \sigma_1 \times \left( \frac{w_p}{h} \right)
\]  

(2.25)

where \(\sigma_p\) (in MPa) is pillar strength and \(\sigma_1\) (in MPa) is uniaxial compressive strength of a cubic specimen (\(w_p / h = 1\)) of critical size. \(w_p\) and \(h\) are the width and height of the pillar (in m), respectively. The recommended safety factor for this formula is 2.0.

**Salamon-Munro Formula:** This formula is based on case studies conducted by Salamon and Munro in South Africa. They selected the following form of pillar strength to apply to square pillars;

\[
\sigma_p = 7.2 \times \left( \frac{w_p^{0.46}}{h^{0.66}} \right)
\]  

(2.26)

where \(\sigma_p\) is pillar strength in MPa, \(w_p\) and \(h\) are the width and height of the pillar (in m), respectively. The recommended safety factor for this formula is 1.6, the range being 1.31 to 1.88.

**Bieniawski Formula:** This formula is based on large-scale in-situ tests on coal pillars conducted by Bieniawski in the USA. The general normalised form of the Bieniawski equation is:

\[
\sigma_p = \sigma_1 \times \left( 0.64 + 0.36 \frac{w_p}{h} \right)
\]  

(2.27)
where \( \sigma_p \) (in MPa) is pillar strength and \( \sigma_1 \) (in MPa) is uniaxial compressive strength of a cubic specimen at critical size. \( w_p \) and \( h \) are the width and height of the pillar (in m), respectively. A safety factor of 1.5 is recommended for short term applications (e.g., in the panels), while a safety factor = 2.0 should be used in the main entries and when pillar recovery on the retreat is contemplated.

It is apparent from Figure 2.16 that the Holland-Gaddy formula is the most conservative one while Bieniawski's formula predicts the highest strength. As it was mentioned, the recommended safety factor varies for different formulae when designing a certain pillar (Bieniawski, 1992).

### 2.3.3 Structural analysis of the floor

Floor stability problems in underground coal mines have existed for many years, yet little theoretical effort (in comparison to that carried out on roof and pillars) has been made to consider the structural behaviour of floor strata in the mine design process. Floor failure may occur in the form of floor heave, pillar punching, floor buckling or a combination of these. In this regard, the mechanical properties of the immediate floor and the in-situ horizontal stress have a significant influence on the interaction between the floor and pillar. When the immediate floor is weak, it fails easily and pillars punch into it. The failed material, then moves into the opening. In the situation that the immediate floor is very strong with respect to the coal pillar, a high stress concentration zone is developed around the toe of the pillar causing severe crushing on the pillar toe. This phenomenon has been investigated by means of finite element modelling of roadways at Ellalong Colliery (Hematian and Porter, 1993b).

Brady and Brown (1985) recommended the expressions given by Hansen in 1970 to calculate the bearing capacity, \( q_u \), of cohesive and frictional materials such as soft rocks. Bearing capacity is expressed in terms of pressure or stress. For uniform strip loading on a half space, bearing capacity is given by classical plastic analysis as given in Equation 2.28.

\[
q_u = 0.5 \gamma W_p N_\gamma + C N_c \tag{2.28}
\]

Where:
- \( \gamma \) = density
- \( W_p \) = width of footing (width of pillar)
- \( C \) = cohesion of floor stratum
CHAPTER 2: Theoretical Aspects of Stability of Roadways and Intersections

*Nc* = bearing capacity factor defined by Equation 2.29

*Nγ* = bearing capacity factor defined by Equation 2.30

\[
N_c = (N_q - 1) \cdot \cot (\phi) \tag{2.29}
\]

\[
N_\gamma = 1.5 (N_q + 1) \cdot \tan (\phi) \tag{2.30}
\]

where \(\phi\) is friction angle of the loaded medium and \(N_q\) is given by

\[
N_q = e^\pi \cdot \tan (\phi) \cdot (\tan \left( \frac{\pi}{4} + \frac{\phi}{2} \right))^2 \tag{2.31}
\]

Equation 2.28 describes the bearing capacity developed under a long rib pillar. For pillars of length \(L_p\), the expression for bearing capacity is modified to reflect the changed pillar plan shape as defined by Equation 2.32.

\[
q_u = 0.5 \gamma W_p N_\gamma S_\gamma + C \cdot \cot (\phi) N_q S_q - C \cdot \cot (\phi) \tag{2.32}
\]

where \(S_\gamma\) and \(S_q\) are shape factors defined by

\[
S_\gamma = 1.0 - 0.4 \left( \frac{W_p}{L_p} \right) \tag{2.33}
\]

\[
S_q = 1.0 - \sin (\phi) \cdot \left( \frac{W_p}{L_p} \right) \tag{2.34}
\]

The safety factor against bearing capacity failure, \(q_u / \sigma_p\), is recommended to be greater than 2.0, since it is assumed that \(\sigma_p\) is uniformly distributed to the floor strata.

2.4 Specific investigations on the stability of intersections in the USA

In 1961, the first attempt to examine intersection support problems more thoroughly was made by Stahl (1961). It was reported that 30 percent of fatalities in underground coal mines had occurred at roadway intersections, even though the exposed roof at these locations rarely exceeded 15 percent of the total exposed roofs. This figure was recorded again by Peng in 1978a.

In the investigation by Stahl, a few tests were made to determine roof action at the intersection. This was to ascertain if impending roof collapse could be predicted. He used sag bolts to measure vertical movement of the strata along the bolt. Most of these bolts
were placed near the centre of the intersection, and a few were placed midway between intersections to determine if sag continued between intersections. No theoretical calculations were made by the investigator, but the following remedies were suggested:

(a) extra bolts at corners that have been trimmed,
(b) cross bars where scaly roof is encountered, and
(c) occasional beams at the approach to an intersection, especially if a crack appears, indicating a shear action along the pillar.

During 1974 to 1978 two major investigations on the stability of roadway intersections were conducted by Langland (1976 and 1978), and by Peng and Okubo (1978a,b). The former project was based on analytical and numerical modelling together with field investigations and laboratory testing of samples to obtain material properties. The latter project which was really two different works, one on three-way intersections and the other on four-way intersections, was carried out through numerical modelling and empirical studies and looked particularly at the shape and dimension of roof falls.

The objective of the project carried out by Langland was to analyse coal mine roof behaviour and particularly to evaluate the support requirements for cross cuts in room and pillar mining. The field investigations included:

- the relative displacement of roof strata (sag),
- the relative displacement between floor and roof (convergence), and
- the total load on selected roof bolts.

These measurements were conducted in the Meigs No. 2 Coal Mine of Southern Ohio Coal, Athens, Ohio by the US Bureau of Mines. Computations were performed using the finite element code, ROCK3D, developed for the US Bureau of Mines. Limitations of the FEM’s were due to the lack of consideration of a realistic failure criterion and no consideration of post-failure behaviour in the code. Also, as it was mentioned above, this investigation was limited to room and pillar mining. Conclusions from this investigation are summarised as follows:

(a) The sag and bolt load changes were a maximum at the corners of the intersection and along the sides of the pillars where shear deformation was the greatest. They were a minimum in the centre.
(b) The maximum convergence occurred in the centre of the intersection and was less in the cross-cuts.
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(c) The convergence rate was highest during the first few days after excavation, then after approximately 20 to 30 days, convergence rate was reduced. The precise mechanism that caused convergence was not well understood. Pillar softening, floor heave or a combination of these might have been responsible.

(d) Mining method (sequence of operation) had a great influence on the behaviour of the structure.

Investigations by Peng and Okubo (1987a and 1978b) were focused on the behaviour of three- and four-way intersections without considering mining operations around the intersection. Their investigations were based on empirical studies of roof falls at intersections and numerical modelling of intersections.

Field data of roof falls at roadway intersections were collected from an underground coal mine 180 m below the surface. The coal seam was the Pittsburgh Seam with an average thickness of 2.4 m. The entry width was 6.0 m and total of 22 roof falls were surveyed during the data collection period. The specifications of roof falls are summarised in Table 2.2.

<table>
<thead>
<tr>
<th></th>
<th>Maximum (m)</th>
<th>Minimum (m)</th>
<th>Average (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{\text{max}}$</td>
<td>16.0</td>
<td>3.3</td>
<td>8.49</td>
</tr>
<tr>
<td>$D_{\text{min}}$</td>
<td>9.0</td>
<td>2.7</td>
<td>5.67</td>
</tr>
<tr>
<td>Height</td>
<td>3.3</td>
<td>1.2</td>
<td>2.50</td>
</tr>
</tbody>
</table>

($D_{\text{max}}$ and $D_{\text{min}}$ are maximum and minimum diameter of roof fall bottom, respectively)

Based on the above observations, Equation 2.31 was proposed to define the nominal bottom dimension, $D_n$, which is an estimation of the average diameter of the roof fall bottom;

$$D_n = \sqrt{D_{\text{max}} \cdot D_{\text{min}}}$$  \hspace{1cm} (2.35)

Then the nominal dimension was related to the height of the roof fall, $H_f$, by:

$$H_f = 0.37 \ D_n$$  \hspace{1cm} (2.36)

It was mentioned in the report that the average nominal dimension for all of the roof falls was about 6.81 m, and the average height of fall calculated by Equation 2.36 was in agreement with that recorded by field data; about 2.5 m. The FE program, NASTRAN,
was used for numerical modelling in this investigation. The models included only one material property, and a linear elastic solution was utilised to analyse the problem. Conclusions made from this investigation could be summarised as follows:

(a) Analysis of field data revealed an arching zone within which roof falls occurred. The boundary of this zone which requires support was defined by the contour line $\sigma_z = 0.1\sigma_v$ where the $\sigma_z$ is the vertical stress at the mid-height of the coal seam and $\sigma_v$ is the overburden stress (Figure 2.17).

(b) The maximum vertical stress at mid-height of the coal seam occurred around the corners of the pillar, while the minimum value occurred at the centre. The maximum and minimum values increased with an increase in entry width (Equation 2.37).

\[
\sigma_{z_{\text{max}}} = 1.27 \sigma_{\text{ave}} \tag{2.37}
\]

where:

- $\sigma_{z_{\text{max}}}$ = maximum vertical stress (MPa)
- $\sigma_{\text{ave}}$ = average vertical stress (MPa) defined by Equation 2.38

\[
\sigma_{\text{ave}} = \left(\frac{w_e + w_p}{w_p}\right)^2 \times \sigma_v \tag{2.38}
\]
where:

\[ W_e = \text{width of entry (m)} \]
\[ W_p = \text{width of pillar (m)} \]
\[ \sigma_v = \text{virgin vertical stress (MPa)} \]

(c) Maximum displacement, \( d_{\text{max}} \), at the roof line occurred at the centre of the intersection, and it was calculated by Equation 2.34.

\[
d_{\text{max}} = 1.47 \times \frac{H_e}{E_c} \times \sigma_{\text{ave}} \tag{2.39}
\]

Where:

\( d_{\text{max}} \) = maximum displacement (m)
\( H_e \) = the entry height (m)
\( E_c \) = elastic modulus of coal pillar (MPa)
\( \sigma_{\text{ave}} \) = average vertical stress (MPa)

The minimum displacement occurred at the centre of pillar and at a height approximately one entry width above the roof line. The difference between maximum and minimum displacement became negligible at this location.

(d) Small tensile stresses were induced at the centre of the intersection and along the centre-line of the entry. Compressive stress increased towards the pillar and reached a maximum value near the ribs.

Peng and Okubo (1978a) suggested that if the rock strata within the arch zone breaks and requires immediate support, the roof bolt patterns should be designed to carry the rock weight in the zone. The idea would be to suspend the broken rock from the overlying intact main roof. Figure 2.18 shows the proposed bolt length distribution pattern for a 6 m wide entry, assuming that at least 0.3 m of the bolt must be anchored in the strata outside the arch zone.
If there are many thinly laminated strata in the immediate roof, the roof bolting is by beam building, based on the principal of increasing the frictional shear resistance between the inter-strata beddings and to reduce or prevent inter-strata sliding. The most critical area where the shear stress reaches the maximum value is located in a horizontal plane approximately 1m above the roof line. The maximum shear resistance required on the horizontal plane was found at some distance from the corners. It decreased towards the centre and the corners of the intersection.

In 1985 - 1986, a purely experimental field investigation was carried out by Hanna, Haramy and Conover to determine the reasons for ground problems at roadway intersections. The objectives of this project were to develop: (i) a method to determine failure modes and stress-displacement relations around roadway intersections, and (ii) engineering guideline's and empirical relations for use in the design of safer intersections.
CHAPTER 2: Theoretical Aspects of Stability of Roadways and Intersections

The basic approach to achieve these goals was to instrument and monitor intersections before, during and after intersection development. It was done to recognise the parameters influencing the short- and long-term stability of the intersection structure and adjacent entries. They also tried to study results obtained from reorienting the mine entry, modifying the bolting pattern and altering the mining sequence and geometry.

A comprehensive instrumentation program was developed to obtain stress and deformation data and to characterise the structural behaviour of four-way intersections. This program was implemented at two underground coal mines using room and pillar mining in the Illinios Coal Basin. Details on the geology of the study area, instrumentation used and the procedure for monitoring have been given in the original reports (Hanna et al, 1985 and 1986b). Results of this research showed that:

(a) Existing horizontal tectonic stresses, mining induced stresses and geological conditions are the main parameters influencing the stability of roadway intersections.

(b) When the orientation of the mine entry was parallel to the horizontal principal stress, numerous roof falls would occur in the direction perpendicular to the maximum horizontal stress and a stable situation would exist in the other direction.

(c) When the orientation of the mine was oblique (45°) to the direction of maximum horizontal stress, then more stable conditions were achieved.

(d) The roof failed by crushing or shearing during the initial box cut. Typical heights of the failed zone ranged from 0.6 m to 1.4 m with a failure angle of 75° from the horizontal line. The longitudinal axes of falls were parallel to the entry (the entry was parallel to \( \sigma_{H_{\text{max}}} \)).

(e) Roof falls assumed a dome-like shape and were usually confined within the intersection. Roof falls had a nominal dimension of about 10 m, resulting in predicted heights of 3.5 m. Most observed falls were higher than 5 m; therefore the equation developed by Peng for the Pittsburgh Coal Seam could not be applied to the mine without modification.

(f) It was expected that the dome failure height increases as the horizontal stress increases.

(g) Typically, one side of the fall cavity was highly fractured and the opposite side was smooth and blocky. This indicated that horizontal stresses were the main cause of rock
shearing in the fractured zone, which gradually progressed into the roof until it intersected a weak or separated plane above the roof bolts (Figure 2.19).

(h) The shearing mechanism appeared to relieve and transfer stress from around the entry to the cantilevered roof layers.

The following measures were suggested by Hanna et al to control the problem.

(i) Longer roof bolts merely resulted in higher falls. To minimise horizontal stress-induced failures, an improved roof support method with anchorage outside the failure envelope was required.

Figure 2.19 Typical shearing failure at roadway intersections (Hanna et al 1986b).
(ii) By using a roof bolting system consisting of 3 m long bolts close to the rib-sides and 1.5 m long bolts at the centre of the roadway, roof falls decreased significantly.

(iii) If roof truss bolts are also integrated into the roof control system, there would appear to be further reduction in the number of roof falls.

(iv) Full pillaring was expected to result in localised relief of horizontal stresses, therefore, improving general mining conditions.

2.5 Conclusions

General conclusions for this Chapter can be summarised as follows:

(a) Theoretical approaches such as beam and plate theories, voussoir beam theory incorporate assumptions relating to the properties of rocks, loading condition and geometry of a structure. The degree of uncertainty inherent in the problem is overcome by employing large factors of safety; therefore, these methods usually lead to over estimated design. However, there are some cases like intersections in a high horizontal stress field with the possibility of sliding effects of strata that are beyond the capability of current theoretical methods.

(b) Application of numerical methods in stability analysis of underground structures has been propagating during the last few decades. It has been proved that 3-D FEM can be successfully employed to investigate the behaviour of roadways and intersections. However, it is necessary to develop new techniques which consider more realistic conditions.

(c) Field investigations and in-situ measurements are the major practices in the "design-as-you-go" approach. Although this method is very expensive and time consuming, its results are very informative and could be used to assess and modify numerical techniques.

Specific conclusions from the literature relating to the stability of roadways and intersections may be summarised as follows:

(a) In-situ stress state has a significant effect on the behaviour of underground structures. When the ratio of horizontal to vertical stress is greater than 1, in particular when greater than 2, the orientation of roadways with respect to the stress field will be a key factor in the stability of the structure.
(b) Mechanical properties of rocks in the strata units will determine the response of the opening to the condition. However, in some circumstances, the property of bedding planes are more dominant and may cause serious unstable conditions. Furthermore, an optimal design of reinforcement system can not be achieved unless the structural behaviour of the bedding planes is taken into account.

(c) Mining method and sequence of operations will affect the stress and displacement patterns around the structures.

(d) Despite many investigations on the stability of roadways and intersections, further detailed investigations which take into account of non-linear behaviour, empirical failure criteria, the properties of bedding planes and different stress states with respect to the Australian underground coal mines are required.
CHAPTER THREE

FUNDAMENTALS OF NUMERICAL MODELLING
3.1 Introduction

Numerical methods represent the most versatile computational method for the various engineering disciplines. If a complex problem is to be solved, the use of numerical methods is most likely necessary. In particular, computer modelling using finite element analysis enables evaluation of complex underground structures in a rapid and cost-effective manner.

The fundamental characteristic of numerical methods is that a large structure is discretized into relatively small elements. Then constitutive equations that describe the individual elements and their interactions are constructed. Finally these equations, which are large in number, will be solved simultaneously and interactively using computers. The results from this procedure include the stress distribution and displacement pattern within the structure. Geotechnical engineers classify the numerical methods into three major models listed in Table 3.1 (Sinha 1989).

Table 3.1 Numerical methods and models for mining engineering (After Sinha, 1989).
3.2 Description of numerical techniques

There is a wide range of numerical methods for analysing the stability of underground structures. Using any of these methods, one must take note of the following four aspects which may cause considerable deviation in results from the realistic situation unless they are properly considered in the modelling technique.

(a) *Numerical specification of the model:* it is essential that the model must realistically represent the in-situ structure. This includes modelling the geometrical configuration, material properties, boundary conditions, loading conditions, support system and interaction of substructures within the main structure. The geometrical configuration and dimensions of the structure dictates whether two- or three-dimensional modelling techniques should be used, while the properties of the material determines if either an elastic or elastic-plastic solution, with the application of an appropriate failure criterion is required.

(b) *Preparation of input data:* the data required to construct the model generally includes the dimensions and shape of the structure, mechanical properties of rocks, state of field stress, reinforcement or support system configuration and sequence of mining operations. These data are mostly obtained from field observations and laboratory testing. It should be noted however, that the relationship between laboratory results and in-situ properties must be determined before the data is used. If all the required data to run the model are not available then some degree of compromise or approximation is necessary.

(c) *Choice of solution method:* each method of solution has its own primary assumptions such as behaviour of materials and loading conditions. Based on these assumptions, each method follows a certain approach and sequential procedure to solve the mathematical equations governing the model. To choose an appropriate solution sequence the first two aspects mentioned above should be considered very carefully.

(d) *Interpretation of the results:* results obtained from modelling are the outcome of undertaking the foregoing aspects of modelling. Therefore, the burden of interpretation and evaluation of the results is on the person who carried out the previous stages of modelling. However, in-situ instrumentation and field observation are reliable tools which can help to assess the reliability of the results and to aid in modifying the modelling technique.
Keeping all the above mentioned points in mind, one must know that models are a simplification of the real situation of underground structures and some approximations are inevitable at different stages of modelling. However, a good knowledge of the fundamentals of numerical methods and a deep understanding of strata mechanics are necessary in using numerical methods for analysing the stability of underground structures.

3.3 Finite element method

In the finite element method (FEM) the structure is modelled as a continuum; however, discontinuities can be modelled individually. The whole structure is divided into finite number of elements that are interconnected at grid points (Figure 3.1). Each element is finite, i.e. geometrically defined and limited in size. The stress-strain relation of the elements' material is described by an appropriate constitutive equation. External loads, excavation shape and motion limitations are applied to the model by introducing boundary conditions and constraint codes for grid points.

![Figure 3.1 Mesh for a plane strain model of a roadway.](image)

In terms of solution method, FEM is an application of the direct stiffness method of structural analysis (Ross 1985). The analysis of the problem is performed by solving the equation matrix that models the mesh set up of elements. These equations relate unknown quantities to known quantities via a global stiffness matrix.
3.3.1 Basic theory of the finite element method

The FEM seeks to analyse a continuum problem in terms of sets of nodal forces and displacements for a discretised domain (Brady and Brown, 1985). The procedure involves a set of routines which generate the stiffness matrix \([K^e]\) and initial load vector \(\{f^e\}\) for all elements. These data and applied external loads, together with boundary conditions are used to determine the nodal displacements for the whole structure. In these computations three major equations are repeatedly used.

(a) The characteristic equation of elements; it is defined by the material property and nodal displacement of grid points surrounding the element.

\[
\{f^e\} = [K^e] \cdot \{u^e_i\}
\]  
(3.1)

where:

- \(\{f^e\}\) = column matrix representing the internal elastic force components induced at grid points
- \([K^e]\) = element stiffness matrix
- \(\{u^e_i\}\) = column matrix representing nodal displacement of the element.

(b) the structural stiffness matrix; it is defined by compiling the element stiffness matrices together.

\[
[K^s] = \sum [K^e]
\]  
(3.2)

Where:

- \([K^s]\) = the structure stiffness matrix.

It should be noted that freedom conditions of individual grid points will affect the construction of the structure stiffness matrix.

(c) the characteristic equation of the structure; it is set up as follow:

\[
\{F^s\} = [K^s] \cdot \{u^e_i\}
\]  
(3.3)
Where:

\[
\{F_s\} = \text{column matrix representing external forces.}
\]

To solve any finite element problem using the displacement method, the following procedure should be carried out precisely:

- construct stiffness matrix for every element
- construct structural stiffness matrix (using Equation 3.2)
- calculate displacement of those grid points which have any displacement (using Equation 3.3)
- calculate forces induced in every element (using Equation 3.1).

It is evident that one of the major problems to be overcome is determination of the element stiffness matrices. This is because: (i) the element stiffness is dependent upon the geometry and material properties of the element, (ii) elements take many and varied forms depending on the shape of the structure, and (iii) the material properties are strain- and time-dependent, particularly for soils and rocks.

During the past few decades several excellent texts have been published that adequately describe the detail of the FEM (Steele 1989, Ross 1985, Zienkiewicz et al 1969). Thus, the theoretical discussion that follows will be limited to the particular linear and non-linear solution methods utilised for the analysis of underground structures.

### 3.3.2 Elastic and elastic-plastic constitutive relationships

As mentioned before, the element stiffness matrix is fundamentally representative for both the shape and material properties of the element. The stiffness matrix explains the relationship between applied stresses and induced strains within the element. In other words, if the strains within the element are known then the stresses can be calculated, or vice versa.

For an elastic stress-stain relationship the generalised Hooke's law for plane stress can be expressed by the following matrix notation;

\[
\sigma = D_e \cdot (\varepsilon - \varepsilon_0)
\]  

(3.4)
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Where:

\[ \mathbf{\sigma} = a \text{ plane stress vector, and it is defined by Equation 3.5}, \]
\[ \mathbf{\sigma} = \{\sigma_x, \sigma_y, \tau_{xy}\} \]  \hspace{1cm} (3.5)

\[ \mathbf{D_e} = \text{the elasticity matrix for plane stress; and it is given by Equation 3.6} \]
\[ \mathbf{D_e} = \frac{E}{1 - \nu} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1 - \nu}{2} \end{bmatrix} \]  \hspace{1cm} (3.6)

\[ \mathbf{\varepsilon_0} = \text{initial strain vector such as thermal strain, shrinkage etc.; and it is defined by} \]
\[ \mathbf{\varepsilon_0} = \{\varepsilon_x, \varepsilon_y, \gamma_{xy}\} \]  \hspace{1cm} (3.7)

\[ \mathbf{\varepsilon} = \text{strain vector; this vector at any point within an element is defined in terms of} \]
\[ \text{displacements by the relationship expressed in Equation 3.8} \]
\[ \mathbf{\varepsilon} = \begin{bmatrix} \varepsilon_x \\ \varepsilon_y \\ \gamma_{xy} \end{bmatrix} = \begin{bmatrix} \frac{\partial u}{\partial x} \\ \frac{\partial v}{\partial y} \\ \frac{\partial u}{\partial y} + \frac{\partial v}{\partial x} \end{bmatrix} \]  \hspace{1cm} (3.8)

If the shape function is substituted into the above Equation, then a general relationship can be written in the matrix notation as given in Equation 3.9;

\[ \mathbf{\varepsilon} = \mathbf{B} \cdot \mathbf{U} \]  \hspace{1cm} (3.9)

Where matrix \( \mathbf{B} \) is independent of the position within the element and is derived from the shape function; thus \( \mathbf{u} = \{u_i; u_j; u_k\} \) would be a global displacement vector shown in Figure 3.2.
According to the principle of virtual displacement (Komatsu and Kitada 1975), the following relationship can be obtained (Equation 3.10).

\[ \mathbf{q} = \mathbf{K}^e \cdot \mathbf{U} \cdot \mathbf{q}_0 \]  

(3.10)

Where:
\[
\begin{align*}
\mathbf{q} &= \text{nodal force vector} \\
\mathbf{U} &= \text{nodal displacement vector} \\
\mathbf{q}_0 &= \text{equivalent nodal force vector due to an initial strain } \mathbf{\varepsilon}_0 \\
\mathbf{K}^e &= \text{stiffness matrix of the element which is denote by Equation 3.11;}
\end{align*}
\]

\[ \mathbf{K}^e = \mathbf{B}^T \cdot \mathbf{D}_e \cdot \mathbf{B} \cdot \mathbf{t} \cdot \mathbf{A} \]  

(3.11)

Where \( \mathbf{t} \) and \( \mathbf{A} \) are thickness and area of the element, respectively.

Finally, with assemblage of Equation 3.10 including all elements, the equilibrium equation of the whole structure can be easily obtained as following:

\[ \mathbf{F}^s = \mathbf{K}^s \cdot \mathbf{U} - \mathbf{F}_0 \]  

(3.12)

Where:
\[
\begin{align*}
\mathbf{F}^s &= \text{listing of all nodal loads} \\
\mathbf{K}^s &= \text{global stiffness matrix of the assembled structure}
\end{align*}
\]
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\[ U = \text{listing of all nodal displacements} \]
\[ F_0 = \text{equivalent nodal forces due to an initial strain} \]

For non-linear materials where a structure is in an elastic-plastic state, the stress-strain increments are generally related by Equation 3.13;

\[ d\sigma = D_{ep} \cdot d\varepsilon \tag{3.13} \]

Where:
\[ d\sigma = \text{incremental stress vector} \]
\[ d\varepsilon = \text{incremental strain vector which is the sum of an incremental elastic-strain vector, } d\varepsilon_e, \text{ and an incremental plastic-strain vector, } d\varepsilon_p, \text{ as follows:} \]
\[ d\varepsilon = d\varepsilon_e + d\varepsilon_p \tag{3.14} \]

and according to Plastic Flow Theory, due to Prandt-Reuss (quoted by Yamada et al 1968), \[ d\varepsilon_p = \lambda \frac{\partial F}{\partial \sigma} \]
where \( \lambda \) is a non-negative scalar defining the magnitude of plastic-strain, and \( \frac{\partial F}{\partial \sigma} \) gives the direction of plastic-strain (\( F \) is an equivalent stress or yield stress confirmed by a standard test); and \( d\varepsilon_e \) is an incremental elastic-strain vector which can be derived from Equation 3.4 in the following form:

\[ d\varepsilon_e = D_{ep}^{-1} d\sigma \tag{3.15} \]

\[ D_{ep} = \text{elastic-plastic matrix; and it is given by Equation 3.16;} \]

\[ D_{ep} = D_e - D_p \tag{3.16} \]

Where \( D_e \) is the elasticity matrix and \( D_p \) is the stress-dependent plastic component, denoted as:

\[ D_p = \left[ \frac{\partial F}{\partial \sigma} \right]^T \cdot D_e \cdot \left[ \frac{\partial F}{\partial \sigma} \right] \tag{3.17} \]

where \( H' \) is the slope of the stress-strain curve in the plastic region.
In the case of axisymmetry and perfect Von Mises plasticity, the stress dependent plastic component is given by:

\[
D_p = \frac{2E}{2\overline{\sigma}^2(1+\nu)} \begin{bmatrix}
S^2_r & S_r S_z & S^2_z \\
S_r S_z & \tau_{rz} S_r & \tau_{rz} S_z \\
S_r S_z & \tau_{rz} S_z & S_\theta^2
\end{bmatrix}
\]

(3.18)

Where \(S_r, S_z\) etc. are the deviatoric stresses \((2\sigma_r - \sigma_\theta - \sigma_z)/3\) and \(\overline{\sigma}\) is the Von-Mises yield stress (For more information, see Yamada 1969; Zienkiewicz et al 1969; Komatsu and Kitada 1975; Ross 1985).

### 3.3.3 Non-linear finite element solution methods

Since 1962 there have been many attempts to give specific solutions for elastic-plastic analysis. A number of these analytical studies have been based on the finite element methods. In this method, material non-linearity can be overcome by adopting an iterative method, an incremental method or an appropriate mixed method. Five of the most well-known of these methods are listed below (quoted by Komatsu and Kitada 1975).

- Tangent Stiffness Method (T.S.M)
- Initial Stress Method (I.S.M)
- Improved Initial Stress Method (I.I.S.M)
- Hybrid Method (H.M)
- Modified Load Incremental Method (M.L.I.M)

Almost, all of the non-linear solution procedures are incremental in character, but selection of the method depends on the nature of the problem. The method used in MSC/NASTRAN will be explained in the next section.

### 3.4 MSC/NASTRAN

MSC/NASTRAN is a general purpose 3-D finite element program which can be used for static and dynamic stress and displacement analysis of structures, solids and fluid
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systems (MSC/NASTRAN Theoretical Manual 1981 and MSC/NASTRAN User's Manual 1991). This program is capable of analysing structural systems using a combination of different finite elements. In addition, a GAP element is also included in NASTRAN for modelling structural separation and frictional effects. This element was used to model the effect of strata sliding over each other.

NASTRAN can be used to perform linear and non-linear analysis. Current solution capabilities for non-linear analyses are limited in their ability and cannot properly simulate the constitutive relationships of rock materials with respect to the post-failure behaviour of rocks. Material property options available in NASTRAN can describe the non-linear portion of the stress-strain curve for the strain-hardening or plastic condition, but not the strain-softening portion; which is required to simulate the post-failure behaviour of rocks.

Since many underground structures such as roadways and intersections are subject to partial yielding of coal or sometimes roof and floor, the analytical solution sequences available in NASTRAN are limited in use for mining engineering. The other limitation with this program is the yielding criteria which are not well adapted to rocks. The available criteria are Von-Mises, Tresca, Mohr-Coulomb and Drucker-Prager.

To overcome these problems, a new procedure has been developed which consists of two programs in FORTRAN 77 to accommodate NASTRAN for simulating the post-failure behaviour of rocks, and considering appropriate failure criteria such as the Hoek and Brown or Bieniawski criterion. The new techniques are described in Chapter 4.

3.4.1 Non-linear solution technique in NASTRAN

Non-linear structural solutions are typically obtained from a trial-and-error search procedure for particular loads or displacements. The search procedure starts from a particular stress state and terminates when the basic equations are satisfied within a known tolerance.

Non-linear solution methods available in NASTRAN provide both geometric and material non-linearity solutions. These nonlinear capabilities for static and transient analysis are provided as self-contained solution sequences SOL66 and SOL99, respectively. In version 67, the structured solution 106 is also available. This solution method may be used to analyse the model while up-grading the stiffness matrix at any iteration loop specified by the load control commands such as RESTART.
The procedure of solution 66 used in this research is simplified in Figure 3.3. The fundamental theory behind this solution is to minimise the error vector \( \delta \), given by Equation 3.19.

\[
\{ \delta \} = \{ P \} + \{ Q \} - \{ F \} \tag{3.19}
\]

Where:

- \( \{ \delta \} \) = error vector of unbalanced forces acting at all grid point components
- \( \{ P \} \) = known vector of applied external loads
- \( \{ Q \} \) = unknown vector of constraint forces due to single and multi-point constraints
- \( \{ F \} \) = vector of grid point forces due to forces generated by element motion and stress. These forces are functions of displacement, temperature and stress history.

Terms in the vector \( \{ F \} \) are dependent on finite elements. In linear static analysis without any thermal load, vector \( \{ F \} \) changes into Equation 3.20;

\[
\{ F_{\text{linear}} \} = [K^l] \cdot \{ u \} \tag{3.20}
\]

Where:

- \( [K^l] \) = linear stiffness matrix
- \( \{ u \} \) = vector for grid point displacements

To obtain a non-linear solution, a "tangent" matrix \([K]\) is used in which the terms are derivatives in the following form:

\[
[K_{ij}] = \left[ \frac{\partial F_i}{\partial u_j} \right]_{u-u^2} \tag{3.21}
\]

Matrix \([K]\) contains both geometric and material non-linearity effects. The approximation occurs because forces are highly variable with displacements, and non-symmetric terms are approximated as well. Details of this solution sequence are covered in MSC/NASTRAN User's Manual 1991.
Figure 3.3 The procedure of non-linear solution in NASTRAN.
Examples of the iterative method used in this technique are shown in Figures 3.4 and 3.5. Curves show a single degree of freedom non-linear force function. The dashed lines show the iteration paths, with slopes equal to the reference matrix. Figure 3.4 illustrates that simple corrective loads are sufficient for low load levels when large stiffness (represented by the tangent slope) changes do not occur. Figure 3.5 shows that matrix updates must be used when larger loads are applied and the non-linear effects become dominant. It should be noted that a divergent search path (Figure 3.5) may possibly occur when inconsistent matrix update procedures are used.

3.4.2 Non-linear materials and yield criteria in NASTRAN

The broad categories of material non-linearity are distinguished as non-linear elastic and plastic materials as shown in Figure 3.6. In both cases, the stress strain relationship is non-linear; however, in the case of nonlinear elasticity, the unloading curve although following a different path from the loading curve, there would be no permanent deformation in the specimen. In the plastic case, the unloading curve again is different from the loading path, but there would be a permanent deformation in the specimen.

In the real situation, the stress strain curve has two major parts called the elastic range and plastic range. The major difference between them is that the strain in the plastic range is not uniquely determined by the stress but is dependent on the past history of stress as well. Due to the path-dependent nature of the plastic strain, the increments of plastic strain are calculated by an incremental procedure and the total strain is obtained by integration or summation.

The stress value at which the elastic range changes to plastic range is called the yield stress. Yield stress is then measured as the value of stress which produces the smallest measurable permanent strain, and it is one of the characteristics of the material. However, when the stress state at a point consists of several stresses in different directions, a criterion is required to determine which combination of multi axial stresses will cause yield.
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Figure 3.4 Single degree of freedom without updating stiffness matrix.

Figure 3.5 Single degree of freedom with updating stiffness matrix.
A number of criteria have been proposed for the yielding of solids. MSC/NASTRAN provides for four of these criteria viz., Von Mises, Tresca, Mohr-Coulomb and Drucker-Prager. The former two are most commonly used in plastic analysis of ductile materials like steel, while the latter two are suitable for analysis of frictional or brittle materials such as concrete, soil and to some extent rock. These criteria are illustrated in Figures 3.7 and 3.8, and summarised in Equations 3.22 to 3.25.

Von Mises: \[ \frac{1}{2} \left[ (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right] = \sigma_y^2 \] (3.22)

Where:
\( \sigma_1 \) = major principal stress
\( \sigma_2 \) = intermediate principal stress
\( \sigma_3 \) = minor principal stress
\( \sigma_y \) = yield stress

Tresca: \[ \frac{\sigma_1 - \sigma_3}{2} \geq \tau_{\text{max}}, \quad \tau_{\text{max}} = \frac{1}{2} \sigma_y \] (3.23)

\( \tau_{\text{max}} \) = the maximum shear stress, equal to half the yield stress value

Mohr-Coulomb: \[ \sigma_1 - \sigma_3 + (\sigma_1 + \sigma_3) \sin \phi = 2C \cos \phi \] (3.24)

Where:
\( C \) = cohesion
\( \phi \) = angle of internal friction
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**Figure 3.7** Von Mises and Drucker-Prager yielding criteria.

**Figure 3.8** Mohr-Coulomb and Tresca yielding criteria.
Drucker-Prager: \[ 3\alpha \sigma_m + (J_2)^{0.5} = k \] (3.25)

\( \sigma_m \) = first invariant of stress, \( \sigma_m = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3) \)

\( J_2 \) = second invariant of the deviatoric stress,

\[ J_2 = \frac{1}{6}[(\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2] \]

\( \alpha, k \) = constant values; \( \alpha = \frac{2\sin \phi}{\sqrt{3} (3 - \sin \phi)} \), \( k = \frac{6C \cos \phi}{\sqrt{3} (3 - \sin \phi)} \)

3.5 Conclusions

It has been shown that the FEM is a very promising method for solving complex problems such as stress and displacement analysis around underground structures. But due to the fact that any FE code has its own limitations and advantages, it is necessary to ensure the suitability of the program before use. For the purpose of this research, MSC/NASTRAN was chosen because of the following advantages: it is three dimensional, it has a GAP element which can be modified to simulate bedding planes, and also provides RESTART command which allows a manual iteration procedure. However, this code has limitations such as the lack of a realistic failure criterion for rock and also it does not consider the post-failure behaviour of rock. In the next Chapter, new techniques which were developed for this research and applied in conjunction with NASTRAN are presented. These newly developed techniques enable the most realistic simulation of roadway and intersection conditions to be achieved.
CHAPTER FOUR

NEW OPTIMISATION TECHNIQUES FOR
FINITE ELEMENT MODELLING OF
ROADWAYS AND INTERSECTIONS
4.1 Introduction

There is always some concern about the accuracy and validity of the results obtained from finite element analysis. The output of any FE analysis is highly influenced by the parameters employed in the various stages of the modelling procedure. Analysis usually produces some results which may or may not be correct/accurate. There is no direct message or index to indicate the appropriateness of the parameters used in the model; therefore, it is very important to calibrate and optimise these parameters by preparing simple models that have theoretical or obvious solutions. The results from the FE analysis are then compared with the results obtained from theoretical solutions or that from field observations. Some of these parameters are modified until a reasonable correlation between FE results and theoretical results is achieved.

Parameters used in a FE modelling procedure can be categorised in three major groups as follows:

(i) Geometrical parameters: dimension of the model, mesh pattern and density of mesh.
(ii) Structural parameters: element types and properties, freedom condition of grid points, boundary condition of the model and loading technique.
(iii) Material parameters: constant values, constitutive equations and failure criterion that explain the behaviour of the materials in different conditions.

This chapter presents selected results from more than 100 models which were developed to help optimise some of the most important parameters which may influence the FE analysis. To carry out this study, the 3-D FE code, NASTRAN was used on the workstation.
4.2 Geometrical parameters

4.2.1 Dimensions of the model

In general, mining extends over an area so large that, for practical reasons, the entire mining area cannot be included in the model. In FE modelling the region excluded from analysis is replaced by appropriate boundary conditions. Determining the appropriate dimensions of a model is a two fold parameter. When the dimensions are increased the results are more accurate, but on the other hand the computer time and cost of modelling will increase significantly; particularly when dealing with 3-D models.

For the purpose of this study, three different model sizes were constructed and analysed for five loading conditions in order to optimise the dimensions of the model for different conditions. Table 4.1 gives the general information of these models.

<table>
<thead>
<tr>
<th>Model size</th>
<th>Number of Grids</th>
<th>Number of Elements</th>
<th>Computer Running Time</th>
<th>Wm, Hm</th>
<th>size* of input file</th>
<th>size* of total output files</th>
</tr>
</thead>
<tbody>
<tr>
<td>Large</td>
<td>2526</td>
<td>2118</td>
<td>00:08:14</td>
<td>Wm = 7.5 Wo, Hm = 10 Ho</td>
<td>480,007</td>
<td>16,043,169</td>
</tr>
<tr>
<td>Medium</td>
<td>2080</td>
<td>1721</td>
<td>00:07:54</td>
<td>Wm = 6 Wo, Hm = 8 Ho</td>
<td>410,062</td>
<td>13,760,788</td>
</tr>
<tr>
<td>Small</td>
<td>1764</td>
<td>1438</td>
<td>00:07:23</td>
<td>Wm = 4.5 Wo, Hm = 6 Ho</td>
<td>344,935</td>
<td>11,662,220</td>
</tr>
</tbody>
</table>

Where:

Wm and Hm = Width and height of the model
Wo and Ho = Width and height of the opening (Wo = 6 m, Ho = 3 m)
E = 15 GPa, V = 0.25 and Gv = 10 MPa

The ratio of horizontal to vertical stress, K, was set equal to 0, 1, 2, 3 and 4

* file size in bytes

The program was set up to obtain the vertical and horizontal stresses at different locations in the floor, pillar and roof as shown in Figure 4.1. Stress results at these locations are given for one series of models (K=2) in Figures 4.2 to 4.4.
Figure 4.1 Different locations around the roadway for analysing the results.

Figure 4.2 Vertical and horizontal stresses in the floor for three different size of models.

Figure 4.3 Vertical and horizontal stresses in the pillar for three different size of models.
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Figure 4.4 Vertical and horizontal stresses in the roof for three different model sizes.

The results showed that there would always be two disturbed zones in the model, one around the opening where the results are very important for the stability analysis of the structure and the other zone around the outer boundary of the model. The dimensions and shape of the opening as well as the stress state have significant effect on the dimensions of the inner disturbed zone while the boundary condition, loading technique and model size determine the dimensions of the outer disturbed zone.

If the model size is too small, the two zones will overlap which in turn reduces the accuracy of the results around the opening. On the other hand, when the model size is too large, there would be a gap (an undisturbed zone) between the two disturbed zones. It is desirable to have some gap, but is not necessary for it to be too large as this will not increase the accuracy of the results. From the computer efficiency viewpoint, a larger model will require more computer memory and a longer running time. Although this problem may not seem significant for 2-D models using an elastic solution, it may cause problems when running 3-D models using non-linear solutions (As described in Chapter 7, for a 15 m cube model about 700 mega bytes of memory would be required to analyse the model under a non-linear solution).

Therefore, the general idea was to choose the optimum dimensions for the model so that the inner and outer disturbed zones do not either overlap or have excessive gaps in between. The results of the research indicated that although the results from the Small Model are about 5 to 8 percent less accurate than that from the two other models, this would not be significant when compared to the huge memory space and very long computer running time needed for the construction and analysis of bigger models. In summary, the Small Model gives equally accurate results in all cases and there is no need to choose bigger models.
4.2.2 Mesh pattern (element type) and mesh density

It is implied in finite element theory that as the number of elements increase to infinity then the displacement and stresses converge to their true values. Also the convergence is generally related to the size of the element and the order of polynomial approximation inside the element. From another point of view, the computer time and required memory space will increase when the number of elements increases. Thus, a compromise between the accuracy of the results and computer time and cost should be made.

A large variety of element types (mesh patterns) have been used to model 2-D and 3-D structures, but the selection of a particular pattern depends on the geometrical and physical characteristics of the structure. In the present research, QUAD4 and SOLID elements were used to model 2-D roadways and 3-D intersections, respectively. These elements are well matched to the configuration of the bedding strata encountered in the structures and allow the post-processing and interpretation of the results to be carried out on specific group of elements with convenience.

The density of mesh is another parameter which has to be optimised in the modelling process. A finer mesh results in a more accurate output but increases the computer cost. In practice, models were divided into zones of varying mesh density. The division of zones into elements was carried out in such a way that the smallest elements (high density zone) were used near the opening where details of the stress and displacement were required, and elements located farther from the opening were larger.

There were two possible ways to build up the structure from zones of varying mesh density. The first technique was to use continuous density change throughout the model without considering any transitional zone (Figure 4.5(a)). The second technique was to consider transitional zone between the high and low density zones (Figure 4.5(b, c)).
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(a) continuous change in mesh density

(b)

(c)

To optimise parameters relating to the mesh density, 15 models were constructed and solved for stress and displacement around a roadway. General information on the three groups of models are summarised in Table 4.2. QUAD4 elements were used in all models but the density of mesh varied in different zones.

Table 4.2 General information of models used to optimise the mesh density.

<table>
<thead>
<tr>
<th>Model Type</th>
<th>Mesh Density in Zone 1</th>
<th>Mesh Density in Zone 2</th>
<th>Mesh Density in Zone 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine</td>
<td>19</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>Normal</td>
<td>9</td>
<td>4</td>
<td>0.25</td>
</tr>
<tr>
<td>Coarse</td>
<td>4</td>
<td>1</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Where:

$W_m = 30 \text{ m and } H_m = 21 \text{ m}$

$W_o = 6 \text{ m and } H_o = 3 \text{ m}$

$E = 15 \text{ GPa and } v = 0.25$

$\sigma_v = 10 \text{ MPa and } K = 1, 2, 3 \text{ and } 4$

Mesh Density = number of elements per equivalent square metre
Examples of the results relating to displacement at various locations in the pillar and roof (Figure 4.6) are compiled in Figures 4.7 and 4.8. It can be concluded that there is no difference between Fine Model and Normal Model, but the results from Coarse Model show a degree of discrepancy.

Figure 4.6 Locations in the model for analysing the results relating to the mesh pattern.

Figure 4.7 Vertical displacement in the pillar for three different model mesh patterns.
A series of models which considered transitional zones were also studied, the outcome of which was as follows:

(i) when the same type of element is used in the transitional zone as in the main zones (Figure 4.5(b)) there is little discrepancies in the results. But if the elements in transitional zone differ from those in the main zones (Figure 4.5(c)), then the results show considerable discrepancies across the transitional zone.

(ii) when the thickness of the strata encountered in the structure are less than 2 m then the application of the transitional zone will not meet the geometrical and physical characteristics of the structure; because the dimension of elements after transitional zone are sometimes bigger than the thickness of the individual stratum. This problem will be more exacerbated when constructing the 3-D models.

The results from this part of the investigation led to the choice of a continuous mesh pattern consisting of three density zones with 9, 4 and 0.25 elements per square metre respectively as the general pattern for modelling roadways and intersections (Figure 4.9).
Figure 4.9 The general pattern for modelling roadways without using any transitional zone.
4.3 Structural parameters

4.3.1 Loading technique

There are two major methods to simulate the stress state in the model. The first method is to consider gravitational load throughout the model. The second method is applying an equivalent force or stress at grid points or on the free faces of the model.

Using gravitational loading technique, as it is not practical to take into account the total cover of the structure in the FE model, it will be inevitable to use an apparent value for either the density of rocks or the gravity force. In either cases the gradient of stress in the model will be equal to \( H/h \) where \( H \) and \( h \) are depth of overburden and thickness of the model, respectively. This ratio is usually more than 10 when modelling underground structures at depths greater than 300 m. Although inertial loading is an accurate method for modelling surface or shallow structures, application of this method for modelling deep structures will not provide a constant uniform stress state around the model (Figure 4.10).

![Figure 4.10 Stress profile around a model using inertial loading.](image_url)

The second method of simulating the in-situ stress state is to apply the same value of in-situ stress on the free faces of the model or to calculate the equivalent forces and apply that at grid points around the model. In both cases, any effect resulting from the high stress gradient along the sides of the model will be eliminated. In this way, a constant uniform stress state will be achieved. The key point in this method is that there are differences between applying the load on the external or on the internal boundary of the model (Figures 4.11(a) and 4.11(b)).
Three series of models were constructed and analysed under four different loading conditions ($\sigma_v = 10$ MPa and the horizontal to vertical stress ratio, $K = 1, 2, 3$ and $4$) to study the differences. The first group were virgin models without any opening and the stress state was applied on the external boundary. The second group were models including the structure, and stresses were applied on the external boundary. The third group of models were the same as the second group but stresses were applied on the internal boundary. These models are called Null, External and Internal Models, respectively. Figure 4.12 shows locations at which results were obtained.
Vertical stress at the mid-height of the pillar and along the centre-line of the roof as well as vertical displacement on the horizontal line in the roof are given in Figures 4.13 to 4.15. Regarding these results, the following conclusions can be made:

(a) The Null Model shows the initial response of the region to the virgin stresses before any structure was created. The stress state remains almost constant, but there is some deformation throughout the model.

(b) The External Model gives the total values of stress and displacement, including the initial response of the region to the virgin stresses (before constructing the opening) plus the disturbance resulting from creating the opening.

(c) The Internal Model gives the relative change of stress and displacement around the structure - from the initial condition (virgin state) to the final condition (disturbed state).

In other words, the initial conditions of stress and displacement are taken as zero state; hence the results show the induced stress and displacement resulting only from the excavation of the opening.

The above conclusions were exactly the same for all four loading conditions, and in summary the results suggest that:

(i) The absolute values of stress around the structure can be obtained from either:
   (a) External Models stress ($\sigma_0$), or
   (b) Internal Model stress + Virgin stress, ($\sigma_i + \sigma_v$).

(ii) The relative displacement around the structure can be obtained from either:
   (a) Internal Model displacement ($D_i$), or
   (b) External Model displacement - Null Model displacement, ($D_0 - D_n$).

Where $\sigma_0$ and $\sigma_i$ = stresses resulting from the External and Internal Models, respectively.  
$D_i$, $D_n$ and $D_0$ = displacements resulting from Internal, Null and External Models, respectively.  
$\sigma_v$ = virgin stress
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Figure 4.13 Vertical stress at mid-height of the pillar.

Figure 4.14 Vertical stress along centre-line in the roof.

Figure 4.15 Vertical displacement on horizontal line in the roof.
These results also indicate that because there are no boundary effect, the model size using the Internal Model could be reduced by 50%. Although on such 2-D problems, this may not appear significant, in 3-D models the size reduction would significantly reduce the amount of required memory and computer running time.

In order to verify the results obtained from the Internal and External loading techniques, it was decided to apply both techniques to the classical problem of a semi-infinite plate having a circular hole inside (Figure 4.16). The results from the internally and externally loaded models were compared with the results calculated using Kirsch Solution (Obert and Duvall 1967). The comparison of the stress against distance from centre of the opening is given in Figure 4.17. If the Kirsch solution is accepted as the true value, it can be seen from Figure 4.18 that the internally loaded model consistently gives more accurate results. This is particularly true for points close to the opening - which would be the critical region in the case of underground openings. These results suggest that it is desirable to use internal loading when modelling underground structures. This applies to both the accuracy of results and computer costs.

Figure 4.16 A semi-infinite plate having a circular hole inside,
(a) External loading and (b) Internal loading models.
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Figure 4.17 The comparison of the stress against distance from centre of the opening, 
S₀: External result; Sₜ: Theoretical result; Sᵢ: Internal result; Sᵥ: Virgin stress.

Figure 4.18 Accuracy of the External and Internal results against theoretical solution, 
E₀: Error of External loading; Eᵢ: Error of Internal loading.

The next point relating to the loading technique is the method of applying horizontal stress on the model. Since each stratum may have a different stiffness, it may seem that horizontal stress should be divided among the strata according to their stiffness. On the other hand, available in-situ measurements have shown a constant uniform distribution of horizontal stress for a limited range of depth. Therefore, two series of models were constructed and analysed to check this matter. In the first series (Uniform Model) a uniform horizontal stress was applied on all strata while in the second series (Stiffness Model) the horizontal stress was divided among the strata according to their stiffness. These two alternatives are structurally shown in Figure 4.19.
Theoretical analysis shows that as long as the two parts in the model are bound together, there will be no difference in the average frictional force, $f$, calculated from either models, (a) or (b). This concept was checked by FE analysis and the results from the FE models were in good agreement with theoretical solutions. Figures 4.21 to 4.23 show shear stress along three different lines in the model (Figure 4.20). Small discrepancies between the results obtained from the Uniform loading and Stiffness controlled models around the boundary of the model is due to the fact that the distribution of frictional force along the bedding planes are not exactly the same. The discrepancies, however, are not important for locations close to the structure.
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**Figure 4.21** Shear stress along REL1 line in the roof.

**Figure 4.22** Shear stress along REL2 line in the roof.

**Figure 4.23** Shear stress along VEL1 line crossing strata vertically.
On the basis of achievements from this investigation, it was proved that loading technique has a great influence on the results of any finite element analysis. The gravitational loading method is the appropriate method for modelling surface or shallow structures where it is possible to model the total cover (overburden), or for situations where the gradient of the stress along the sides of the model are very small. The uniform loading method is suitable for modelling deep structures but it must be noted that if the load is applied on the external boundary of the model, results are absolute magnitudes of stress and displacement. To find realistic displacements around the structure (relative values), the initial response of the model to the virgin stresses must be taken from the absolute values. On the other hand, if the load is applied on the internal boundary of the model, results are relative values of stress and displacement, and to find the actual value of stress, the virgin stress must be added to the relative values. The Internal Loading Technique, however, has advantages such as; the results are more accurate, it is easy to add virgin stress to model results to get the absolute magnitude of the stress and it is possible to reduce the model size because there is no disturbance around the outer boundary of the model. There is no significant difference between applying the horizontal stress uniformly or distributing it according to stiffness.

4.3.2 Bedding planes

Bedding planes may allow strata units to separate from or slide over each other. This condition will significantly affect the load transmission between the strata units. Therefore, the behaviour of a structure may be governed more by the bedding planes than the individual strata units due to the effect of bedding planes. The ability to handle bedding planes is considered to be one of the most important features of any FE program. This part of the study attempts to show the effect of bedding planes on the stability of underground structures by using a special element, the GAP element, for modelling bedding planes.

In general, as an underground structure is created, the stress state is redistributed and slip or separation may occur if the frictional resistance and/or the tensile strength of the bedding plane is overcome. Once the bedding plane starts to slip or open, the build up of horizontal and vertical stress around its location will decrease, thereby reducing the shear stress on the adjacent strata and changing the stress pattern in that region. The strata and the bedding plane itself will respond to these changes; for example the vertical abutment stress will be transferred farther into the rib reducing the normal stress over the bedding plane and in consequence causing the bedding plane to slip more. Also the confining pressure on the pillar will be decreased reducing coal pillar strength. These progressive
changes may eventually result in serious instability. Babcock and Bickel (1984) suggested that a mechanism such as this can create a coal burst. Factors such as the property of the bedding plane, property of the strata, stress state and geometrical configuration of the structure influence this mechanism.

In this research, two different series of models, Fixed and Gap Models, were constructed and analysed to study the significance of bedding plane properties on the behaviour of underground structures.

**Fixed Model:** In this model adjacent strata units are connected to each other, and no transitional element is considered. This model suggests that the two strata are tied together and act as one. After analysing the model for stress and displacement, the induced stresses on the interface, including shear and normal stresses are obtained. Using these stresses and the properties of the bedding plane a safety factor against shear (along the bedding plane) can be determined using Equation 4.1

\[ S.F = \frac{\mu \cdot N}{\tau} \]  

(4.1)

Where:

- \( S.F \) = shear stress safety factor
- \( N \) = normal stress on the bedding plane (MPa)
- \( \mu \) = friction coefficient of the bedding plane
- \( \tau \) = shear stress on the bedding plane (MPa)

**Gap Model:** In this model the adjacent strata units are connected to each other using special transitional elements called GAP elements. These elements must accurately represent the properties of the bedding planes; axial stiffness before and after closure and frictional properties in case of sliding. Figures 4.24 and 4.25 illustrate the structural characteristics and load-displacement curves of the GAP element employed for modelling the bedding planes.
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Figure 4.24 Structural characteristics of transitional elements called GAP Elements.

\[ \mathbf{k}_{xa} \] stiffness in X direction after closure
\[ \mathbf{k}_{xb} \] stiffness in X direction before closure
\[ \mathbf{k}_t \] shear stiffness in Y-Z plane
\[ \mu_y \] friction coefficient in Y direction
\[ \mu_z \] friction coefficient in Z direction

\( U, V \) and \( W \) are displacements in the X, Y and Z directions, respectively

Figure 4.25 Load-displacement characteristic of GAP Elements.

\( F_0 \) initial load on the gap element
\( U_0 \) initial opening of the gap element

Where \( F_f = \begin{cases} \mu F_x \text{ if } F_x > 0 \\ 0 \text{ if } F_x < 0 \end{cases} \)

\( F_f \): frictional resistance of the gap element
\( F_x \): normal load over the gap element
\( \mu \): friction coefficient of the gap element
Two series of models, Fixed and Gap, were analysed under four different loading conditions ($\sigma_v = 10$ MPa and the ratio of the horizontal to vertical stress, $K = 1, 2, 3$ and 4) in order to compare the response of both model types to various stress regions. Figure 4.26 shows the location of the gap element modelled bedding plane and the points from where the results were obtained. The mechanical properties of the strata units and the bedding plane encountered in these models are summarised in Tables 4.3(a) and 4.3(b), respectively. Two different sets of mechanical properties, PID100 and PID200, were used for gap elements in order to simulate the active and inactive bedding plane, respectively.

Figure 4.26 Location of the gap elements modelled bedding plane and the points from where the results were obtained.
Since a GAP element is considered to have non-linear behaviour, a non-linear method of solution was used to analyse the models. The total load was applied to the model over 5 increments (subcases) to simulate the non-linear load path. Each subcase was performed with 5 iterations to allow the solution to reach convergence.

Table 4.3(a) Mechanical properties of the strata units in the Fixed and Gap Models.

<table>
<thead>
<tr>
<th>Number</th>
<th>Elastic Modulus E (GPa)</th>
<th>Poisson's Ratio V</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>20</td>
<td>0.20</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>0.22</td>
</tr>
<tr>
<td>3</td>
<td>10</td>
<td>0.25</td>
</tr>
<tr>
<td>4</td>
<td>3.5</td>
<td>0.30</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>0.22</td>
</tr>
<tr>
<td>6</td>
<td>15</td>
<td>0.20</td>
</tr>
<tr>
<td>7</td>
<td>20</td>
<td>0.22</td>
</tr>
<tr>
<td>8</td>
<td>25</td>
<td>0.20</td>
</tr>
</tbody>
</table>

* Property Index.

Table 4.3(b) Mechanical properties of gap elements used in Gap Models.

<table>
<thead>
<tr>
<th>PID #</th>
<th>u₀</th>
<th>F₀</th>
<th>Kₐ</th>
<th>Kₜ</th>
<th>μᵧ = μZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>5.0</td>
<td>0</td>
<td>1.1+E12</td>
<td>110</td>
<td>9.0+E8</td>
</tr>
<tr>
<td>200</td>
<td>5.0</td>
<td>0</td>
<td>1.1+E12</td>
<td>1.0+E10</td>
<td>9.0+E8</td>
</tr>
</tbody>
</table>

u₀ = initial thickness of the gap element (mm)
F₀ = initial load on the gap element (N)
Kₐ = axial stiffness after closure (Pa / m)
Kₜ = axial stiffness before closure (Pa / m)
Kₜ = shear stiffness when gap element is closed, can be μᵧ<Kₐ, (Pa / m)
μᵧ, μZ = coefficient of friction in the Y and Z directions.

The results from the study (for K = 2) are presented in Figures 4.27 to 4.29 (note the variation of scales). These figures illustrate, in a comparative form, the variation of the results obtained from the fixed and gap models.
Figure 4.27 Vertical stress on the mid-height of the pillar (K=2).

Figure 4.28 Shear stress below and on the bedding plane (K=2).

Figure 4.29 Vertical displacement along centre line in the roof (K=2).
When the conditions are such that the bedding plane remains closed the results suggest that the vertical stress at the mid-height of the pillar, indicated in Figure 4.27(a), do not vary significantly between the fixed and gap models; in actual fact showing a fairly constant 1 MPa difference for any given point. This is as expected, as the gap remaining closed would allow the rock to act as a fairly cohesive unit. Conversely, when the conditions are such that the gap opens, Figure 4.27(b), there is a significant variation in the results; particularly at the abutment (note the variation in scale). The fixed model produces the same results (as expected), however, the gap model shows a significant increase in the peak abutment stress and depending on the horizontal stress, reaches a value between 2 and 3 times the virgin vertical stress. In this situation, as the gap element allows separation across the bedding plane, the lower roof layer acts as a beam held at the abutments. This situation is closer to reality, as literature suggests that the peak abutment load is between 3 and 5 times the cover load (Wilson, 1972) and occurs some distance into the rib (the coal at the edge of rib fails, pushing the stress concentration further into the rib where it can be accommodated due to the triaxial conditions).

Similar to the above analysis, Figure 4.28(a) and 4.28(b) show the shear stress in the elements directly below the bedding plane for both the fixed and gap models, with and without separation. When no separation occurs, the shear stress in the elements of both the fixed and gap models are similar, Figure 4.28(a), with the peak shear stress occurring at the edge of the ribs. Now comparing the results when the gap element is allowed to open shows quite a significant variation, Figure 4.28(b). On the bedding plane in the gap model, the shear stress is zero as expected (this was a check that the model was performing to expectations). The fixed model acts as before, but the gap model shows a considerable decrease in the shear stress at the rib edge. It was also noted that any increase in the horizontal stress field did not affect the results significantly. It should be mentioned that no gap elements were used at locations close to the outside boundary of the model where, in reality, sliding or separation would not occur.

Figures 4.29(a) and 4.29(b) show that although separation does not occur, there was still some movement of the roof strata towards the opening. Also, the gap model shows closure of the gap. When separation of the gap element occurred, Figure 4.29(b), the lower roof beam deflected significantly, but the roof which essentially acts as a fixed model had similar displacement to that of the initial model. This comparison would be important when determining the effect that roof bolting has on controlling strata movement.
Due to this investigation, it can be concluded that an interface or bedding plane between the coal and surrounding strata, and/or between the strata units around an underground opening usually represents a sudden change in the mechanical and structural properties of the media. The bedding planes have their own material properties, such as axial and transverse stiffness, cohesion and frictional coefficient. These properties usually vary when the bedding plane is structurally active. When a joint is open and/or the normal stress is low to moderate, its stiffness, tension and frictional coefficient are less than those of the adjacent strata. But when the bedding plane is closed and/or the normal stress is high, the mechanical properties of the bedding plane are similar to the surrounding strata.

In the situation where the bedding plane is not active, no sliding or separation occurs, there would not be significant difference between the results from the Fixed and Gap models. But when the bedding plane is active which means that the situation is such that sliding and/or separation occurs on the bedding plane, then the results of the Gap model will be totally different from the Fixed model.

This study showed that the stress distribution and displacement pattern around the structure are influenced by the behaviour of the bedding planes. Thus, it is necessary to use the Gap Model in order to simulate more realistically the behaviour of the structure including any possible separation and/or sliding effect of the strata.

### 4.3.3 Support systems; point anchor bolts and fully grouted bolts

The aim of any bolting system is either to withstand the shear stress induced on the bedding planes, to suspend the lower strata to the upper ones, or both. In any of these cases, the idea is to tie the individual layers to build up a thick unit which acts as a safe roof over the opening (details on roof bolting mechanisms were comprehensively discussed in Chapter 2).

To evaluate the effect of bolting on the behaviour of the structure, a further two series of models were constructed and analysed. In the first series, fully grouted bolts and in the second series, point anchor bolts (pretensioned bolts) were taken into account (Table 4.4). The roof bolt pattern and their specifications are illustrated and summarised in Figure 4.30 and Table 4.5, respectively. The results for the loading condition $K = 2$ are given in Figures 4.31 and 4.32.
CHAPTER 4: New Optimisation Techniques for FE Modelling of Roadways and Intersections

3.0
2.0
1.0
0.0

Figure 4.30 Roof bolt pattern proposed in the Fixed and Gap Models.

Table 4.4 Various models to study bolting effects on the behaviour of the structure.

<table>
<thead>
<tr>
<th>Bedding Model</th>
<th>Bolting Model</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fixed Model</td>
<td>No Bolts</td>
<td>FNB</td>
</tr>
<tr>
<td></td>
<td>Point Anchored Bolt</td>
<td>FTB</td>
</tr>
<tr>
<td></td>
<td>Grouted Bolt</td>
<td>FGB</td>
</tr>
<tr>
<td>Gap Model</td>
<td>No Bolts</td>
<td>FNB</td>
</tr>
<tr>
<td></td>
<td>Point Anchored Bolt</td>
<td>FTB</td>
</tr>
<tr>
<td></td>
<td>Grouted Bolt</td>
<td>FGB</td>
</tr>
</tbody>
</table>

Table 4.5 Specification of the bolts considered in the Fixed and Gap Models.

<table>
<thead>
<tr>
<th>Type of Bolt</th>
<th>Length (m)</th>
<th>Diameter (mm)</th>
<th>Prestress (MPa)</th>
<th>E (GPa)</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grouted</td>
<td>2.0</td>
<td>22</td>
<td>00.0</td>
<td>200</td>
<td>0.3</td>
</tr>
<tr>
<td>Point Anchored</td>
<td>2.0</td>
<td>22</td>
<td>32.1</td>
<td>200</td>
<td>0.3</td>
</tr>
</tbody>
</table>
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Figure 4.31 Shear stress on the bedding plane in models simulating different roof bolts, (Fixed model, $K = 2$).

Figure 4.32 Shear stress on the bedding plane in models simulating different roof bolts, (Gap model, $K = 2$).
The combination of bedding plane and bolting system simulation led to the following conclusions:

(a) the effect of any bolting system is insignificant when using Fixed Models. This is because in Fixed Models the strata are tied together and the use of bolts will not significantly change the stiffness of the system.
(b) the effect of the bolting system is appreciable in Gap Models where the strata in the model are allowed to separate and/or slide, if the situation is so. This effect is more significant when separation or sliding is more pronounced.

4.4 Material properties

Many combinations exist for the analysis of various material properties and related factors which may have significant effect on the stability analysis of underground structures. In general the property of rocks encountered in finite element analyses can be categorised into three groups as follows:

(a) Linear-elastic: A linear-elastic condition implies that deformation or strain changes within the material are proportional to their corresponding load or stress changes. The ratio of applied stress to the observed strain is termed Elastic modulus, $E$. This parameter remains constant during the analysis.

(b) Non-linear elastic: Non-linearity is incorporated into the analysis by examining the elastic modulus against the level of stress in incremental stages of stress. Material properties are changed against the stress condition for each element. The elastic modulus of any element which passes a certain level of stress will be changed according to its stress-strain characteristic curve. The number of cycles for non-linear analysis is usually between 4 and 10. However, there is no limit to this number to get more accurate results, but it will increase the computer running time significantly.

(c) Post-failure softening: Post-failure behaviour is mostly observed within rocks. In this case, the stress-strain curve has two distinct parts; pre-failure and post-failure. The slope of the first part is always positive and shows a progressive decrease in its value as it gets close to the peak strength. This trend eventually causes the stress-strain relationship to pass through a peak and decrease to a residual value; the slope of the second part is negative until it reaches the residual value where $E = 0$. 
4.4.1 Description of the post-failure modelling procedure

To consider the post-failure behaviour of rocks, a relationship between post-failure strength and post-failure stiffness is established and employed in an iterative procedure. During the solution phase material properties such as the stress-strain curve, elastic modulus, Poisson's ratio and internal angle of friction are updated after each iteration if the degree of failure of individual elements and the extent of the failure zone has been changed. Analysis is terminated when a steady state condition is reached.

To carry out this technique, three programs are executed in turn. The 3-D finite element program, NASTRAN, executes the model with specific material properties at each iteration under increasing load increments. The program SFCONT receives the computed stress values of elements from NASTRAN and calculates the safety factor of every element and prepares a database file which stores all the information about each element. The database file is sent to the program SFDRAW, a graphical post-processor for SFCONT, which prepares various plots of the model showing safety factors for the elements and the extent of the yield zone around the structure. Both SFCONT and SFDRAW were developed as part of this research program.

If the safety factor is less than 1.0 for any of the elements, then material property number, PID#, for that element is changed for succeeding finite element runs to a property number which is representative of the deteriorated property. There are 3 to 5 material property numbers available for every type of rock; representing deterioration of the material as shown in Figure 4.33. If the final PID# is used, and the yield zone still changes or the safety factors for some of the elements are less than 1.0, then there would be no further change to the property of those failed elements and the next run would be carried out at a higher level of loading, if required.
The technique for performing post-failure analysis is illustrated in Figure 4.34, and summarised as follows:

(a) prepare the original model (using MSC/XL)
(b) execute the model by applying incremental loads (using MSC/NASTRAN)
(c) calculate the safety factors of elements based on the stresses computed in the previous stage and the failure criteria specified in the SFCONT program
(d) identify failed zones (using SFDRAW)
(e) determine the degree and extent of the failed zone (using SFDRAW); if any significant change exists, then revise the property of the failed elements and return to step (b)
(f) test for ultimate load; if the ultimate load has not been reached, then compute the load increment, add it to the previous load and return to step (b)
(g) prepare the final output.

It must be mentioned that a built in 'restart' option is available in NASTRAN, version 67, which allows the user to re-run a previously executed model from any given load. Figure 4.35 illustrates the initial and succeeding runs for load-displacement of the structure. This procedure ensures that the stress-history in the rock material has been saved in successive runs, and changes in the material properties are considered over the appropriate loading range, and not from the initial load.
Figure 4.34 Post-failure modelling procedure.
4.4.2 Description of failure criteria used in the post-failure analysis

Stability analysis of an underground structure depends mostly upon the relationship between the induced stress and the strength of the rocks surrounding the structure. Thus, in order to assess the stability of such structures, it is necessary to have a knowledge of the stress induced around the opening as well as to have an appropriate criterion or a set of rules which can predict the response of the surrounding rocks to the induced stress.

In SFCONT, three major failure modes, compressive, tensile and shear have been considered. In the case of compressive failure mode, any of the three criteria: Mohr-Coulomb, Bieniawski or Hoek and Brown may be used. To apply these criteria, the following parameter groups are required as the input data:

(i) Induced stress: Principal stresses induced in each of the elements.
(ii) Strength indexes: Internal angle of friction ($\phi$), cohesion (C), uniaxial compressive strength ($C_0$), shear strength ($\tau$), and tensile strength ($T$).

The first group of data is the output file from running NASTRAN, and the second group of data is given as an input file to SFCONT.
INDUCED STRESSES

After the original model is properly checked and verified, the model is executed to determine the stresses and displacements throughout the structure. The stress values for any element can be determined for grid points and/or the centre of elements. The principal stress values at the centre of elements can also be found.

On the basis of available evidence, Hoek and Brown (1980) believe that it is possible to ignore the influence of the intermediate principal stress upon the failure of rocks encountered in the stability analysis of underground structures. This assumption is important to make the failure criteria as simple as possible, and makes it easier to express these criteria from the data obtained from laboratory tests and from other researchers work. Consequently, the failure criteria used in this study are defined in terms of the major and minor principal stresses. These stresses are then checked against a failure criterion which is explained in the form of a particular equation to determine if localised failure occurs.

FAILURE CRITERIA

The Mohr-Coulomb is the simplest and most frequently used failure criterion. This criterion can be interpreted so that failure occurs when the induced shear stress on the failure plane is equal to or greater than the cohesion of the material and the frictional resistance of the rock on that plane. The Mohr-Coulomb criterion can also be expressed in terms of the major and minor principal stresses as given by Equation 4.2.

\[ \sigma_1 = C_0 + N_\phi \times \sigma_3 \]  

Where

\( \sigma_1 \) = major principal stress  
\( \sigma_3 \) = minor principal stress  
\( C_0 \) = uniaxial compressive strength  
\( N_\phi \) = friction number and may be calculated from \( N_\phi = (1 + \sin(\phi)) / (1 - \sin(\phi)) \)  

where \( \phi \) is internal angle of friction

The use of Equation 4.2 is limited to the condition where \( \sigma_3 > T_0 \) (\( T_0 \) is tensile strength), otherwise, it is assumed that tensile failure has occurred, regardless of the value of \( \sigma_1 \).
The safety factor, $SF$, is calculated by SFCONT based on Mohr-Coulomb criterion as follows:

If $\sigma_3 > T_0$ Then $SF = \frac{C_0 + N\phi \times \sigma_3}{\sigma_1}$ \hspace{1cm} (4.3)

If $\sigma_3 < T_0$ Then $SF = 0$

Bieniawski 1974, proposed an empirical formula to express the failure criterion as follows:

$$\sigma_m = 1 + n.(\sigma_{3n})^m$$ \hspace{1cm} (4.4)

Where:

$\sigma_{1n}$ and $\sigma_{3n}$ = normalised major and minor stresses ($\sigma_{1n} = \frac{\sigma_1}{C_0}$ and $\sigma_{3n} = \frac{\sigma_3}{C_0}$)

$\sigma_1$ and $\sigma_3$ = the major and minor principal stresses, respectively

$C_0$ = uniaxial compressive strength

$n$ and $m$ = constants value determined by fitting a straight line to a group of points of log($\sigma_{1n}$) and log($\sigma_{3n}$) obtained from laboratory triaxial testing on rock specimens.

The safety factor, $SF$, is calculated by SFCONT based on Bieniawski's criterion as follows:

If $\sigma_3 > 0.0$ Then $SF = \frac{1 + n.(\sigma_{3n})^m}{\sigma_{1n}}$ \hspace{1cm} (4.5)

If $\sigma_3 < 0.0$ Then $SF = 0$

Hoek and Brown 1980, defined a relationship between the major and minor principal stresses at failure for brittle rocks containing discontinuities, and expressed it as follows:

$$\sigma_1 = \sigma_3 + \sqrt{m.C_0.\sigma_3 + s\times C_0^2}$$ \hspace{1cm} (4.6)

or $$\sigma_{1n} = \sigma_{3n} + \sqrt{m.\sigma_{3n} + s}$$ \hspace{1cm} (4.7)

Where:

$\sigma_1$ and $\sigma_3$ = the major and minor principal stresses, respectively

$C_0$ = uniaxial compressive strength

$m$ = rock property factor

$s$ = degree of fracturisation of the sample
The safety factor, $SF$, is calculated by SFCONT based on the Hoek and Brown criterion as follows:

If $\sigma_3 > \frac{1}{2} C_0 \cdot (m - \sqrt{m^2 + 4s})$ then $SF = \frac{\sigma_3 + \sqrt{m \cdot \sigma_3 + s}}{\sigma_{ln}}$ \hspace{1cm} (4.8)

If $\sigma_3 \leq \frac{1}{2} C_0 \cdot (m - \sqrt{m^2 + 4s})$ then $SF = 0$

From the above, the safety factor calculated for the individual elements, based on any failure criterion, indicates one of the following:

(i) $SF > 1$ element in compression, but not yet failed,
(ii) $SF \leq 1$ element has failed in compression or shear,
(iii) $SF = 0$ element has failed due to tension

Using the procedure described above, a model was developed and tested to determine the ability of the various procedures to cope with analysis of the post-failure behaviour of rocks. The model was a 2-D simulation of a roadway and was used to make a comparison of the results obtained from a normal analysis and a post-failure analysis.

The model consisted of a 12 m sandstone roof, a 3 m coal pillar and a 12 m mudstone floor. Information on the mechanical properties of the rocks are summarised in Table 4.6. The stress-strain relationship for coal is taken as non-linear, and the characteristic curve used in the analysis is depicted in Figure 4.36.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>$E$ * (GPa)</th>
<th>$V$</th>
<th>$T_0$ (MPa)</th>
<th>$C$ (MPa)</th>
<th>$\phi$ (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>roof</td>
<td>15</td>
<td>0.22</td>
<td>10</td>
<td>16</td>
<td>55</td>
</tr>
<tr>
<td>coal (peak)</td>
<td>3</td>
<td>0.30</td>
<td>1</td>
<td>1.5</td>
<td>30</td>
</tr>
<tr>
<td>coal (residual)</td>
<td>3</td>
<td>0.30</td>
<td>0.5</td>
<td>0.5</td>
<td>25</td>
</tr>
<tr>
<td>floor</td>
<td>10</td>
<td>0.25</td>
<td>10</td>
<td>12</td>
<td>45</td>
</tr>
</tbody>
</table>

* to yield point
The geometry of the structure as well as the mesh pattern surrounding it are illustrated in Figure 4.37. The model consisted of 1340 grid points and included 1134 QUAD4 elements. The stress state employed in the model was $\sigma_v = 10$ MPa and $\sigma_h = 20$ MPa. In the first run of the model the total load was applied over 5 load increments (subcases) to simulate the load path. Each subcase was performed with 5 iterations to allow the solution to reach convergence.

The results, principal stresses at the centre of the elements, were obtained at the end of each load increment and sent to the SFCONT program. This program read the failure parameters of the three rock types from a database and processed the results to determine the safety factor of each element. After determination of failed elements and location of the yield zone around the roadway, the properties of the failed elements were revised to simulate changes in their behaviour. Succeeding runs were then performed to analyse the model under the new conditions. This procedure was repeated 6 times after which the model reached a steady state.

The results from the initial run and succeeding runs are shown in Figures 4.37(a) and 4.37(b), respectively. These figures illustrate the difference in extent of the failed zone around the structure where post-failure behaviour is considered and when it is not considered. It indicates that post-failure behaviour has a significant effect on the stability of underground structures. Comparison of the results proved the credibility of the new procedure for modelling the post-failure behaviour of rocks.
4.5 Conclusions

The FE method has specific features and its fundamental concepts are based on certain assumptions. Using this technique in various fields of engineering, one should be aware of these assumptions which sometimes are referred to as limitations of the method. In the present Chapter, a number of new techniques were developed and implemented in the FE code MSC/NASTRAN to overcome some of the major limitations of FEM in analysing the stability of underground structures; particularly roadways and intersections. The most important cases are summarised as follows:

(i) The FE method is conceptually applicable into the continuum media; therefore, it is basically assumed that all parts (elements) of the structure are bound together as a unit. This assumption affects the stress and displacement patterns throughout the model. The FE method does not cater for separation in the form of either failure or large movements in the structure. Implementing GAP elements wherever separation or sliding is predicted can be a solution to this problem. It should be mentioned that a degree of experience with FE modelling and a good understanding of the physical features of the structure are
required to use this technique properly. For complex situations, it is suggested to use the fixed model and analyse the model for the shear and normal stresses on the bedding plane; then calculate the shear safety factor on the bedding plane and design the bolting system accordingly. This approach is discussed in Chapter 6.

(ii) Generally, joints and fractures are inherent in the strata units surrounding underground structures, and they have significant effect on the behaviour of the structure. Although it is possible to model all these features in the FE model by using GAP elements, there are two major difficulties in implementing this. Firstly, it is very time consuming and costly to map the 3-D geometry and measure the mechanical properties of all discontinuities around the structure. Secondly, to construct a FE model with too many GAP elements is very complex and cumbersome. From the practical point of view, it is suggested to simplify the model into several continuous layers with bedding planes between them and include the joints' effects into the property of rock mass units.

(iii) It is widely accepted that the stress-strain curve of any rock is influenced by the confining pressure applied to it. On the other hand, most FE codes are not able to consider different characteristic curves for a given material during the analysis phase. It is clear that if the elastic modulus of the rock remains constant and only the peak strength is changed due to the variation of confining pressure, then the results of FEA will not be affected by this limitation. For cases where the stress-strain curve changes because of variations in confining pressure, it is possible to use the same procedure as that developed to consider the post-failure behaviour of rocks. This technique uses an iterative procedure that during the solution phase, the stress-strain curve is revised based on the confining pressure induced on the element.

Although the new techniques which have been developed during this research can solve a number of FE limitations, there are still a few more problems that need more research. Time effect and abutment loading resulting from the excavation of adjacent faces over the gate entries are examples of such problems.
CHAPTER FIVE

SITE INVESTIGATION AND DATA ACQUISITION FOR NUMERICAL ANALYSIS
CHAPTER 5

SITE INVESTIGATION AND DATA ACQUISITION FOR NUMERICAL ANALYSIS

5.1 Introduction

Stability analysis of underground structures requires comprehensive data on the mechanical properties of rocks and the geological and geometrical properties of the strata surrounding the opening. In the finite element method, the core of analysis is based on the calculation of material stiffness. Determination of stiffness involves an evaluation of elastic modulus and Poisson's ratio for elastic materials, and equivalent material parameters defining elastic-plastic characteristics of inelastic materials as discussed in Chapter 3.

Material strength is equally important in regards to stability analysis of underground structures. Uniaxial and triaxial compressive strength, tensile strength, shear strength, angle of internal friction, cohesion and other material properties are essential to clarify the failure mechanism of underground structures.

Sedimentary rocks, particularly those in coal measure units, have a wide range of mechanical properties depending on the local conditions. As such, it is important to carry out laboratory testing on samples which are collected from the same site as the field work is performed. For the purpose of this research, Ellalong Colliery was chosen as a site of investigation. And samples were collected from this mine. In this Chapter, the procedure for and results from laboratory testing and field investigations are presented. Related FE modelling based on the data acquired from the site investigation and laboratory testing are given in Chapters 6 and 7.

5.2 Site investigation

5.2.1 General information about Ellalong Colliery

Ellalong Colliery is located in the Newcastle Coalfield, 60 km west of Newcastle and 12 km south of Cessnock (Figure 5.1). It is operated by Newcastle Wallsend Coal Company Pty Ltd. and the major shareholder is Oak Bridge Ltd. The recoverable reserves are about 50 million tonnes. The production capacity in 1991-1992 was 2.2 million tonne,
The principal markets to the company are coking coal to Europe, boarded at Port of Newcastle. Underground machinery consists of one Longwall Unit with Dowty 4 leg Chock Shields, five Continuous Miners, one Anderson Strathclyde AM500 DERD Shearer and nine Shuttle cars.

![Figure 5.1 Location of Ellalong Colliery.](image)

### 5.2.2 Mine layout and geometry of roadways

Ellalong Colliery mines the Greta Seam using the retreat longwall technique at a depth of more than 500 m. Access to the mine is obtained through two vertical shafts, namely upcast No.1 and downcast No.2. All development and access-roadways are rectangular in shape, driven within the coal seam. The working area of the seam is divided into two large districts by 2 East development. Extraction of the north district was completed in 1992 and mining of the south district started in early 1993. Figure 5.2 presents a general view of the mine layout.

The thickness of the seam varies between 3.2 m and 4 m (an average of 3.4 m), but the height of roadways is about 3.0 m. The remainder of the seam is left as top coal in the immediate roof in some parts of the mine. The width of roadways vary from 4.5 m to 5.0 m.
Figure 5.2 General plan view of Ellalong Colliery.
5.2.3 Stratigraphic column

Many surface and underground core drill holes have established the broad pattern of roof and floor lithology profiles across the mining area at Ellalong. The general and latest profiles (at the particular location of this investigation) are given in Figure 5.3. Accordingly, mechanical properties of standard rock samples were comprehensively studied during the course of the present research, and results are presented in Section 5.3.

![Stratigraphic columns for Ellalong Colliery](image)

(a) general, (b) specific for the location of this investigation.

5.2.4 Structural features

There are major faults which transgress the region in a NNW-SSE direction. One of them passes through the north district and had some minor influence on the longwall panels in this part of the mine. During underground observations at roof fall sites and discussion with underground personnel, it was noted that bedding planes resulting in bed separation and/or sliding of the roof strata were related to the changes in lithology, and...
was varied from site to site. At roof fall sites in 1 East and 2 East developments, bedding lamination was more apparent, but still had not caused any appreciable separation or sliding between roof layers. Two distinct fracture types were observed in the roof holes as partings parallel to laminations and mining induced shear fractures (Shepherd et al 1987). The partings were sub-horizontal and were usually dilated. Mining induced shear fractures, so called low angle shear failure, were distinctive in that they generally occurred at angles between $20^\circ$ and $45^\circ$ to the horizontal across beds.

### 5.2.5 Support system

A brief description of the primary and secondary support arrangements in the main entries, gate entries and cut-throughs is presented below (Fuller et al 1987);

- **primary support** consists of 5 or $6 \times 2.1$ m steel bolts, 24 mm diameter, fully encapsulated, placed through steel W straps at 1.0 m centres (Figure 5.4),

![Figure 5.4 Primary roadway support system at Ellalong Colliery.](image)

- **secondary support** consists of:
  1. more steel bolts placed through longitudinally oriented W straps with approximately the same pattern as the primary supports,
  2. cable bolts about 8.0 m long, 25 mm diameter installed in staggered rows of 3 and 4 cables per row, with each row spaced 1.0 m along the roadway.
The combination of primary and secondary support techniques varies throughout the mine due to variations in conditions caused by local geological features.

5.3 Laboratory testing

Information on the mechanical properties of rocks was obtained from:

(a) laboratory tests conducted during this research.
(b) tests results from previous geotechnical investigations undertaken at Ellalong Colliery, generalised and compiled in Table 5.1.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>E (GPa)</th>
<th>ν</th>
<th>UTS (MPa)</th>
<th>C (MPa)</th>
<th>ϕ (degree)</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandstone/f.g.</td>
<td>17</td>
<td>0.20</td>
<td>10</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>sandstone/c.g.</td>
<td>22</td>
<td>0.20</td>
<td>8</td>
<td>17</td>
<td>40</td>
</tr>
<tr>
<td>shale</td>
<td>27</td>
<td>0.20</td>
<td>5</td>
<td>10</td>
<td>27</td>
</tr>
<tr>
<td>mudstone, siltstone</td>
<td>10</td>
<td>0.24</td>
<td>2</td>
<td>10</td>
<td>25</td>
</tr>
<tr>
<td>coal</td>
<td>3.5</td>
<td>0.28</td>
<td>1.2</td>
<td>2</td>
<td>30</td>
</tr>
<tr>
<td>pebbly sandstone</td>
<td>12</td>
<td>0.19</td>
<td>5</td>
<td>12</td>
<td>35</td>
</tr>
<tr>
<td>sandstone/f.g.</td>
<td>20</td>
<td>0.17</td>
<td>8</td>
<td>15</td>
<td>40</td>
</tr>
<tr>
<td>sandstone, shale and mudstone</td>
<td>8</td>
<td>0.25</td>
<td>7</td>
<td>10</td>
<td>40</td>
</tr>
<tr>
<td>sandstone</td>
<td>25</td>
<td>0.20</td>
<td>10</td>
<td>18</td>
<td>40</td>
</tr>
</tbody>
</table>

A variety of laboratory tests; uniaxial and triaxial compression, direct shear, point load and Brazilian tests were conducted on coal, shale, mudstone and three types of sandstone. These tests were designed to provide a complete information on the stress-strain relationship (including elastic-plastic and post-failure behaviour) and the various parameters used in different failure criteria for the rock samples. Since the procedure and equipment used for the laboratory testing were in accordance with ISRM standards (Brown 1981), only a brief description of test equipment and testing procedure is presented.
5.3.1 Preparation of samples

The validity of laboratory test results depends on the quality and quantity of samples, and to what degree the samples represent the in-situ behaviour of rocks in the strata units. In this research, the number of samples, preparation and testing procedure, and interpretation of results conformed to ISRM standards.

Although it was very difficult to prepare a truly undisturbed sample, due care was taken to reduce the possible effects resulted from any disturbance factors such as transportation, storage, preparation and testing of samples.

Somewhere around 250 samples of coal, shale, mudstone and three different types of sandstone were prepared by either coring irregular bulk samples or in-situ core drilling. Bulk samples, in the form of rough blocks of $0.4 \times 0.5 \times 0.15$ m size, were collected from underground fresh openings without the use of explosives. The openings were situated at a depth of about 450 to 500 m from the surface. These samples were sealed and covered with plastic sheets to preserve moisture and carried to the laboratory. To improve the core recovery, and also to prevent any possible damage to core samples, all of the bulk samples were encapsulated in a concrete casting. The in-situ cores were packed and carried to the laboratory with utmost care.

The coring operation was carried out perpendicular to the bedding planes using a radial ram drilling machine ("Z-J" model 23032 x 10(I)). To prevent disintegration of argillaceous samples due to water, compressed air was used as the flush medium. The spindle speed was set at 100 rpm to begin drilling and increased to about 600 rpm. The NX samples were then wrapped in cling film and kept in a safe place until the cutting and grinding operations.

The NX cores were cut into the required lengths to get standard samples for various tests (Table 5.2). After cutting the samples, both end faces of the specimen were ground to get smooth and parallel faces. The maximum deviation of end faces from parallelism did not exceed 0.05 degrees. Again, all of the specimens were covered with cling film until testing time.
Table 5.2 Dimensions of NX samples for different tests.

<table>
<thead>
<tr>
<th>Type of Tests</th>
<th>Sample Length (mm)</th>
<th>L/D Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial Compressive</td>
<td>110 - 120</td>
<td>2.00 - 2.25</td>
</tr>
<tr>
<td>Triaxial Compressive</td>
<td>105 - 115</td>
<td>2.00 - 2.15</td>
</tr>
<tr>
<td>Direct Shear</td>
<td>110 - 140</td>
<td>2.00 - 2.60</td>
</tr>
<tr>
<td>Point Load</td>
<td>65 - 80</td>
<td>1.20 - 1.50</td>
</tr>
<tr>
<td>Brazilian</td>
<td>30 - 55</td>
<td>0.60 - 1.00</td>
</tr>
</tbody>
</table>

5.3.2 Complete stress-strain test in uniaxial compression

Complete stress-strain tests under uniaxial compression are the most informative tests in rock mechanics, but are still very straight forward. Most of the characteristic properties of rock specimens can be determined by doing these tests. These properties include:

- complete stress-strain relationship
- post-failure behaviour
- uniaxial compressive strength
- elastic modulus
- Poisson's ratio
- stiffness

For the purpose of this research, two types of machines were used to carry out the uniaxial tests. There were normal and servo controlled machines. In the normal machine the rate of loading was set up and kept constant during the testing, therefore, when the specimen reached its peak strength, it crashed suddenly and it was not possible to obtain post-failure behaviour of the rock. This was due to release of elastic energy which was stored in the machine. However, the servo controlled machines performed the test at a constant rate of displacement. In the servo controlled machine, the load increases to the maximum load bearing capacity of the specimen. As the specimen starts to crush, the load gradually falls to the residual strength of the specimen. During the unloading period, some of the elastic energy stored in the machine is returned to the specimen in a controlled manner such that a balance of energy between the specimen and the machine is always maintained. The performance of the servo controlled machine is provided by a feedback control system in which the displacement rate is kept at a constant value.
Either strain gauges or transducers were used to measure displacement. In order to determine Poisson's ratio, two strain gauges were mounted on the specimen, one horizontally and one vertically. When using transducers, only the longitudinal strain of the specimen could be measured and it was not possible to determine Poisson's ratio. The displacement rate was set up within the range of 80 to 130 μm/min in order to carry out the test within 10 to 15 minutes. The accuracy of readings for load was 10 N and for displacement was 0.01 mm. Loads and corresponding strains were read at constant intervals. All the readings were sent to a control panel connected to the testing machine and recorded there. This information was processed by a computer program. The program was set up to calculate all the variables and to prepare data files relating to every test carried out.

About 60 samples were tested and results were recorded in a data file. In cases where the results were not in a reasonable range, only the failure stress (compressive strength) was included in the data file. Results of uniaxial tests are presented in the form of stress-strain curves and tangent elastic modulus calculated at 50% of the ultimate strength (Figures 5.5 to 5.10). It can be noted from these figures that only coal specimens and to some extent mudstone had residual strength in uniaxial compression conditions (more discussion will be presented after describing the triaxial compressive tests). A summary of the results from the uniaxial tests is compiled in Table 5.3.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Young's Modulus E (50%) (GPa)</th>
<th>Poisson's ratio ν</th>
<th>Stiffness K (KN/mm)</th>
<th>UCS (peak) (MPa)</th>
<th>UCS (residual) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>coal</td>
<td>range 3.0 - 4.7 4.05</td>
<td>0.24 - 0.32 0.28</td>
<td>57.1-97.6 82</td>
<td>15.0 - 32.5 23.00</td>
<td>11.0 - 15.0 14.80</td>
</tr>
<tr>
<td></td>
<td>mean 3.71 4.01</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>shale</td>
<td>range 1.11 - 1.79 1.41</td>
<td>0.13 - 0.23 0.19</td>
<td>23.6-37.7 28</td>
<td>29.8 - 35.4 31.00</td>
<td>2.0 - 2.5 1.60</td>
</tr>
<tr>
<td></td>
<td>mean 1.4 1.45</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>mudstone</td>
<td>range 5.69 - 9.35 7.93</td>
<td>0.25 - 0.32 0.28</td>
<td>105 - 150 120</td>
<td>27.3 - 42.6 30.00</td>
<td>7.5 - 10.0 8.00</td>
</tr>
<tr>
<td></td>
<td>mean 7.1 8.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>sandstone (f.g.)</td>
<td>range 5.00 - 5.80 5.60</td>
<td>0.19 - 0.22 0.21</td>
<td>75 - 88 85</td>
<td>56.0 - 68.0 58.50</td>
<td>2.0 - 10.0 0.00</td>
</tr>
<tr>
<td></td>
<td>mean 5.3 5.70</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>sandstone (m.g.)</td>
<td>range 8.80 - 10.4 9.80</td>
<td>0.19 - 0.20 0.195</td>
<td>133 - 157 148</td>
<td>59.0 - 74.0 65.5</td>
<td>1.0 - 10.0 0.00</td>
</tr>
<tr>
<td></td>
<td>mean 9.3 9.40</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>sandstone (c.g.)</td>
<td>range 12.0 - 13.7 12.5</td>
<td>0.18 - 0.20 0.19</td>
<td>180 - 206 190</td>
<td>67.0 - 80.0 75.00</td>
<td>3.0 - 10.0 0.00</td>
</tr>
<tr>
<td></td>
<td>mean 12.3 12.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
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Figure 5.5 Uniaxial compressive tests on coal specimens.

Figure 5.6 Uniaxial compressive tests on shale specimens.

Figure 5.7 Uniaxial compressive tests on mudstone specimens.
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Figure 5.8 Uniaxial compressive tests on sandstone (f.g.).

Figure 5.9 Uniaxial compressive tests on sandstone (m.g.).

Figure 5.10 Uniaxial compressive tests on sandstone (c.g.).
A specific study was carried out into the behaviour of cubic coal samples by Australian Coal Industry Research Laboratories Ltd., ACIRL, in 1989. The coal blocks were tested in ACIRL's 200 tonne servo-controlled machine using a strain control technique. Results of that investigation are illustrated in Figures 5.11 and 5.12. General conclusions from testing coal block samples are summarised below:

(i) for samples with approximately 50 mm × 50 mm base dimension, there was a considerable increase in the strength with increasing width to height ratio.
(ii) post-failure stiffness, $\lambda'$, significantly decreased with increasing width to height ratio. But, the elastic stiffness, $\lambda$, before peak strength remained constant.
(iii) the strength decreased when increasing size of the sample. It is recorded in the main report that a 150 mm cube sample had suggested an in-situ strength of about 23 MPa. This figure is not in agreement with other investigations around the world (it is generally shown that the in-situ strength of coal seams is between 5 and 7 MPa).

![Figure 5.11 Elastic modulus before peak strength, $\lambda$, and post-failure deformation modulus, $\lambda'$, of coal blocks vs width/height ratio (data after ACIRL 1989).](image)

![Figure 5.12 Compressive strength of coal cubes vs cube size (data after ACIRL 1989).](image)
5.3.3 Triaxial compressive tests

Generally speaking most rocks at depth are in a triaxial stress state. Since the response of rocks in the confined condition is different from that in an unconfined condition, it is necessary to carry out triaxial tests to determine the in-situ behaviour of rocks. Furthermore, to establish a failure criterion or determine constants for a particular failure criterion, it is essential to perform triaxial compressive tests under different confining pressures.

In this part of the laboratory investigations, a series of triaxial tests were conducted to determine the stress-strain curve of the six types of specimens under different confining pressures as well as to figure out the index parameters of various failure criteria. For each type of rock, up to ten triaxial tests were carried out on NX specimens at 5, 10, 15 and 20 MPa confining pressures (those tests where results were not reliable were deleted). The servo control which was used for uniaxial tests was used to apply axial load, and a triaxial cell with a hydraulic oil pump was used to apply the confining pressure.

Specimens were protected against the penetration of oil by means of a few millimetres thick rubber jacket. The specimen with the rubber jacket was placed inside the triaxial cell. The confining pressure was generated by using a hydraulic oil pump connected to the servo controlled machine, and the axial stress was applied to the specimen via a ram passing through a bush in the top of the cell and hardened steel end caps. The confining and axial load were increased simultaneously until the predetermined confining pressure was set up. Then the axial load was increased at a constant displacement rate while the confining pressure was kept constant. Axial deformation of the specimen was monitored by a linear variable differential transducer (LVDT) mounted outside the cell.

The data obtained from the triaxial tests can be presented in various manners depending on the final application. In this study, assessing the strength parameters of rocks in various failure criteria and evaluating the effect of confining pressure on the complete stress-strain relationship and post-failure behaviour were the main objectives of the investigation. Stress-strain curves are plotted for the tested rocks and presented in Figure 5.13 to 5.18. The general discussion on the results of uniaxial and triaxial compressive tests are presented in three parts: post-failure behaviour, failure criteria parameters and significance of confining pressure on the mechanical properties of rocks in the strata units.
Figures 5.13 to 5.18 Triaxial compressive tests on tested rocks.
5.3.3.1 Post-failure behaviour of rocks

Results of compressive tests indicated that the pre-failure behaviour of rocks show a progressive departure from elastic behaviour as the curve approaches the peak strength. This trend eventually causes the stress-strain relationship of the rock to pass through a peak and then to decrease to the residual strength. The behaviour of rock beyond the peak strength is significantly related to the type of rock. The study of post-failure behaviour has practical importance in mining where despite the occurrence of fracturing, the failed rock is still capable of supporting considerable load. This is more pronounced at depth greater than 100 m where lateral constraint is significant.

The results of uniaxial and triaxial tests obtained from the laboratory tests were very informative and helpful for preparing real input data for the finite element models. It was shown that soft rock specimens (coal, shale and mudstone) had a considerable residual strength while all types of sandstone did not show appreciable load carrying capacity beyond their peak strengths under uniaxial condition. However, when the confining pressure exceeded 5.0 MPa all rocks demonstrated significant residual strength.

5.3.3.2 Failure parameters of rocks

In this investigation three failure criteria; Mohr-Coulomb, Bieniawski and Hoek and Brown were taken into account. A complete discussion on the theory of these criteria was presented in Chapter 4. The subject of special interest here was to determine the values of parameters contributing to each of the criteria based on the results obtained from the triaxial tests; and also to discuss some problems associated with the application of these criteria.

The Mohr-Coulomb criterion defined in Equation 5.1 postulates a linear relationship between confining pressure and the compressive strength of rocks. To date, many investigations have shown that the internal angle of friction decreases with increasing the confining pressure and as such results in a reduction in the rate of increase of rock strength. Thus, the Mohr-Coulomb criterion tends to over-estimate rock strength. The parameters $C_q$ (uniaxial compressive strength) and $N_\phi$ (friction number) of this criterion can be easily determined from simple regression analysis of triaxial tests carried out under different confining pressures.
\[ \sigma_1 = C_0 + N_\phi \times \sigma_3 \]  \hspace{1cm} (5.1)

Where:
- \( \sigma_1 \) = major principal stress
- \( \sigma_3 \) = minor principal stress
- \( C_0 \) = uniaxial compressive strength of the rock sample
- \( N_\phi \) = friction number \( \left( = \frac{1 + \sin(\phi)}{1 - \sin(\phi)} \right) \) where \( \phi \) is internal angle of friction

The Bieniawski criterion is expressed in Equation 5.2.

\[ \sigma_{1n} = 1 + n.(\sigma_{3n})^m \]  \hspace{1cm} (5.2)

Where:
- \( \sigma_{1n} \) = normalised major stress \( (\sigma_{1n} = \sigma_1 / C_0) \)
- \( \sigma_{3n} \) = normalised minor stress \( (\sigma_{3n} = \sigma_3 / C_0) \)
- \( n \) & \( m \) = constants

Since \( m \) is usually less than 1.0, the term \( (\sigma_{3n})^m \) will not have a real value if \( \sigma_{3n} < 0 \). Therefore, the application of this criterion to the stability analysis of underground structures is limited to the conditions where \( \sigma_{3n} > 0 \).

The Hoek and Brown criterion is another empirical failure criterion which is widely used for rock engineering design purposes. This criterion, as defined in Equation 5.3, predicts a parabolic Mohr envelope for rock strength.

\[ \sigma_{1n} = \sigma_{3n} + \sqrt{m.\sigma_{3n} + s} \]  \hspace{1cm} (5.3)

Where:
- \( \sigma_{1n} \) = normalised major stress
- \( \sigma_{3n} \) = normalised minor stress
- \( m \) & \( s \) = constants

Although a complete discussion on the derivation of this failure criterion has been given by Hoek and Brown (1980), there are still uncertainties about estimation of constants \( m \) and \( s \). Hoek and Brown suggest values for \( m \) and \( s \) for all rocks in particular rock groups, eg. all argillaceous rocks such as siltstone, mudstone, shale, etc are in a single group having the same values for \( m \) and \( s \). This means that these rocks have the same strength according to this classification. It should be also mentioned that an advanced
statistical program is required for the multi-regression analysis of the fitting curve to the Bieniawski and Hoek and Brown criteria.

The Experimental criterion; Because of the limitations of the conventional failure criteria, an attempt was made to establish a new criterion. This new Experimental criterion is expressed in Equation 5.4.

\[
\sigma_{1n} = K + (P + T \times \sigma_{3n})^{0.5}
\]  

(5.4)

Where:
\( \sigma_{1n} \) = normalised major stress
\( \sigma_{3n} \) = normalised minor stress
K, P & T = constants. The procedure to determine constants K, P and T is as follows:

- obtain the uniaxial compressive and uniaxial tensile strengths of the rock,
- carry out two triaxial compressive tests,
- plot the above results in a coordinate system as shown in Figure 5.19, (note that the axis have been rotated for the convenience of the statistical program)

![Figure 5.19 Plot of measured values in the Experimental failure criterion.](image)

- fit a polynomial equation like \( Y = ax^2 + bx + c \) to the data, and determine parameters a, b and c. This can be done using a very simple program such as Cricket Graph on PC or MAC computers.
- calculate K, P and T as follows:

\[
K = -\frac{b}{2a} \quad P = K^2 - \frac{c}{a} \quad T = \frac{1}{a}
\]
Sensitivity analysis on the Experimental criterion showed that this criterion is sensitive to the K value and not much to P and T. It is believed that this criterion is useful for laboratory studies as well as full scale stability analysis of underground structures because of the following advantages:

(i) it presents a mathematical equation which considers the non-linear relationship between triaxial strength and confining pressure,  
(ii) it is applicable for all ranges of confining pressures, \( \sigma_3 \),  
(iii) the constants can be easily determined using a simple program,  
(iv) it is in good agreement with measured values as shown in Figures 5.20 to 5.25.

In order to determine the various parameters of the four failure criteria described above, the peak and residual strength of rocks tested are summarised in Tables 5.4 and 5.5. Regression analysis using the statistical package, SAS, was used to fit the Bieniawski and Hoek and Brown equations to the measured values. Results of this investigation are plotted in Figures 5.20 to 5.25 and summarised in Tables 5.6 to 5.9.

<table>
<thead>
<tr>
<th>Confining Pressure (MPa)</th>
<th>Coal (MPa)</th>
<th>Shale (MPa)</th>
<th>Mudstone (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>peak</td>
<td>residual</td>
<td>peak</td>
</tr>
<tr>
<td>0</td>
<td>23.04</td>
<td>14.80</td>
<td>31.00</td>
</tr>
<tr>
<td>5</td>
<td>61.25</td>
<td>43.75</td>
<td>63.93</td>
</tr>
<tr>
<td>10</td>
<td>78.75</td>
<td>47.50</td>
<td>84.40</td>
</tr>
<tr>
<td>15</td>
<td>98.75</td>
<td>56.25</td>
<td>96.89</td>
</tr>
<tr>
<td>20</td>
<td>107.50</td>
<td>70.00</td>
<td>109.84</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Confining Pressure (MPa)</th>
<th>Fine grain (MPa)</th>
<th>Medium grain (MPa)</th>
<th>Coarse grain (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>peak</td>
<td>residual</td>
<td>peak</td>
</tr>
<tr>
<td>0</td>
<td>58.50</td>
<td>----</td>
<td>65.50</td>
</tr>
<tr>
<td>5</td>
<td>72.50</td>
<td>14.00</td>
<td>80.50</td>
</tr>
<tr>
<td>10</td>
<td>89.00</td>
<td>32.50</td>
<td>98.00</td>
</tr>
<tr>
<td>15</td>
<td>100.25</td>
<td>59.00</td>
<td>113.00</td>
</tr>
</tbody>
</table>
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Figure 5.20 Failure criteria for peak strength of coal (M- Mohr-Coulomb, B- Bieniawski, H- Hoek and Brown, E-Experimental).

Figure 5.21 Failure criteria for peak strength of shale (M- Mohr-Coulomb, B- Bieniawski, H- Hoek and Brown, E-Experimental).
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Failure criteria for mudstone

Figure 5.22 Failure criteria for peak strength of mudstone (M- Mohr-Coulomb, B- Bieniawski, H- Hoek and Brown, E-Experimental).

Failure criteria for sandstone (f.g.)

Figure 5.23 Failure criteria for peak strength of sandstone (f.g.) (M- Mohr-Coulomb, B- Bieniawski, H- Hoek and Brown, E-Experimental).
Figure 5.24 Failure criteria for peak strength of sandstone (m.g.).
(M- Mohr-Coulomb, B- Bieniawski, H- Hoek and Brown, E-Experimental).

Figure 5.25 Failure criteria for peak strength of sandstone (c.g.)
(M- Mohr-Coulomb, B- Bieniawski, H- Hoek and Brown, E-Experimental).
Table 5.6 Constants in the Mohr-Coulomb criterion.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>C₀ (MPa)</th>
<th>N₀</th>
<th>mean of square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>23.0</td>
<td>4.766</td>
<td>3.55</td>
</tr>
<tr>
<td>Shale</td>
<td>31.0</td>
<td>4.351</td>
<td>1.87</td>
</tr>
<tr>
<td>Mudstone</td>
<td>30.0</td>
<td>5.371</td>
<td>3.30</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>58.5</td>
<td>2.860</td>
<td>0.02</td>
</tr>
<tr>
<td>Sandstone (m.g.)</td>
<td>65.5</td>
<td>3.179</td>
<td>0.01</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>75.0</td>
<td>4.228</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 5.7 Constants in the Bieniawski criterion.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>n</th>
<th>m</th>
<th>mean square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>4.058</td>
<td>0.585</td>
<td>0.13</td>
</tr>
<tr>
<td>Shale</td>
<td>3.331</td>
<td>0.610</td>
<td>0.02</td>
</tr>
<tr>
<td>Mudstone</td>
<td>4.112</td>
<td>0.591</td>
<td>0.23</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>2.609</td>
<td>0.941</td>
<td>0.02</td>
</tr>
<tr>
<td>Sandstone (m.g.)</td>
<td>3.278</td>
<td>1.018</td>
<td>0.00</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>1.932</td>
<td>0.571</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 5.8 Constants in the Hoek and Brown criterion.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>m</th>
<th>s</th>
<th>mean square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>17.63</td>
<td>1.0</td>
<td>0.60</td>
</tr>
<tr>
<td>Shale</td>
<td>12.71</td>
<td>1.0</td>
<td>0.40</td>
</tr>
<tr>
<td>Mudstone</td>
<td>19.15</td>
<td>1.0</td>
<td>0.70</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>4.47</td>
<td>1.0</td>
<td>0.02</td>
</tr>
<tr>
<td>Sandstone (m.g.)</td>
<td>5.25</td>
<td>1.0</td>
<td>0.02</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>8.33</td>
<td>1.0</td>
<td>0.28</td>
</tr>
</tbody>
</table>

Table 5.9 Constants in the Experimental criterion.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>K</th>
<th>P</th>
<th>T</th>
<th>mean square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>0.128</td>
<td>1.109</td>
<td>23.256</td>
<td>0.28</td>
</tr>
<tr>
<td>Shale</td>
<td>-0.007</td>
<td>1.376</td>
<td>13.624</td>
<td>0.21</td>
</tr>
<tr>
<td>Mudstone</td>
<td>-0.530</td>
<td>3.069</td>
<td>30.303</td>
<td>0.65</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>0.256</td>
<td>0.502</td>
<td>6.329</td>
<td>0.30</td>
</tr>
<tr>
<td>Sandstone (m.g.)</td>
<td>0.190</td>
<td>0.577</td>
<td>7.610</td>
<td>0.07</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>0.185</td>
<td>0.766</td>
<td>9.259</td>
<td>0.11</td>
</tr>
</tbody>
</table>
The "mean square value", which is a statistical index parameter showing the accuracy of fit of a curve, was reasonably close to zero when fitting the Bieniawski and Experimental criteria curves to the measured values. The results obtained from different criteria were used in the stability analysis of a 2-D model of a roadway to examine their applicability in FEM. The strata column and dimensions of the model are shown in Figure 5.26. Mechanical properties of rocks in the strata units are based on the best conclusion made on the laboratory results and are tabulated in Tables 5.10 to 5.14.

![Figure 5.26 Strata column and dimension of the 2-D model of the roadway.](image)

Table 5.10  Mechanical properties of strata units used in stability analysis of a roadway.

<table>
<thead>
<tr>
<th>Strata units</th>
<th>Elastic Modulus (GPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandstone (m.g)</td>
<td>10.0</td>
<td>0.20</td>
</tr>
<tr>
<td>sand. f.g. + mudstone</td>
<td>7.0</td>
<td>0.25</td>
</tr>
<tr>
<td>sand. c.g. + shale</td>
<td>5.0</td>
<td>0.20</td>
</tr>
<tr>
<td>coal</td>
<td>3.5</td>
<td>0.30</td>
</tr>
<tr>
<td>mudstone</td>
<td>8.0</td>
<td>0.25</td>
</tr>
<tr>
<td>sandstone (c.g)</td>
<td>12.5</td>
<td>0.20</td>
</tr>
</tbody>
</table>
Table 5.11 Constants for the Mohr-Coulomb criterion ($\sigma_1 = C_0 + N_\phi \times \sigma_3$).

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>UTS$_i$ (MPa)</th>
<th>UCS$_i$ (MPa)</th>
<th>$N_\phi$</th>
<th>mean square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone (m.g.)</td>
<td>4.66</td>
<td>65.5</td>
<td>3.179</td>
<td>0.01</td>
</tr>
<tr>
<td>sand f.g. + mud</td>
<td>5.40</td>
<td>44.3</td>
<td>4.115</td>
<td>1.66</td>
</tr>
<tr>
<td>sand c.g. + shale</td>
<td>3.60</td>
<td>53.0</td>
<td>4.295</td>
<td>1.11</td>
</tr>
<tr>
<td>Coal</td>
<td>1.27</td>
<td>23.0</td>
<td>4.766</td>
<td>3.55</td>
</tr>
<tr>
<td>Mudstone</td>
<td>6.80</td>
<td>30.0</td>
<td>5.371</td>
<td>3.30</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>4.66</td>
<td>75.0</td>
<td>4.228</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 5.12 Constants for the Bieniawski criterion ($\sigma_{1n} = 1 + n \times (\sigma_{3n})^m$).

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>$n$</th>
<th>$m$</th>
<th>mean square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone (m.g.)</td>
<td>3.278</td>
<td>1.018</td>
<td>0.00</td>
</tr>
<tr>
<td>sand f.g. + mud</td>
<td>3.360</td>
<td>0.766</td>
<td>0.12</td>
</tr>
<tr>
<td>sand c.g. + shale</td>
<td>2.631</td>
<td>0.591</td>
<td>0.01</td>
</tr>
<tr>
<td>Coal</td>
<td>4.058</td>
<td>0.585</td>
<td>0.13</td>
</tr>
<tr>
<td>Mudstone</td>
<td>4.112</td>
<td>0.591</td>
<td>0.23</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>1.932</td>
<td>0.571</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Table 5.13 Constants for the Hoek and Brown criterion ($\sigma_{1n} = \sigma_{3n} + \sqrt{m \cdot \sigma_{3n}^s}$).

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>$m$</th>
<th>$s$</th>
<th>mean square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone (m.g.)</td>
<td>5.25</td>
<td>1.0</td>
<td>0.20</td>
</tr>
<tr>
<td>sand f.g. + mud</td>
<td>11.81</td>
<td>1.0</td>
<td>0.36</td>
</tr>
<tr>
<td>sand c.g. + shale</td>
<td>10.52</td>
<td>1.0</td>
<td>0.34</td>
</tr>
<tr>
<td>Coal</td>
<td>17.63</td>
<td>1.0</td>
<td>0.60</td>
</tr>
<tr>
<td>Mudstone</td>
<td>19.15</td>
<td>1.0</td>
<td>0.70</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>8.33</td>
<td>1.0</td>
<td>0.28</td>
</tr>
</tbody>
</table>

Table 5.14 Constants for the Experimental criterion ($\sigma_{1n} = K + (P + T \times \sigma_{3n})^0.5$).

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>$K$</th>
<th>$P$</th>
<th>$T$</th>
<th>mean square (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone (m.g.)</td>
<td>0.190</td>
<td>0.577</td>
<td>7.610</td>
<td>0.07</td>
</tr>
<tr>
<td>sand f.g. + mud</td>
<td>-0.137</td>
<td>1.786</td>
<td>18.316</td>
<td>0.48</td>
</tr>
<tr>
<td>sand c.g. + shale</td>
<td>0.089</td>
<td>0.938</td>
<td>16.258</td>
<td>0.16</td>
</tr>
<tr>
<td>Coal</td>
<td>0.128</td>
<td>1.109</td>
<td>23.256</td>
<td>0.28</td>
</tr>
<tr>
<td>Mudstone</td>
<td>-0.530</td>
<td>3.069</td>
<td>30.303</td>
<td>0.65</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>0.185</td>
<td>0.766</td>
<td>9.259</td>
<td>0.11</td>
</tr>
</tbody>
</table>
The model was analysed under 10.0 MPa vertical stress with the ratio of horizontal to vertical stress equal to 2.5. The major and minor principal stresses, $\sigma_1$ and $\sigma_3$, were obtained for all elements and a safety factor was then calculated based on each of the failure criteria as expressed in Equations 5.5 to 5.8:

**Mohr-Coulomb**

If $\sigma_3 > T_0$ Then $SF = \frac{C + N \phi \cdot \sigma_3}{\sigma_1}$ \hspace{1cm} (5.5)

If $\sigma_3 < T_0$ Then $SF = 0$

**Bieniawski**

If $\sigma_3 > 0.0$ Then $SF = \frac{1 + n \cdot (\sigma_{3n})^m}{\sigma_{3n}}$ \hspace{1cm} (5.6)

If $\sigma_3 < 0.0$ Then $SF = 0$

**Hoek and Brown**

If $\sigma_3 > \frac{1}{2} C \cdot (m - \sqrt{m^2 + 4s})$ Then $SF = \frac{\sigma_{3n} + \sqrt{m \cdot \sigma_{3n} + s}}{\sigma_{3n}}$ \hspace{1cm} (5.7)

If $\sigma_3 \leq \frac{1}{2} C \cdot (m - \sqrt{m^2 + 4s})$ Then $SF = 0$

**Experimental**

If $\sigma_3 > T_0$ Then $SF = \frac{K + \sqrt{P + T \cdot \sigma_{3n}}}{\sigma_{3n}}$ \hspace{1cm} (5.8)

If $\sigma_3 < T_0$ Then $SF = 0$

The safety factor contour lines around the roadway calculated using the different failure criteria are plotted in Figures 5.27 to 5.30. Comparing these figures it can be seen that with the exception of minor variations close to the opening there is not much difference between the results obtained by applying the different failure criteria (these calculations are based on the laboratory results). In the next section, the in-situ strength of rocks predicting by the various failure criteria will be examined using the same model.
Figure 5.27 to 5.30  Safety factor contour lines calculated based on the Mohr-coulomb, Bieniawski, Hoek and Brown and Experimental criteria, respectively.
5.3.3.3 Significance of confining pressure on the mechanical properties of rocks

The results of the triaxial tests were used to examine the significance of confining pressure on the mode of rock deformation in the pre- and post-failure states as well as on the strength properties of the tested rocks. Variations of the basic material constants vs confining pressure are illustrated in Figures 5.31 to 5.39. The aim of this study was to determine the most realistic behaviour of rocks and to use them in numerical analyses of underground structures.

According to the results illustrated in Figures 5.31 to 5.39, the following conclusions can be made:

(i) as expected, increases in the confining pressure led to a higher peak strength for all rocks type. The increase of strength was much more pronounced in soft rocks (coal, shale and mudstone) in comparison with sandstone specimens. The increase in peak strength was 4 to 5 times the uniaxial strength for the former group and less than 2 for the latter. For rocks type which showed significant residual strength, the increase in residual strength also had the same trend as that for peak strength.

(ii) increases in the confining pressure had little effect on the elastic modulus (at 50% strength) of the soft rocks, and almost no effect on the sandstones. This characteristic of rocks would be important from the viewpoint of FEA, because when using linear elastic solutions it would not be necessary to change the tangent modulus under different confining pressures.

(iii) with increasing confining pressure the strain at peak strength increased slightly for coal and shale, and appreciably for sandstones, but remained unchanged for mudstone.

(iv) for increasing confining pressure the internal angle of friction for all rocks type decreased. This was more pronounced when confining pressure exceeded 10.0 MPa. This behaviour of rocks, therefore, indicated that a non-linear failure criteria must be used for the stability analysis of underground structures, particularly in deep mines where the confining pressure is significant.
Figures 5.31 to 5.36 Ultimate and residual strengths of rocks versus confining pressure.
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Young’s modulus envelops at 50% strength; Coarse grain, Fine grain.

Figure 5.37 Elastic modulus envelop at 50% of peak strength versus confining pressure for rocks.

Figure 5.38 Strain envelop at peak strength versus confining pressure for rocks.

Figure 5.39 Angle of internal friction envelop versus confining pressure for rocks.
5.3.4 Direct shear tests

There are some situations around underground excavations where the rocks are subject to direct shear. Under these conditions it would be necessary to consider the shear strength of the material in order to assess the stability of the structure. For the purpose of this investigation, a standard shear box was used to evaluate the direct shear parameters of the samples.

Since the direct shear strength is directly related to the normal stress applied to the plane of failure, at least four acceptable tests were carried out on each rock type at various normal stresses ranging from 0.0 to 10.0 MPa. Cylindrical NX samples with a length of about 110 cm were prepared for this test. The accuracy of measuring loads and displacements were 1 KN and 0.01 mm respectively. Measurements of load and displacement were taken at regular intervals; 10 to 15 readings were recorded before failure occurred. Results of direct shear tests are illustrated in Figures 5.40 to 5.45. It may be noted that the rate of shear displacement observed was between 0.1 to 0.5 mm/min.

5.3.5 Tensile tests

The tensile strength of rocks is much less than their compressive strength; about 1/8 to 1/10. As there are tensile zones around roadways and intersections, it would be essential to have an estimation of the tensile strength of rocks in order to analyse the stability of such structures.

There are two methods for determining the tensile strength of rock samples, direct tensile test and indirect tensile test. The indirect method can be performed either by the Brazilian or point load test. Although the direct tensile test is more accurate, indirect methods are more often used due to their simplicity, and time and cost saving. The theory behind these methods has been comprehensively explained in various rock mechanics texts.
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Direct shear tests on coal

\[ \tau = 0.1 + 0.5 \sigma \]

Direct shear tests on shale

\[ \tau = 0.15 + 0.674 \sigma \]

Direct shear tests on mudstone

\[ \tau = 1.31 + 0.485 \sigma \]

Direct shear tests on sandstone

\[ \tau = 0.62 + 1.2 \sigma \]

\( (f. g.) \)

\[ \tau = 0.46 + \sigma \]

\( (m. g.) \)

\[ \tau = 1.23 + 0.95 \sigma \]

\( (c. g.) \)

Figures 5.40 to 5.45 Direct shear strength of rocks.
5.3.5.1 Brazilian tests

For the purpose of this research, the uniaxial compressive apparatus was used to perform Brazilian tests on the specimens. For each rocks type, 10 to 20 NX samples with L/D ratio of about 0.65 to 0.75 were prepared and tested. The constant rate of displacement was set up at 0.016 mm/sec, and in this way the testing time was limited between 15 and 60 seconds. The indirect tensile strength of the specimens was calculated using Equation 5.9, and the results are compiled in Table 5.15.

\[
\sigma_t = \frac{2P}{\pi L D}
\]  
(5.9)

Where:

- \(\sigma_t\) = indirect tensile strength (MPa)
- \(P\) = failure load (N)
- \(L\) = length of the specimen (mm)
- \(D\) = diameter of the specimen (mm)

Table 5.15 Tensile strength of the rocks obtained from Brazilian tests.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>range</td>
</tr>
<tr>
<td>Coal</td>
<td>0.89 - 1.67</td>
</tr>
<tr>
<td>Shale</td>
<td>1.60 - 3.87</td>
</tr>
<tr>
<td>Mudstone</td>
<td>5.67 - 8.31</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>2.32 - 6.60</td>
</tr>
<tr>
<td>Sandstone (m.g.)</td>
<td>2.60 - 6.65</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>2.92 - 5.90</td>
</tr>
</tbody>
</table>
5.3.5.2 Point load tests

This is a field version of the uniaxial compressive test which gives an index of strength the so called point load index, $I_s$. This index is used to estimate the uniaxial compressive and tensile strength of samples as expressed in Equations 5.10 to 5.14.

$$I_s = \frac{P}{D^2} \quad (5.10)$$

Where:

$I_s$ = point load index (MPa)
$P$ = axial load (N)
$D$ = diameter of the specimen (mm); the standard value is 50 mm, otherwise the following correction should be carried out (Equation 5.11).

$$I_s(50) = n.I_s \quad (5.11)$$

$$n = (D/50)^{0.45} \quad (5.12)$$

$$\sigma_c = C.I_s(50) \quad (5.13)$$

$$\sigma_t = t.I_s(50) \quad (5.14)$$

Where:

$\sigma_c$ = indirect compressive strength (MPa)
$\sigma_t$ = indirect tensile strength (MPa)
$I_s(50)$ = standard point load index (for a specimen with 50 mm in diameter)
C and $t$ = constant values. C and $t$ have been suggested by Bieniawski (1974), Hoek and Bray (1977), and Hassani et al (1980); C = 22 to 29 and $t$ = 0.8 to 3.0

Point load tests were conducted on specimens with saw cut faces prepared for the purpose of this test. Specimens were located between platens and the load was continuously increased such that failure occurred between 10 and 60 seconds. Results are summarised in Table 5.16. Since the C and $t$ values are mostly suggested for hard rocks, the results show some discrepancies, particularly for coal, shale and mudstone.
Table 5.16 Indirect compressive and tensile strengths obtained from point load tests.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Point load index</th>
<th>Compressive Strength $\sigma_c$ (MPa)</th>
<th>Tensile Strength $\sigma_t$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coal</td>
<td>range: 0.29 - 0.77</td>
<td>6.96 - 18.48</td>
<td>0.27 - 0.72</td>
</tr>
<tr>
<td></td>
<td>mean: 0.50</td>
<td>12.00</td>
<td>0.47</td>
</tr>
<tr>
<td>Shale</td>
<td>range: 1.20 - 2.10</td>
<td>28.80 - 50.40</td>
<td>1.30 - 1.97</td>
</tr>
<tr>
<td></td>
<td>mean: 1.78</td>
<td>42.72</td>
<td>1.67</td>
</tr>
<tr>
<td>Mudstone</td>
<td>range: 0.50 - 2.90</td>
<td>12.00 - 69.60</td>
<td>0.47 - 2.73</td>
</tr>
<tr>
<td></td>
<td>mean: 1.54</td>
<td>36.96</td>
<td>1.45</td>
</tr>
<tr>
<td>Sandstone (f.g.)</td>
<td>range: 1.20 - 3.71</td>
<td>28.8 - 89.04</td>
<td>1.13 - 3.49</td>
</tr>
<tr>
<td></td>
<td>mean: 2.60</td>
<td>62.40</td>
<td>2.45</td>
</tr>
<tr>
<td>Sandstone (m.g.)</td>
<td>range: 1.40 - 3.04</td>
<td>33.60 - 72.96</td>
<td>1.32 - 2.86</td>
</tr>
<tr>
<td></td>
<td>mean: 2.44</td>
<td>58.56</td>
<td>2.30</td>
</tr>
<tr>
<td>Sandstone (c.g.)</td>
<td>range: 1.27 - 3.17</td>
<td>30.48 - 76.08</td>
<td>1.20 - 3.00</td>
</tr>
<tr>
<td></td>
<td>mean: 2.10</td>
<td>50.40</td>
<td>2.00</td>
</tr>
</tbody>
</table>

5.3.6 Scale effect and in-situ properties

In view of the very high cost of obtaining information on the in-situ properties of strata units and of the scarcity of such information, a comprehensive quantitative study of the in-situ properties was not carried out in this research. However, some attempts have been made to use whatever information is available to provide some form of general guidance on the relationship between laboratory results and the in-situ properties of rocks in the strata units.

When using laboratory test results to analyse the stability of underground structures, there is always some concern as to what extent the test on small size samples is representative of large in-situ rock masses. Accordingly, this aspect of rock mechanics is reviewed here briefly to help in making some correlation between the laboratory measured quantities and the in-situ values.

The size and scale effect is generally considered by applying a reduction factor to the measured parameters such as uniaxial compression strength, cohesion, angle of internal friction, elastic modulus and Poisson's ratio. Moreover, there are some differences between the shape of the stress-strain curve obtained from laboratory testing and that from in-situ conditions.

Literature relating to this subject indicated that although the size effect and in-situ properties have been the centre of interest among mining engineers for the last 30 years,
there is not a standard procedure or approach to the key-function of scale effect. It is believed that engineering judgement and local experience are still the basic consideration in selecting scale factors. However, there are a number of investigations on this issue, Bieniawski (1984), Tang and Peng (1988), Cunha (1990) which have been reviewed. It was decided to use the results of these investigations for the purpose of the present research.

5.3.6.1 Scale effect on the strength and elastic modulus of rocks

In the last 30 years, many attempts have been made to establish a quantitative relationship between laboratory results and the in-situ properties of rocks. Among them, Tang and Peng (1988) used a reduction factor of 1/4 to 1/6 on the laboratory values for coal, and it was reported that results correlated well with field observations. In another investigation, Cunha (1990) indicated that for soft rocks, the reduction factor might be 1/2 to 1/5, but for hard rock, this reduction is more pronounced, between 1/5 to 1/10. (these are not in agreement with reduction factors suggested by others like Wilson, 1980). He exempted elastic modulus from this rule and suggested that the mean value of elastic modulus obtained from laboratory tests is not very far from the in-situ elastic modulus.

There have also been many attempts to make a correlation between RQD (rock quality designation) and $E_m/E_l$ (the ratio of rock mass modulus $E_m$ to laboratory modulus $E_l$). In this regard, Coon and Merritt (1970) defined a correlation factor of 0.544 for this relationship. Also, Heuze (1980) reported that the moduli values measured in the laboratory are, on the average, 2.5 times higher than the in-situ values. In particular, most of the in-situ modulus results seemed to be between 0.2 to 0.6 of the laboratory values.

For good quality rock masses, $RMR > 50$, Bieniawski (1978) obtained a direct correlation between the in-situ modulus and rock mass rating (RMR) as expressed in Equation 5.15.

$$E_m = 2 \cdot RMR - 100 \quad (5.15)$$

Where:

$E_m$ = Rock mass elastic modulus (GPa)

$RMR$ = Rock mass rating
This correlation was re-examined by Serafim and Pereira (1983), quoted by Bieniawski (1984), and they provided many results in the range of RMR < 50 which showed a non-linear relationship between RMR and $E_m$.

The strength reduction from laboratory measured data to in-situ values has received considerable attention in the past, particularly as related to mine pillar designs (Bieniawski, 1984). Many attempts have been made to explain this relationship by means of empirical relationships. Four of these works are reviewed here.

*Protodyakonov (1964)* expressed the relationship between the strength of a specimen and the strength of the in-situ rock mass as Equation 5.16.

\[
\frac{\sigma_d}{\sigma_m} = \frac{d + m}{b + d/b} \quad (5.16)
\]

Where:
- $\sigma_d$ = strength of cubic specimen with side length of $d$ (MPa)
- $\sigma_m$ = in-situ strength of the rock mass (MPa)
- $b$ = distance between discontinuities in the rock mass (m)
- $d$ = side length of the specimen (m)
- $m$ = strength reduction factor, given in Table 5.17

<table>
<thead>
<tr>
<th>Rock Strength (MPa)</th>
<th>Stress State</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Compression</td>
</tr>
<tr>
<td>$&gt; 75$</td>
<td>$2 &lt; m &lt; 5$</td>
</tr>
<tr>
<td>$&lt; 75$</td>
<td>$5 &lt; m &lt; 10$</td>
</tr>
</tbody>
</table>

*Wilson (1980)* suggested that to obtain the in-situ strength of the rock mass, the unconfined laboratory strength, $C_0$, be divided by a factor $f$ where

- $f = 1$ for strong massive unjointed rock (including concrete),
- $f = 2$ for widely-spaced joints or bedding planes in strong rocks,
- $f = 3$ for more jointed, but still massive rocks,
- $f = 4$ for well-jointed and weaker rocks,
- $f = 5$ for unstable seat earth and closely-cleated rock such as coal,
- $f = 6$ and 7 for weak rock in the neighbourhood of fault zones.
Hoek and Brown (1980) expressed the correlation between laboratory compressive strength and in-situ strength in the form of an empirical failure criterion which was presented in Equation 5.3 earlier in this section. Constants $m$ and $s$ in Equation 5.3 are dependent on the properties of the rock and the extent to which the rock mass has been fractured by being subjected to $\sigma_1$ and $\sigma_3$. A range of values for $m$ and $s$ was given for different rocks type as well as for in-situ situations (Hoek and Brown, 1980). For rock masses, Priest and Brown (1983) proposed the following estimations for $m$ and $s$:

$$m = m_1 \cdot \exp\left(-\frac{RMR-95}{13.4}\right)$$  \hspace{1cm} (5.17)

$$s = \exp\left(-\frac{RMR-100}{6.3}\right)$$  \hspace{1cm} (5.18)

For intact rock $m = m_1$ is determined from a fit of Equation 5.3 to triaxial test data from laboratory specimens; taking $s = 1$ for intact rock.

Weakening Coefficient is a classification system for coal measure formations proposed by Singh (1986). This system was developed to define a reduction factor, WC (weakening coefficient), which could be applied to the intact sample values in order to determine the rock mass properties. This system includes the following parameters:

- Rock quality designation, RQD,
- Joint spacing index, $K_1$,
- Joint surface index, $K_2$,
- Joint filling index, $K_3$,
- Joint aperture index, $K_4$.

The overall joint coefficient and the weakening coefficient are calculated using Equations 5.19 and 5.20.

$$K = K_1 \times K_2 \times K_3 \times K_4$$  \hspace{1cm} (5.19)

$$WC = K \times RQD$$  \hspace{1cm} (5.20)

Where $WC$ is the weakening coefficient of the rock mass and $RQD$ is the rock quality designation. A correlation was made between RMR values and the corresponding weakening coefficient by Gahrooe (1989) as expressed in Equation 5.21.
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\[ WC = 0.018 \times e^{(0.039 \text{ RMR})} \]  

(5.21)

The weakening coefficient may be used to determine the constants \( m \) and \( s \) in the Hoek and Brown criterion by using the following empirical equations:

\[
\log (k_1) = 0.118 + 1.827 \times \log (WC) \]

(5.22)

\[
\log (k_2) = 0.047 + 4.052 \times \log (WC) \]

(5.23)

\[ m = m_i \times k_1 \]

(5.24)

\[ s = s_i \times k_2 \]

(5.25)

Where:

\( WC \) = weakening coefficient of the rock mass  
\( m \) & \( s \) = constants for the rock mass in the Hoek and Brown criterion  
\( m_i \) & \( s_i \) = constants for intact rock in the Hoek and Brown criterion

For the purpose of this study, the South African Council for Scientific and Industrial Research (CSIR) classification system, the RMR system, was used to evaluate the strata units behaviour. The WC system, using the weakening coefficient, was used to determine the in-situ properties of the strata units from laboratory values.

Assessment of the geotechnical parameters for different strata units including: uniaxial strength of intact samples (UCS), rock quality designation index (RQD), joint spacing, joint condition, ground water condition and the effect of joint strike and dip orientations in the roadway are carried out according to the following descriptions and summarised in Table 5.18.

The uniaxial compressive strength of intact rocks were taken from Table 5.3. The RQD and joint spacing were calculated from \( 2 \times 12 \) m cores obtained from underground borehole drillings at the site of investigation. The ground water condition was surveyed in the gate entries of Longwall LW10. The general water condition was dry to wet, and it was not a serious problem in the stability of structures; therefore, class I and class II were taken into account for different strata units in this regard. Since all roadways were driven within the coal seam (parallel to the bedding planes), the effect of joint strike and dip orientation in roadways fell in the third category: "Dip 0° - 20° irrespective of strike" which is the "unfavourable" condition. As a result, an adjustment factor of -10 is considered on the overall RMR rating.
Table 5.18 Geotechnical parameters used to determine the RMR of the strata units.

<table>
<thead>
<tr>
<th>Strata unit</th>
<th>UCS (MPa)</th>
<th>RQD (%)</th>
<th>spacing of joints (mm)</th>
<th>condition of joints</th>
<th>water condition</th>
<th>RMR</th>
<th>RMR (adjusted)</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandstone (m.g)</td>
<td>65 (7)</td>
<td>55.3 (13)</td>
<td>103 (10)</td>
<td>class II (20)</td>
<td>class I (10)</td>
<td>50 (60)</td>
<td></td>
</tr>
<tr>
<td>sand. f.g. + mudstone</td>
<td>40 (7)</td>
<td>78.5 (17)</td>
<td>165 (10)</td>
<td>class II (20)</td>
<td>class II (7)</td>
<td>51 (61)</td>
<td></td>
</tr>
<tr>
<td>sand. c.g. + shale</td>
<td>31 (4)</td>
<td>44.1 (8)</td>
<td>107 (10)</td>
<td>class III (12)</td>
<td>class II (7)</td>
<td>31 (41)</td>
<td></td>
</tr>
<tr>
<td>coal</td>
<td>23 (2)</td>
<td>25-50 (8)</td>
<td>50-120 (10)</td>
<td>class III (12)</td>
<td>class I (10)</td>
<td>32 (42)</td>
<td></td>
</tr>
<tr>
<td>mudstone</td>
<td>30 (4)</td>
<td>&lt;25 (3)</td>
<td>60-100 (10)</td>
<td>class II (20)</td>
<td>class I (10)</td>
<td>37 (47)</td>
<td></td>
</tr>
<tr>
<td>sandstone (c.g)</td>
<td>75 (7)</td>
<td>78 (17)</td>
<td>175 (10)</td>
<td>class II (20)</td>
<td>class I (10)</td>
<td>54 (64)</td>
<td></td>
</tr>
</tbody>
</table>

In addition to the RMR index, the Norwegian Geotechnical Institute (NGI) system (Q index) and the Central Mining Research Station (CMRS) system (R index) were also examined. The NGI tunnelling quality index (Q) developed by Barton et al (1974) is based on the evaluation of a large number of case histories of underground excavation stability, particularly civil engineering cases. There are three suggestions to correlate RMR and Q indexes denoted in Equations 5.26 to 5.28.

\[
RMR = 19 \ln (Q) + 26 \quad (Singh, 1986) \quad (5.26)
\]

\[
RMR = 9 \ln (Q) + 44 \quad (Bieniawski, 1989) \quad (5.27)
\]

\[
RMR = 18.79 \ln (Q) + 13.48 \quad (Sunu, 1988) \quad (5.28)
\]

The CMRS classification is a new system developed by Venkateshwarlu and Raju (1987). This system is proposed for coal measure rocks and is based on the evaluation of 52 collieries in India, some of those included more than one case study. The method has five basic parameters as follows:

- Layer thickness (approximately equivalent to RQD)
- Structural features (faults, slips, slickensides, joint sets, structural irregularities)
- Slake durability
- Intact rock strength
- Ground water

Besides the above factors, the stress state (including depth of cover and horizontal stresses) and proximity of other excavations were taken into account in the CMRS calculation of the R index. Based on 44 case studies by Sheorey (1991), the following correlation was made between R (CMRS system) and Q (NGI system):

$$R = 46 + 12 \log (Q)$$  \hspace{0.5cm} (5.29)

In the present research, the Q and R indexes are also determined using the RMR values and Equations 5.27 and 5.29. The Q and R values determined for various strata units encountered in this investigation are in the range of those given by Sheorey (R = 35 to 60). The results for Ellalong strata units are tabulated in Table 5.19. These values will be used to estimate rock load or mean support load density (MLD) in roadways and intersections in the following Chapters.

<table>
<thead>
<tr>
<th>Strata units</th>
<th>RMR (CSIR)</th>
<th>Q (NGI)</th>
<th>R (CMRS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandstone (m.g)</td>
<td>50</td>
<td>2.0</td>
<td>49</td>
</tr>
<tr>
<td>sand. f.g. + mudstone</td>
<td>51</td>
<td>2.2</td>
<td>50</td>
</tr>
<tr>
<td>sand. c.g. + shale</td>
<td>31</td>
<td>0.2</td>
<td>38</td>
</tr>
<tr>
<td>coal</td>
<td>32</td>
<td>0.3</td>
<td>39</td>
</tr>
<tr>
<td>mudstone</td>
<td>37</td>
<td>0.5</td>
<td>42</td>
</tr>
<tr>
<td>sandstone (c.g)</td>
<td>54</td>
<td>3.0</td>
<td>52</td>
</tr>
</tbody>
</table>

The weakening coefficient, WC, and constants of strata units m and s, in the Hoek and Brown criterion are determined by applying the RMR values into Equations 5.21 to 5.25. These values are compared with those calculated using Equations 5.17 and 5.18 proposed by Priest and Brown (1983). Although both methods gave very low values for m and s, the weakening coefficient method seemed to be more conservative. Results of the study are tabulated in Table 5.20. The uniaxial in-situ compressive strength has been calculated using Equation 5.3, taking $\sigma_3=0$ and substituting constant s from Table 5.20

$$\text{UCS} = (s)^{0.5} \times \text{UCSi}$$.
CHAPTER 5: Site Investigation and Data Acquisition for Numerical Analysis

Table 5.20 Constants m and s for strata units based on the weakening coefficient and the Priest and Brown equations.

<table>
<thead>
<tr>
<th>Strata</th>
<th>Index</th>
<th>WC</th>
<th>Priest and Brown</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RMR</td>
<td>WC</td>
<td>m</td>
</tr>
<tr>
<td>sandstone (m.g)</td>
<td>50</td>
<td>0.127</td>
<td>0.158</td>
</tr>
<tr>
<td>sand. f.g. + mudstone</td>
<td>51</td>
<td>0.132</td>
<td>0.381</td>
</tr>
<tr>
<td>sand. c.g. + shale</td>
<td>31</td>
<td>0.060</td>
<td>0.082</td>
</tr>
<tr>
<td>coal</td>
<td>32</td>
<td>0.063</td>
<td>0.147</td>
</tr>
<tr>
<td>mudstone</td>
<td>37</td>
<td>0.076</td>
<td>0.228</td>
</tr>
<tr>
<td>sandstone (c.g)</td>
<td>54</td>
<td>0.148</td>
<td>0.333</td>
</tr>
</tbody>
</table>

The in-situ values for m and s in the Hoek and Brown failure criterion given in Table 5.20, were used to calculate the in-situ safety factor around the roadway. This was done to examine if these suggested methods could be used in the FEM. Figure 5.47 shows the safety factor contour lines around the roadway based on the in-situ values of m and s. It can be seen that the results are very far from reality. It is believed that the estimation of in-situ values of m and s using the Priest and Brown and the weakening coefficient methods are an appreciable underestimate of the in-situ strength of the rock mass, in particular the coal measures. These values should not be used for stability analysis of underground structures by the FE method.

An attempt was made to solve the foregoing problem of finding the in-situ values of rock mass strength. The reduction coefficients for m and s in Equations 5.17 and 5.18, as

\[ r_m = \exp \left( \frac{\text{RMR} - 95}{13.4} \right) \text{ and } \quad r_s = \exp \left( \frac{\text{RMR} - 100}{6.3} \right) \]

were plotted against the RMR (Figure 5.46). It can be seen that the reduction factor is significant when RMR < 50. Many different equations were tested but when the results were utilised to calculate the in-situ safety factor of the elements around the roadway, unrealistic results were obtained. It was then decided to develop a new method in which the reduction factor is applied to the safety factor rather than to the individual constants m and s. The following is a new approach for estimation of the in-situ strength of strata units based on the RMR index. In the Experimental failure criterion, the safety factor is calculated using parameters obtained from laboratory tests. The overall reduction coefficient of the safety factor, \( r_{sf} \), is then calculated based on the RMR values for various rock types as shown in Figure 5.46 and as follows:
Figure 5.46 Reduction factor of constants $m$ and $s$, $r_m$ and $r_s$, based on the Priest and Brown equations, and the safety factor reduction coefficient, $r_{sf}$, in the Experimental criterion.

\[ r_{sf} = 0.1\sqrt{RMR} \]  \hspace{1cm} (5.30)

\[ SF_{\text{in-situ}} = r_{sf} \times SF_{\text{intact}} \]  \hspace{1cm} (5.31)

Where:

- $r_{sf}$ = reduction coefficient to safety factor
- $SF_{\text{in-situ}}$ = safety factor calculated for the in-situ condition
- $SF_{\text{intact}}$ = safety factor calculated based of the laboratory testing results

Figure 5.48 shows the in-situ safety factor contour lines around the roadway calculated using the new approach. It can be seen from Figures 5.47 and 5.48 that the Experimental criterion in conjunction with the new approach of calculating the in-situ strength gave more realistic predictions of the failure zone around the roadway.
5.3.6.2 Scale effects on the stress-strain relationship of rocks

There are limited investigations describing the scale effect on the stress-strain relationship obtained from laboratory testing (Heuze and Salem 1977). Generally speaking, when compared with the in-situ values, many rock types behave more linear-elastic when intact or laboratory scale samples are tested. The same rock in-situ generally is more deformable and shows elastic-plastic behaviour. However, there is no certain rule to find out the in-situ stress-strain curve from laboratory results. For rocks surrounding deep structures, at depths of more than 400 m, it is most likely that the ductile behaviour of rocks before and after the peak strength would be more pronounced. On the other hand, in most cases of the FE analysis, an ideal elastic or an ideal elastic-perfectly plastic material model does not give very accurate results. It is suggested, therefore, that it would be more reasonable to use a complete stress-strain curve including ductile behaviour for rocks in the strata units.

For the purpose of FEA in Chapters 6 and 7, the stress-strain relationship for strata units at Ellalong Colliery were obtained from triaxial compressive results at 5 MPa confining pressure presented in Figures 5.13 to 5.18. These were used for non-linear solution
techniques. For simple linear elastic solutions, however, material properties summarised in Table 5.21 will be utilised.

<table>
<thead>
<tr>
<th>Strata units</th>
<th>elastic modulus (GPa)</th>
<th>Poisson's Ratio</th>
<th>UCS (MPa)</th>
<th>UTS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandstone (m.g)</td>
<td>10.0</td>
<td>0.20</td>
<td>45.0</td>
<td>4.0</td>
</tr>
<tr>
<td>sand. f.g. + mudstone</td>
<td>7.0</td>
<td>0.25</td>
<td>35.0</td>
<td>4.0</td>
</tr>
<tr>
<td>sand. c.g. + shale</td>
<td>5.0</td>
<td>0.20</td>
<td>25.0</td>
<td>3.0</td>
</tr>
<tr>
<td>coal</td>
<td>3.5</td>
<td>0.30</td>
<td>15.0</td>
<td>1.5</td>
</tr>
<tr>
<td>mudstone</td>
<td>8.0</td>
<td>0.25</td>
<td>20.0</td>
<td>4.0</td>
</tr>
<tr>
<td>sandstone (c.g)</td>
<td>12.5</td>
<td>0.20</td>
<td>50.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

5.4 In-situ measurements and underground observations

To verify the FE results, a field study was carried out from August 92 to July 93 and also major previous reports relating to the subject of inertest were consulted; ACIRL 1972, 1987 and 1989, Fuller et at 1987, Fuller and O'Grady 1990 (Barrett, Fuller & Partners, BFP), Fabjanczyk and Guy 1990, Gale 1991 (Strata Control Technology, SCT).

It is a fact that the field data are always neither complete nor conclusive, and on the other hand, there are some limitations with FE modelling and analysis. Therefore, the two groups of results, from numerical modelling and field investigation, must be compared with caution. The comparison should be more qualitative than quantitative because of the following reasons:

(a) Properties of rocks utilised in FE models are based on the conclusions made from laboratory results. Although some reduction factors were applied, the in-situ properties might vary to some extent. The discrepancies are more pronounced for the mechanical properties of the bedding planes.
(b) The strata column taken for FE modelling is a general one while variations from site to site, and specific features may occur in the real situation.
(c) The behaviour of rocks are usually time dependent according to the type of rock and features of the structure. Results of field instrumentation include the time effect while FEA gives final changes after complete relaxation of the structure.
(d) The method, equipment, operation and accuracy of different instruments are still major areas of study; hence, the measured values may have some deviation from the actual values of stress and displacement.

However, many attempts have been made to more closely match the predicted values from FEA with field results. Results from field investigations are interpreted in the following sections.

### 5.4.1 Field measurements of stress

The in-situ stress state was measured of a number of sites, one of them in the inby section of 2 East area using CSIRO and USBM stress cells (SCT 1991). These measurements were aimed to investigate virgin horizontal stress above the broken zone of the roadway as well as the stress magnitude transferred through the broken roof as an estimation of the residual strength. These measurements were conducted by overcoring and hydro-fracturing at different locations. It was reported that there were difficulties in measuring virgin stresses due to severe rock discing and borehole break-out during drilling and overcore monitoring. But the stress transferred through the roof strata was successfully measured by means of USBM cells. The results of this study are given in Table 5.22.

| Table 5.22 In-situ stress measurements at Ellalong Colliery (SCT 1991). |
|---|---|---|---|---|---|
| **Virgin stress measurements** | | | | | |
| test | hole | instrument | E (GPa) | $\sigma_1$ (MPa) | $\sigma_2$ (MPa) | $\sigma_1$ bearing $^\circ$ (E of N) |
| 1 | 1 | HI | 16 | 34.7 | -- | Approx. 55 |
| 5 | 2 | USBM | 19 | 39.0 | 21.0 | 48 - 51.5 |
| 6 | 3 | USBM | 16 | 32.1 | 22.1 | 45.5 - 54 |
| **Stress transferred through the roof strata** | | | | | | |
| test | hole | instrument | E (GPa) | $\sigma_1$ perpendicular to the roadway (MPa) | horizon above the roof (m) |
| 13 | 4 | USBM | 5.7 | 8.7 | 1.47 |
| 14 | 4 | USBM | 8.0 | 7.3 | 1.8 |
| 15 | 4 | USBM | 8.4 | 9.9 | 2.1 |
The stress state (direction) at Ellalong Colliery is shown in Figure 5.49. Vertical stress is approximately proportional to the overburden weight at 2.5 MPa/100 m depth, about 12.5 MPa at a depth of 500 m. The major horizontal stress, $\sigma_1$, varies between 30 MPa and 40 MPa and its direction is 45° to 55° E of N. The minor horizontal stress is 15 MPa to 22 MPa perpendicular to the $\sigma_1$ direction. These values are greater than the average values for the Australian continent. Variations are primarily due to rock stiffness, depth and effect of local structures.

![Stress directions (virgin stresses) at Ellalong Colliery.](image)

**5.4.2 Roof displacement and height of the softening zone**

Roof displacement and height of the softening zone were monitored using extensometers at a number of locations (these locations were previously shown in Figure 5.2). It was indicated that the height of the softening zone was the height where considerable roof displacement was initiated (SCT 1991). This was, in most cases, the maximum height of roof falls. Results of field measurements are summarised in Figure 5.50. Variation in roof displacement and height of the softening zone are due to the changes in the horizontal stress perpendicular to the roadway, the width of the roadway and roof lithology.
The relationship between parameters such as the width of the opening, horizontal stress, roof displacement and the height of roof falls is almost the same for results obtained from both the field instrumentation and FEA (Results of FEA are given in Chapters 6 and 7). General conclusions from the field investigations are as follows:

(i) roadways parallel to the maximum horizontal stress will have less displacements and shorter height of roof falls.

(ii) maximum displacement at the centre of the roof is dependent on the stress state (horizontal and vertical stresses), height and width of the opening, lithology and mechanical properties of surrounding strata.

(iii) height of the softening zone, the potential roof fall height, is governed by the width of the opening and the horizontal stress value. The heights range from 1.5 m to 4 m for low horizontal stresses and from 4 m up to a maximum of 7 m for very high horizontal stresses. These figures are for roadways with widths ranging from 4.5 m to 5 m.

(iv) high stress concentrations during retreat of longwall faces had enormous effect on the behaviour of the tail gate roadways. It is suggested that when modelling this situation using the FEM, elastic modulus and strength of rocks should be reduced while increasing the vertical stress up to 3 to 4 times the normal value.
(v) roof displacement increased up to 250 mm, depending on the location of monitoring site with respect to the face end, but the height of softening zone remained unchanged at locations where there was no further effect from the adjacent working panels, it was around 7 m for the worst condition.

(vi) it was indicated that failure in the roof was usually initiated from 1 m above the roof line at low stress concentration levels and extended into the upper zones, up to 5 m into the roof. In cases that high stress concentration zones were closer to the roof line, appreciable rock fracturing occurred high above the roof (Figure 5.51).

![Figure 5.51](image)

Figure 5.51 Relationship between roof displacement and height of the softening zone, (data after SCT 1991).

5.4.3 Stress monitoring around the finish position of retreat longwalls

A program of monitoring was conducted in the main-gate near finish position during the final stages of the retreat of longwall No.3. This work was carried out under a contract between Ellalong Colliery and ACIRL (ACIRL 1987). The aim of the project was to monitor roof behaviour at the finish position. This investigation had three different parts. Lateral roof strain was measured by triangular arrays of pins at three sites. The roof failure zone was monitored across the roadway via five 6.2 m long borescope holes. Roof bolt torques were tested to determine if the tension in the bolts was lost as the face approached. The results of the study are summarised below;
(i) the roof surface area was expanded as the face approached to within 25 m of each site. The maximum elongation recorded in ten days was 0.51%. The shorter axis of the strain ellipse (indicating the direction of the maximum horizontal stress) made an angle of 50° to the roadway axis direction (RAD). But, when a deep rib-side gutter developed on the longwall block side, the strain ellipse rotated such that the shorter axis made an angle of 21° to the RAD.

![Figure 5.52 Lateral roof strain at the finish position of a retreat longwall, [A] the face at 25 m distance from measuring site, [B] the face close to the site resulting in a rib-side gutter on the longwall block-side.](image)

(ii) the borescope observations indicated that initial fracturing first occurred when the face was 50 m away. The fracture zone was concentrated over the chain pillar side up to 4.1 m into the roof. Another fracture occurred at a height of 6.1 m and a few minor ones developed on the longwall block side above and below the bolting horizon. Later fracturing occurred when the face was 35 m away. This time, it was located low on the chain pillar side but quickly spread up to 4 m on the longwall block side. Finally major shear zones were spread right across the roof, approximately at the seam horizon when the face was 10 m away. It was shown that fracturing was initially confined to the roof while the top coal in the immediate roof was still intact and had good condition except for slight sagging. As the face approached to within 10 m, the top coal also began to develop lateral shears and as a consequence, the immediate roof started to crack and gutter.

(iii) results of roof bolt torque testing did not show any damage by the roof shearing on the bolt tension. The bolt tension was maintained or increased as the face approached.

This work and underground inspections confirmed that lateral stress, orientation of the gate entries, property of roof rocks and distance of the face from finish position are the
major parameters contributing to the behaviour of the roof and any roof fall accidents at the finish positions.

5.4.4 Roof fall surveys

A number of sites were inspected for roof fall incidents as shown previously in Figure 5.2. Based on the available information, three factors were found to be involved with the roof falls as follows:

(i) high lateral stress concentrations, particularly around the south east corner of the finish position of the north district longwalls.

(ii) deterioration of the physical properties of roof rocks in gate entries after extraction of the first longwall and damage around end-junction due to high abutment pressure from the retreat of the second longwall. Maximum damage to the roof occurred when the abutment stress of the second longwall acted on the roof strata which had already been weakened by stress concentrations from the previous longwall.

(iii) existence of a very low strength and laminated sandstone over the coal seam. This condition caused some serious roof falls, particularly at intersections in 2 East development.

The three-dimensional shape of roof falls were not recorded in previous investigations and only a few of roof falls were surveyed during the course of the present study. Based on the surveyed cases, it was indicated that the bottom diameter of the roof falls were limited to the span of the roadways or the diagonal span of intersections. Guttering started from either side and then propagated into the roof making an arch zone over the opening. The height of the roof falls varied from site to site mainly relating to the strength of rocks, stress levels and span of the openings. Average bottom diameter and height of the roof falls ranged from 4.0m to 6.5 m and from 1.5 m to 7.0 m, respectively.

5.5 Conclusions

Extensive laboratory testing was carried out on six rocks type. The mechanical properties of these rocks including complete stress-strain curve, elastic modulus, Poisson's ratio, compressive and tensile strength were obtained. In addition, theoretical and empirical failure criteria were examined against the triaxial compressive test results.
Some problems encountered with the conventional criteria were discussed and as a result a new experimental criterion was established.

Attention was also drawn to the effect of scale upon the mechanical properties of the rock mass. An attempt was made to use available techniques and methods to predict the in-situ properties of strata units from laboratory results. It was shown that previous methods for determining the in-situ parameters of the Hoek and Brown criterion were too conservative. Therefore, a new approach was proposed to estimate the in-situ safety factor of rocks around an excavation based on the RMR index.

In addition to the above, a field investigation program was carried out at Ellalong Colliery from where the rock samples were collected. The results of this investigation included all the data on the stress and displacement patterns around the underground structures.
CHAPTER SIX

STABILITY EVALUATION OF ROADWAYS
(2-D FINITE ELEMENT MODELS)
CHAPTER 6

STABILITY EVALUATION OF ROADWAYS
(TWO-DIMENSIONAL FINITE ELEMENT MODELS)

6.1 Introduction

Many general and site-specific investigations have been carried out to determine strata behaviour around roadways in underground coal mines (Lackey 1973, Choi et al 1975, Whittaker and Singh 1981, Jeremic 1981, Pariseau and Eitani 1981, Mark and Bieniawski 1986, Peng 1986, Gale 1986, Shephered 1987, Haramy and Kneisley 1989, Dar and Smelser 1990, Singh et al 1982 and Frith et al 1990). A variety of methods such as theoretical, experimental, empirical and numerical techniques were utilised to assess the stability of roadways and to identify important parameters affecting stability. It has been shown that the following parameters may significantly affect the behaviour and stability of roadways:

(a) state of virgin stress, particularly high horizontal stress,
(b) geometry and dimension of the structure,
(c) mechanical properties of rocks in the strata units,
(d) abutment pressures resulting from longwall faces,
(e) abnormal or irregular geological features.

This Section indicates the influence of important parameters on the behaviour and stability of roadways and intersections with special reference to Australian underground coal mines. It aims to pinpoint the mechanism of interaction between the roof, pillar and floor and then to design the most effective support system for different conditions. The main numerical technique used for analysis of the roadways was the finite element method using newly developed techniques (Chapter 4). The data required for construction of the models was obtained from laboratory testing and field investigations carried out at Ellalong Colliery (Chapter 5). The research has two distinct parts and is presented in two separate chapters. Stability analysis of 2-D models of roadways are discussed in Chapter 6, and that for 3-D models of four-way (+) and three-way (T) intersections in Chapter 7.

This Chapter addresses the significance of high horizontal stress, post-failure behaviour of rocks and abutment pressures resulting from longwall faces on the stability of
roadways. The first part is a parametric analysis of a general model of a roadway which is subject to changing ratios of horizontal to vertical stress, $K$, in order to evaluate the behaviour of the roof, pillar and floor under various horizontal stresses. In the second part, site-specific models of main roadways are studied with consideration of the post-failure behaviour of rocks in the strata units. Lastly, in the third part, an attempt has been made to simulate conditions around gate entries experiencing abutment pressures resulting from the adjacent longwall face.

6.2 Effect of high horizontal stress on the stability of roadways

Horizontal stress values of several times the vertical stress have been recorded by many investigators around the world, particularly in Australia (Brown and Hoek 1978; Denham, Alexander and Worotnicki, 1979; Jeremic, 1981; Stephanson and Brown 1988). Also, field measurements at Ellalong Colliery indicated a high ratio of horizontal to vertical stress, $K = 3$ to $4$ (Chapter 5). Understanding the real behaviour of structures in a high horizontal stress field such as that in Australia is critical due to the fact that the high horizontal stress may cause serious stability problems. Several attempts have been made to determine the relationship between horizontal stress (magnitude and direction) and the mechanics of instability of roadways. Among them, Jeremic (1981) and Gale and Blackwood (1987) carried out independent research in Western Canada and Australia, respectively. The results of these investigations were discussed in Chapter 2.

6.2.1 General description of 2-D model of roadways

Based on the comprehensive information obtained from laboratory testing and field investigations, a series of two-dimensional models of roadways were constructed and analysed under four different loading conditions; $\sigma_v = 10$ MPa and $K = 1, 2, 3$ and $4$ (the ratio of $\sigma_H$ to $\sigma_V$). Therefore, if the component of the horizontal stress parallel to the longitudinal axis of the roadway is ignored, the in-situ condition of the structure will match one of these conditions. Also, evaluating and comparing the results of various loading conditions provides a deeper insight into the behaviour of roadways driven in various directions with respect to the horizontal stress field. This will provide guidelines for designing the mine layout.

The 3-D finite element code NASTRAN and the program SAFETY (developed as part of this research) were used to simulate and analyse features of the structure and to determine the failed zone in the surrounding rocks. Assuming symmetrical conditions around the roadway, only half of the domain was taken for modelling. The division of the
structure into elements was done in such a way that elements were smallest near the opening where details of the stress concentration and deformation pattern were required. Applying load by forces at grid points on the boundary of the model was the main technique of loading. Models were analysed for stresses and displacements within and around the structure, the results are presented in the following Sections.

6.2.2 Interpretation of results relating to high horizontal stress models

In the analysis, the interaction of all constitutive elements (roof, pillar and floor) were taken into account in a unique sequential solution procedure. For the sake of simplicity and convenience, the behaviour of the roof, pillar and floor are studied separately by grouping the results into three sections. The results in each section include stress and strain of elements and/or force and displacement of grid points. Only figures relating to case $K = 3$ are presented in the thesis. The strata column considered in this study and locations from where the results were obtained are shown in Figures 6.1 and 6.2, respectively.

![Figure 6.1 Strata units considered in general 2-D FE models.](image-url)
Figure 6.2 2-D model of roadway and locations from where the results were obtained.
(a) Results relating to the roof

The general pattern of vertical stress distribution around the roadway under various K values are shown in Figure 6.3 (numbers on the contour lines show the ratio of induced to virgin stress). It can be seen that when K increases, the destressed zone in the roof gets closer to the roof line while the disturbed zone in the pillar extends further into the rib-side. In addition, there is a vertical tensile zone on the rib-side when K > 2. It is also clearly shown that when the horizontal stress is significant, there is a particular high stress concentration pattern over the rib-line.

![Figure 6.3 Vertical stress concentration patterns around the roadway under various loading conditions.](image)

To check for the possibility of strata sliding over each other, it would be essential to have a very precise picture of shear and normal stress patterns on the bedding planes. Figures 6.4 and 6.5 give shear and normal stresses induced on the bedding planes located at different depths into the roof for K = 3.

![Figure 6.4 Shear stress at different levels above the roof line (K= 3).](image)
Comparison of the results from all loading conditions leads to the following conclusions summarised in Figure 6.6:

(i) The maximum shear stress values decrease when the bedding plane is located farther away from the roof line in all load cases.
(ii) The value of shear and normal stress on the bedding planes decreases when $K$ increases.
(iii) Location of the maximum shear stress for all bedding planes in all load cases is close to the rib line, ie about 0.5 m from the rib-side.
(iv) Location of the minimum normal stress is always at the mid-span of the roof line.
The distribution of shear stress along vertical lines at different locations in the roof was also studied in order to estimate an appropriate bolt length. Results for \( K = 3 \) are given in Figure 6.7. The general pattern of shear stress distribution along vertical lines for other loading conditions was almost the same, with important points being summarised as follows:

(i) There is almost no shear stress along the line REV1. Therefore, it would be pointless to install a fully grouted bolt at this location to withstand the shear stress induced on the bedding planes.

(ii) The maximum shear stress on the line REV2 occurs at locations 0.8 m to 3.5 m from the collar of the bolt; and at 0.5 m and 3.5 m for the line REV3. Fully grouted bolts with a length of less than, 3.0 m will not then provide the maximum resistance under these conditions. This conclusion was supported by field investigations (Chapter 5).

(iii) Shear stress along the line REV4 suggests that a bolt installed at this location would experience high shear stress over its full length.

(iv) The general conclusion is that a constant and even pattern of the bolting does not meet the optimum criteria for resistance to shear.

(v) To determine the most effective location and length of bolts, the shear safety factor on the bedding planes at various levels above the roof-line must be calculated based on the shear and normal stresses induced on and the properties of bedding planes (This will be discussed later in this Section).

Separation of the strata is another critical parameter which must be taken into account during the design of a support system. To study this parameter, the displacement along
various horizontal and vertical lines in the roof was analysed. Figure 6.8 shows the vertical displacement along horizontal lines located at different depth into the roof for $K = 3$. Figure 6.9 compares the vertical displacement along the vertical line at the mid-span of the roof under various loading conditions.

Figure 6.8 Vertical displacement along horizontal lines in the roof ($K = 3$).

Figure 6.9 Vertical displacement along the centre line of the roadway under various loading conditions.
Comparing the results relating to vertical displacement in the roof, the following conclusions can be made;

(i) Vertical displacement in the roof changes rapidly over the rib line. This causes a high tensile stress concentration at the top corners of the roadway which in turn leads to guttering. This mechanism is more pronounced when the horizontal stress is greater than the vertical stress, $K > 1$. In fact, guttering will occur if a roadway is driven at an angle other than parallel to the major horizontal stress. The severity of the guttering is a function of the drivage direction with respect to the direction and magnitude of $\sigma_{H1}$. The top of this guttering zone will be limited by a stratum which has either high tensile strength or is far enough from the roof line, therefore, eliminating large variations in its vertical displacement over the rib line. Based on the field investigations at Ellalong, this zone was limited to the line RGH2; 2.5 m above the roof line.

(ii) In the case of high horizontal stress, $K > 2$, bed separation is less likely to occur. In other words, sag of the roof in a roadway parallel to the major horizontal stress will be more pronounced than that in the roadway perpendicular to the major horizontal stress.

(iii) Maximum strata separation may occur at the mid-span of the roadway, therefore, theoretically, the best place for installing bolts to limit bed separation is at this location.

(iv) Any bolting system which has to prevent roof movement towards the roadway, should be designed in such a way that bolts are anchored in competent strata where vertical displacement is a minimum.

(b) Results relating to the pillar

Coal seam properties have a significant effect on the total response of the whole structure. In finite element modelling all variables relating to the mechanical properties of the coal seam are taken into account. After executing the model, vertical stress on the horizontal planes at different elevations in the pillar was chosen for studying the pillar reaction under different loading conditions. Figure 6.10 illustrates the vertical stress pattern induced within the pillar body for $K = 3$. 

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Comparing the results for all loading conditions, the following conclusions can be made:

(i) The maximum vertical stress concentration is located at the toe of the pillar. Its value is about two to three times $\sigma_v$; this value was the same for all loading conditions.

(ii) The maximum vertical stress concentration at the top front of the pillar was the same as that at the toe when $K = 1$, but when the ratio $K$ increases, the value of this stress concentration decreases (about 1.5 $\sigma_v$ for $K = 4$).

(iii) There is a destressed zone with respect to the vertical stress at the mid-height of the pillar. The zone extends to about 1.0 m into the pillar. The solid core of the pillar starts at a distance of 1.0 m from the rib-side where the maximum vertical stress is about 1.2 times $\sigma_v$.

(iv) When horizontal stress is greater than the vertical stress, there will be a vertical tensile zone moving 0.5 m into the pillar. Behind that, stress rapidly builds up to a high compressive zone.

In order to study the deformation of the pillar more comprehensively, the horizontal displacement occurring in the pillar under various conditions was studied. Figure 6.11 shows the maximum horizontal displacement at the mid-height of the pillar under various loading conditions. In addition, Figure 6.12 illustrates the difference between maximum horizontal displacement of the pillar and the roof at various distances from the rib-side.
The following conclusions can be made based on the displacement results in the pillar:

(i) The maximum horizontal displacement occurs at the mid-height of the pillar. The magnitude of the displacement depends on the distance from the rib-side and the value of K. The horizontal displacement in the pillar increases with K increasing.

(ii) The difference between the displacement in the coal seam and roof or floor stratum are more pronounced at locations closer to the rib-side. When K increases the difference is more significant. This mechanism may cause more shear concentration over the rib-
line. It must, however, be noted that the properties of rocks in the roof and floor as well as coal seam have significant effect on this behaviour.

(c) Results relating to the floor

Although the floor is not as important as the roof and ribs from the viewpoint of stability, on occasions floor heave causes serious problems for the mining operation. Besides, the instability of floor may indirectly affect the stability of the whole structure. In the course of this research, the behaviour of the floor was studied by plotting the vertical displacement of the floor at different distances from the floor line. Results for $K = 3$ are presented in Figure 6.13.

Comparing the results, it can be concluded that maximum floor heave occurs at the centre-line and its value decreases when $K$ increases. The vertical displacement in the floor also has a very rapid change under the rib line. This mechanism causes toe failure in the pillar. When $K$ increases the situation gets worse.

(d) Failure zone around the roadway

Based on the information on the failure parameters of rocks in the strata units (Chapter 5), the yield zone around the roadway has been computed by using the SAFETY program and a procedure designed through the course of this research. The results are given in Figure 6.14. The general conclusions are that; the ratio of horizontal to vertical stress, $K$, has a great effect on the stability of the structure. When $K \leq 1$, failure occurs
within the pillars (ribs) and not in the roof or floor, but as $K$ increases the failure zone extends towards the roof and floor and is less apparent in the pillars. When $K = 4$, the second layer in the floor, the shale stratum, fails intensively. It is believed that this significantly affects the stability of the whole structure.

Figure 6.14 Failed zone around the roadway under various loading conditions (Gray parts show tensile failure in the pillar, spalling).
6.2.3 Guide-lines for the design of support system in roadways

The requirements of roof bolts and dowels in underground coal mines which have bedded strata around the openings are mainly:

(a) to prevent shearing along the bedding planes.
(b) to bind individual strata into laminated beams.
(c) to hold in place mesh and straps and to suspend failed strata from overlying competent strata.

Various approaches have been developed for designing the roof bolt pattern in different conditions. A brief review of those was presented in Chapter 2. The strategy in this research is to determine the structural failure mechanism first, and then design the appropriate roof bolting pattern in accordance to the particular condition.

Design of the bolting system includes determination of two major parameters: location and length of bolts. To achieve an optimum design, three structural failure criteria should be taken into account; arch failure in the roof, shearing of the bedding planes and guttering over the edges of the roadway. The maximum height of the yield zone over the roadway can be estimated from figures such as Figure 6.14 according to the K value. The shear safety factor along the bedding planes is calculated from Equation 6.1. The shear safety factor determines the most probable location where sliding of the bedding plane may occur.

$$S.F = \frac{\mu \cdot N}{\tau} \quad (6.1)$$

Where:

$S.F =$ shear stress safety factor
$N =$ normal stress on the bedding plane (MPa)
$\tau =$ shear stress on the bedding plane (MPa)
$\mu =$ friction coefficient of the bedding plane,

$(\mu=0.6 \text{ based on the available data at Ellalong})$

For the purpose of this investigation, the shear safety factor was calculated along the bedding planes based on the data in the pair of graphs (shear and normal stress patterns for each loading condition); the results for $K = 3$ are presented in Figure 6.15. According to these computations, the critical location where sliding may occur is between 0.8 m and
1.8 m from the centre line of the roadway. In addition, bolts length should also be checked against shear distribution along vertical lines in the roof as mentioned previously.

![Figure 6.15 Shear safety factor along the bedding planes at different levels (K= 3).](image)

6.2.4 General conclusions relating to the effect of high horizontal stress

Achievements of the investigation relating to the high horizontal stress can be summarised as follows;

(a) when K > 1, the destressed zone extends further into the roof and over the pillar, and a tensile zone appears in the immediate roof above the opening. This condition results in a higher arch failure in the roof, and demands a heavier support system (longer and more bolts, particularly around the mid-span of the roadway).

(b) the shear and normal stresses along the bedding planes decreases with an increase of K. Location of the maximum shear stress for all bedding planes in all loading conditions is closer to the rib-line than to the centre-line.

(c) the vertical distribution of shear stress in the roof is almost the same for all conditions of loading. However, the maximum shear stress is located on the line close to the rib-line resulting in a high possibility of guttering at this location.

(d) vertical displacement in the roof changes rapidly over the rib-line resulting in a high tensile stress at the top corners of the roadway. This mechanism is more pronounced when K > 1.

(e) guttering will occur if a roadway is driven at an angle other than parallel to the major horizontal stress when K > 1. The severity of guttering is a function of the drivage
direction, strength of rocks and width of the roadway. Inclined roof bolts close to the rib-lines should be designed in such a way that bolts are anchored in the strata over the ribs where vertical displacement is minimum.

(f) the maximum stress concentration at the top and toe of the pillar decreases when K increases.

(g) there is a destressed zone at the mid-height of the pillar, about 1.0 m thick. When K > 2, a vertical tensile zone will appear in the confining part of the pillar causing severe spalling.

(h) when K increases, the maximum floor heave and roof sag decreases. Therefore, roadways parallel to the major horizontal stress will have more floor heave and roof sag.

(i) when K ≤ 1, failure occurs within the ribs (pillars) and not to any great extent in the roof or floor. But as K increases, the failure zone tends towards the roof and floor rather than in the pillars.

(j) to design an optimum support system in a high horizontal stress field, three potential modes of failure should be taken into account; arch failure in the roof, sliding along the bedding planes and guttering over the edges of the roadway.

The design of a particular support system for the site-specific roadways (main roadways and gate roadways) at Ellalong will be presented in the next Sections of this Chapter.

6.3 Stability analysis of main roadways (effect of post-failure behaviour of rocks)

One of the limitations of most FE codes is that they can not simulate the strain softening behaviour of rocks after the peak strength. Analysis continues until the peak strength of rock is reached, as most of the programs can not consider the negative slope of the stress-strain curve. They consider a perfectly plastic behaviour for materials which means that the strength of the material remains at the peak value while the strain increases. This assumption implies a higher strength value for the rock leading to an overestimate of rock strength. However, in recent years, there have been many attempts to solve this problem (Kawahara et al 1981; Minh et al 1981; Kripakov et al 1988; Chen and Karmis 1988; Park and Gall 1989; Park 1992; Fama 1993a,b; and Hematian and Porter 1993a).

This part of the present research describes the application of a new technique for consideration of the post-failure behaviour of rocks during FEA of main roadways (Chapter 4). The significance of the post-failure behaviour of rocks on the stability of roadways are addressed later in this Section.
6.3.1 Site specific models for assessing stability of main roadways

The general geometry and mesh pattern of the roadway, strata column and stress field are compiled in Figure 6.16. The properties of rocks in the strata units and the stress-strain curves used in the non-linear and post-failure solutions are based on the results presented in Chapter 5.

Figure 6.16 Geometry and mesh pattern of the roadways, strata column and stress field considered in site specific models (PID# are property identification numbers).
6.3.2 Interpretation of results relating to main roadways

Models were analysed using three different solution techniques: linear-elastic, non-linear elastic-plastic and elastic-plastic with consideration of the post-failure behaviour of failed rocks. Figures 6.17 to 6.19 compare the vertical, horizontal and shear stress patterns around the roadway resulting from the different solution methods. Numbers in Figures 6.17 and 6.18 are the stress concentration rate (the ratio of induced to virgin stresses) and those in Figure 6.19 are induced shear stress values in MPa. It can be seen that there was not much difference between the results obtained from linear and non-linear models for locations close to the opening. But when the post-failure behaviour of rocks is taken into account (the softening model) there was considerable difference between the results. The destressed zone around the opening gradually extends into the surrounding strata in the softening model.

Figure 6.17 Vertical stress concentration for the linear, non-linear and softening models of main roadways (numbers on the contour lines are the ratio of induced to virgin stresses and dimensions along sides of the model are in metres).
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Figure 6.18 Horizontal stress concentration for the linear, non-linear and softening models of main roadways (numbers on the contour lines are the ratio of induced to virgin stresses and dimensions along sides of the model are in metres).

Figure 6.19 Shear stress distribution for the linear, non-linear and softening models of main roadways (numbers on the contour lines are induced shear stress values in MPa, and dimensions along sides of the model are in metres).
The highest level of vertical stress on the pillar is between 1.3 and 1.4 times the virgin stress, and its location moves into the pillar when the post-failure behaviour of coal is considered. In this situation, the confining part of the pillar around the core starts to loose its integrity resulting in a considerable reduction in the overall strength of the pillar. This phenomenon has been well shown in Figure 6.20 which illustrates the progressive yield zone around the roadway. To govern the general stability of the pillar, it is essential to use some sort of support system to keep the confining part of the pillar intact. This was practiced in the mine by utilising polymer grid structures in conjunction with composite plastic bolts.

To determine the yield zone around the roadway, the model was analysed for principal stresses. The results were then examined against the failure criteria described in Chapter 5. Figure 6.20 illustrates the yield zone around the main roadway resulting from different solution techniques. It can be seen that when the strain softening of rocks was taken into account, the yield zone had expanded further into the surrounding strata, particularly in the floor and coal seam. The yield zone in the immediate floor is the result of the high tensile zone induced in the floor. However, this condition was restricted to the specific site of this investigation where the floor was not very strong. The general condition of the floor in the mine was quite stable.

Figure 6.21 compares the vertical displacement along the centre-line in the roof resulting from the different solutions with that from field measurement. It is clearly shown that the linear elastic model gives unrealistic results with respect to the displacement of the roof. The maximum sag in the mid-span of the roof is 13 mm in the linear elastic model while the measured value is about 40 mm. On the other hand, results from the softening model are very close to the measured values. It is also shown that the maximum displacement in the roof occurs within the top coal and the next stratum.
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Figure 6.20 Failure zone around main roadways resulting from different models.

Figure 6.21 Vertical displacement along the centre-line of the roof for main roadways, 1) linear, 2) non-linear and 3) softening model.
In addition, the shear safety factor on the 1st and 2nd bedding planes located at 1 m and 3.5 m from the roof line were calculated using Equation 6.1. Results of this study are presented in Figures 6.22 and 6.23.

![Figure 6.22 Shear safety factor along the 1st bedding plane for main roadways.](image)

![Figure 6.23 Shear safety factor along the 2nd bedding plane for main roadways.](image)

Both the linear and non-linear models suggest that sliding is most likely to occur on the first bedding plane and the second one remains quite stable. On the contrary, the softening model demonstrates that the immediate roof has failed under the new stress distribution and is not stiff enough to build up high shear stress on the contact surface. Moreover, since the failed roof is suspended from the upper stratum, it imposes a uniform distributed load on the upper stratum resulting in a high shear stress on the
second bedding plane. In the case where the second bedding plane is left to fail, the boundary of any roof fall would extend further into the roof. The roof falls surveyed in the field supported the theory concluded from the softening model (Chapter 5).

6.3.3 Design of reinforcement system for main roadways

To design the optimum support system, three potential modes of failure were taken into account, as shear failure on the bedding planes, arch failure over the roof-line and guttering over the rib-lines. For this purpose, the shear safety factor was calculated for the first and second bedding planes (Figures 6.22 and 6.23), the height of the yield zone was estimated from Figure 6.20 and the shear stress concentration over the edges of the roadway was taken from Figure 6.19.

In summary, the bolting system has to be designed to meet the following requirements:

(a) to carry the weight of the immediate roof that is to be suspended from the upper strata. The thickness of the immediate roof, for this specific site and under normal loading conditions (without any abutment pressures from the longwall face), is 1 m to 1.5 m.

(b) to withstand the shear stress induced on the bedding plane located between the sandstone/shale and sandstone/mudstone. The value and location of the critical shear stress could be estimated from Figure 6.23.

(c) to overcome the high shear stress concentration over the corners of the roadway as shown in Figure 6.19.

The following is the simplified calculation for designing the roof bolt pattern based on the above requirements; the pattern of which is shown in Figure 6.24.

\[
\begin{align*}
&h, \text{ load height} = 1.5 \text{ m} \\
&\gamma_1, \text{ coal density} = 1.5 \text{ tonne/m}^3 \\
&\gamma_2, \text{ sandstone/shale density} = 2.5 \text{ tonne/m}^3 \\
&S, \text{ bolts spacing} = 1.0 \text{ m} \\
&L, \text{ bolt length} = 2.0 \text{ m} \\
&D, \text{ bolt diameter} = 20 \text{ mm} \\
&P_y, \text{ bolt yield strength} = 6.5 \text{ tonne} \\
&P_u, \text{ bolt ultimate strength} = 10.6 \text{ tonne} \\
&R = \text{ tensile load induced in each bolt (tonne)} \\
&sf = \text{ factor of safety}
\end{align*}
\]
R = S^2 \times (h_1 \times \gamma_1 + h_2 \times \gamma_2)
R = 1 \times (1.0 \times 1.5 + 0.5 \times 2.5)
R = 2.75 \text{ tonne}

\[
sf = \frac{P_y}{R} \Rightarrow sf = \frac{6.5}{2.75} \Rightarrow sf = 2.36
\]

In addition, two cable bolts per row are needed to overcome the potential shear failure on the second bedding plane plus two inclined bolts crossing over the edges of the rib sides to prevent initiation of the break off line at these locations as shown in Figure 6.24.

![Figure 6.24 Pattern of the reinforcement system designed for main roadways.](image-url)
6.4 Stability analysis of gate roadways (effect of abutment pressures)

It is thoroughly proved that a longwall face induces a particular stress distribution within and around the panel. The immediate roof starts to detach from the overlying strata and sag, resulting in a destressed zone over the working area. As the face moves away from its starting position, the dimension of the overhanging roof will increase. In this condition, the overburden pressure is transferred to the solid coal in front of the face and to the chain pillars in the gate entries. Eventually, after a critical length of advancement the immediate roof begins to cave. In good caving conditions, the caved material will expand sufficiently to completely fill the void space behind the face. As the face continues to advance, the stress distribution in the goaf gradually builds up to the virgin conditions, but never exceeding, after a prolonged period. The stability of gate roadways and the strategy for designing the support system in them are based on the new stress distribution and the direction of longwall mining.

In advance longwall mining, pump packing, big-bag chocks and wood or concrete cribs are used along the cave edge to impose a breaking off line, to provide support for the gate roadways and to maintain the competence of the immediate roof over the roadway. In retreat longwall mining which all Australian underground longwalls are practicing, there is no need to fully support roadways behind the face on the goaf edge. At Ellalong Colliery, concrete cribs were used at cut throughs in the gate entries to create a break off line in order to stop caving encroaching on the adjacent roadway.

To study the stability of the gate roadways ahead and behind the longwall face, three models were constructed and analysed. These models simulated existing conditions on the A-A, B-B and C-C sections as shown in Figure 6.25.
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Figure 6.25 A typical retreat longwall panel and various sections used for stability analysis of gate roadways.
The height of the caved zone and also the mechanical properties of the caved material in the goaf area play significant roles in the induced stress pattern around the longwall panel. The height of the caved zone was calculated as 8.5 m assuming a bulking factor of 1.35; based on the lithology and field observations. The stress strain characteristic of caved material is assumed as shown in Figure 6.26. It is considered that the ability of the caved material in the goaf area to support the uncaved strata increases with its compaction. However, the maximum stress on the caved material would never be greater than the virgin stress. In order to model this condition, the stress-strain curve has three distinct parts; the first part simulates the increasing load bearing capacity as the material is being compacted. The second part is representative of the compacted material which acts more elastically. The third part shows the maximum load bearing capacity of the material equal to the virgin stress, i.e., the material must be modelled as plastic otherwise the stress will increase beyond the virgin value at this point.

![Figure 6.26 Stress strain curve of the caved material.](image)

An attempt has also been made to simulate powered supports in the face by using solid elements. A modulus of elasticity of 12.5 MPa and a Poisson's ratio of 0.3 were assigned to the row of elements adjacent to the longwall face. A maximum load of 400 tonne was considered for the powered supports. It must be mentioned that the width of roadways in the gate entry was taken as 5.0 m because some spalling was observed on the pillars and rib sides in the field. This assumption will result in a margin of safety in the results.

### 6.4.1 Interpretation of results relating to the centre line of the face

In order to estimate the abutment pressures ahead and behind the face, the model was analysed for stress and displacement across the A-A section. Two different solution methods, linear and non-linear with consideration of the strain softening, were used.
Vertical stress concentration contour lines (the ratio of induced to virgin stress) on the A-A section and vertical stress profile on the roof line of the longwall are given in Figures 6.27 and 6.28, respectively. It was deduced that a reduced elastic modulus in the softening model resulted in a different stress pattern around and ahead of the face.

Figure 6.27 Vertical stress concentration around the longwall face for the A-A section.

Figure 6.28 Vertical stress profile on the roof line of the longwall face for the A-A section.
In the softening model, the stress above the longwall face reduced while the front abutment pressure increased. An overall high stress redistribution took place away from the longwall face towards the direction of the unmined panel as shown in Figure 6.28. When the same solution technique was used for B-B and C-C sections, it was also shown that stresses at the pillar corners were significantly reduced, especially in parts next to the goaf area in section C-C. Stresses were transferred from the pillar corners to the pillars core. All of this was due to the increased deformation induced in the softening part of the coal seam. According to the results obtained from A-A section, the maximum front abutment pressure was about 25 MPa located 2.0 m ahead of the face. These stress values were used to simulate the loading conditions for the B-B section model.

6.4.2 Interpretation of results relating to gate roadways ahead of the face

The B-B section was analysed under the new stress conditions resulting from stress analysis of the A-A section. The safety factor contour lines were calculated based on the stability analysis of the gate roadway ahead of the face, the B-B section. Results are illustrated in Figure 6.29. This shows that variations of the mechanical properties of the strata units and the bedding planes have significant effect on the stress distribution and, consequently on the induced yield zone around the roadway. This also indicates that the maximum disturbed zone within the pillar is less than 10.0 m from the rib-side.

Figure 6.29 Safety factor contour lines around the gate roadway ahead of the face.
In this particular condition, the top coal was totally failed and the height of the failure zone extended half way up to the second stratum in the roof, 3.5 m above the roof line. This prediction is very close to that found from roof fall surveys carried out in the mine and that calculated using empirical equations based on rock mass classification presented in Chapter 2. The RMR rating for the immediate roof is about 30 (Table 5.14) and the width of the roadway is 5 m; therefore, using Equation 6.2 (Unal 1986) will result in a load height of 3.5 m as follows (equal to the value obtained from FEA):

$$h = \frac{100 - \text{RMR}}{100} \times B$$  \hspace{1cm} (6.2)

Where:
- \(h\) = load height (m)
- \(\text{RMR}\) = rock mass rating (CSIR system)
- \(B\) = span of the roadway (m)

\[ h = \left(\frac{100 - 30}{100}\right) \times 5.0 \Rightarrow h = 3.5 \text{ m} \]

In addition, the shear safety factor on the bedding planes were calculated and are presented in Figure 6.30. It is perceived that the second bedding plane would be in a critical condition, and if an adequate support system is not implemented, roof failure would extend further into the roof.

![Figure 6.30 Shear safety factor along bedding planes for the gate roadway ahead of the face.](image-url)
6.4.3 Design of reinforcement system for gate roadways ahead of the face

Based on the results obtained from stability analysis of the gate roadways, two alternatives for the roof bolt pattern in the gate entry roadways ahead of the face are proposed:

**Pattern A:**

- $h$, load height = 3.5 m
- $\gamma_1$, coal density = 1.5 tonne/cum
- $\gamma_2$, sandstone/shale density = 2.5 tonne/cum
- $S$, bolts spacing = 1.0 m
- $L$, bolt length = 4.0 m
- $D$, bolt diameter = 24 mm
- $P_y$, bolt yield strength = 12.8 tonne
- $P_u$, bolt ultimate strength = 13.4 tonne

\[ R = S^2 \times (h \times \gamma_1 + h \times \gamma_2) \]
\[ R = 1 \times (1.0 \times 1.5 + 2.5 \times 2.5) \]
\[ R = 6.75 \text{ tonne} \]

\[ sf = \frac{P_y}{R} \Rightarrow sf = \frac{12.8}{6.75} \Rightarrow sf = 1.85 \]

Two inclined bolts crossing over the edges of the roadway should also be used to prevent initiation of the break off line at these locations.

**Pattern B:**

- $S$, bolts spacing = 1.0 m
- $L$, bolt length = 2.5 m
- $D$, bolt diameter = 20 mm
- $P_y$, bolt yield strength = 6.5 tonne
- $P_u$, bolt ultimate strength = 10.6 tonne
In addition, three cable bolts per row are needed to overcome the potential shear failure on the second bedding plane and also to suspend the laminated beam built by roof bolts to the upper stratum. Also two inclined bolts crossing over the edges of the roadway are needed to prevent initiation of the break off line at these locations. Figure 6.31 illustrates alternative B for the reinforcement system in gate roadways ahead of the face.

Figure 6.31 Pattern of the reinforcement system designed for gate roadways.
6.4.4 Interpretation of results relating to gate roadways behind the face

Figures 6.32 to 6.34 show the safety factor, vertical stress concentration and shear stress contour lines around the gate roadway behind the face along section C-C, respectively. According to these results, it can be concluded that the failure zone above the gate roadway behind the face would extend beyond the second bedding plane and be limited by the third stratum in the roof. This condition implies that the roof is cut along one side and acts as a cantilever beam over the roadway. The load on the support system at this location would be very high. In practice, two powered supports are located at the M/G not only to push the AFC but also to keep the face entrance open.

![Safety factor contour lines on the C-C section for the gate roadway behind the face.](image)

Looking at the stability of the pillars in the gate entry roadways behind the face, there is a high possibility of shear failure around the corners, and also spalling is most likely to occur on the surface of the pillar facing the longwall panel. However, the maximum vertical stress concentration will be about 1.4 times the virgin stress plus the additional load resulting from the cantilevered immediate roof. The maximum stress is located at a distance of 1.5 m from the pillar edge. The total disturbed zone in the pillar is less than 10.0 m wide. The results from section B-B also indicated that the disturbed zone in the other side of the pillar under the abutment pressure ahead of the longwall face was less than 10.0 m (as shown in Figure 6.29). Therefore, it can be concluded that the 30 m wide pillar between two roadways in the gate entries is an over-design and could be reduced.
to 25 m or 20 m. It should be mentioned that this analysis is from a stress balance point of view, however, other factors as minimum height / width ratio to stop uncontrolled failure in the pillar must be taken into account.

Figure 6.33 Vertical stress concentration contour lines on the C-C section for the gate roadway behind the face.

Figure 6.34 Shear stress contour lines on the C-C section for the gate roadway behind the face.
6.5 Conclusions

Part of the work outlined in this chapter included a comprehensive investigation into the behaviour of both general and site-specific roadways. Finite element method was used to analyse the stability of 2-D models of roadways. The input data required for the analyses were obtained from laboratory testing and field investigations that were presented in Chapter 5. During the study, new FE optimisation and modelling techniques which were developed during this research program and explained in Chapter 4 were employed to prepare the most realistic models.

The research suggested that three major factors which may significantly affect the stability of roadways are; high horizontal stress, post-failure behaviour of rocks and abutment pressure resulting from longwall faces. The behaviour and stability of main and gate roadways were analysed and appropriate reinforcement systems were then designed for different conditions. Based on the research results the following conclusions can be drawn:

(i) High horizontal stress, \( K > 1 \) (\( K \) is the ratio of horizontal to vertical stress), has significant effect on the behaviour of roadways; (1) As \( K \) increases the destressed zone in the roof extends further into the roof and over the pillar, and also a tensile zone appears in the immediate roof. This condition will eventually result in high arch failure over the opening. (2) There is a destressed zone at the mid-height of the pillar with a thickness of about 1.0 m, and when \( K > 1 \), a vertical tensile zone will be induced at the rib-side causing severe spalling. (3) Vertical displacement in the roof changes rapidly over the rib-line. This causes a high tensile stress concentration at the top corners of the roadways. This mechanism is more pronounced when \( K > 1 \). (4) Roadways parallel to the major horizontal stress exhibit increased roof sag and floor heave.

(ii) Numerical results clearly showed that: (1) Linear models do not present realistic behaviour of underground structures, particularly when rocks in the strata units are relatively weak. (2) On the other hand, results obtained from the softening model were closely matched with the measured values. (3) It was shown that a non-linear FE model which uses an iterative technique for modelling progressive failure is an accurate tool for the stability analysis of underground structures.

(iii) The research showed that abutment pressures resulting from longwall faces have appreciable effect on the stability of gate roadways, particularly at locations close to face
end. In this condition, an adequately designed support system is required to maintain the stability of gate roadways.

(iv) To design an optimum reinforcement system for underground roadways, three potential modes of failure; arch failure in the roof, shearing of the strata and guttering over the edges of the roadway must be taken into account.
CHAPTER SEVEN

STABILITY EVALUATION OF INTERSECTIONS
(3-D FINITE ELEMENT MODELS)
CHAPTER 7

STABILITY EVALUATION OF INTERSECTIONS
(THREE-DIMENSIONAL FINITE ELEMENT MODELS)

7.1 Introduction

Intersections are formed when pillars between two roadways are cut through to permit access for mining machinery, men, materials and ventilation purposes. Roadway intersections are grouped into two types: four-way (+) and three (T) intersections. These are particularly susceptible to ground control problems due to the inherent wide roof span, excessive stress concentrations and variable intersection shapes. Stresses induced during intersection development may result in a high incidence of roof and rib failures. Although it is acceptable to use 2-D finite element methods when analysing stability of roadways, 3-D methods must be used when analysing intersections. In addition, the stability analysis of intersections in a high horizontal stress field, such as that existing in Australian underground coal mines, is very complex in comparison to the situations where there is a low horizontal stress field.

Since major factors affecting the behaviour of intersections have significant interaction with each other and may vary from mine to mine, it is not possible to differentiate their individual effect using a limited number of case studies dealing with site-specific problems. Investigations carried out by Stahl 1961; Langland 1978; Peng and Okubo 1978 and Hanna et al 1986 in the USA were reviewed and presented in Chapter 2.

This final part of the present research is conducted to indicate the influence of important parameters on the behaviour and stability of intersections. The main technique used for the analysis was the finite element method using the data obtained from laboratory testing and field investigations (Chapter 5). Individual parameters such as vertical stress, the ratio of horizontal to vertical stress and the width of roadways were varied to investigate the significance of these parameters on the stress and displacement patterns induced around the intersection.

Although most intersections in gate entries are three-way, the stability analysis of four-way intersections, which are more common in the main entries, is more complex and critical than that of three-way intersections. Therefore, more investigations were carried out on the behaviour of four-way intersections. The results of this study are presented in
two distinct parts. The first part describes the parametric analysis of 3-D models of four-way intersections and also presents the results obtained from site-specific models of four-way intersections for Ellalong Colliery conditions. The second part describes the results obtained from an investigation of the general and site-specific three-way intersections for Appin Colliery.

### 7.2 Description of general 3-D models of four-way (+) intersections

In order to determine the general behaviour of intersections under various conditions, three series of 3-D models were constructed and analysed. Major parameters encountered in these models were vertical stress, the ratio of horizontal to vertical stress and the width of the openings. Assuming symmetric conditions around the intersection, only a quarter of the structure was taken for modelling. 8-node solid elements were used to build the models. The number of elements, grid points and computer running time for each series of models are given in Table 7.1. A general view of the intersection is illustrated in Figure 7.1 and the properties of rocks are tabulated in Table 7.2.

<table>
<thead>
<tr>
<th>Code</th>
<th>intersection dimensions (m)</th>
<th>number of grids</th>
<th>number of elements</th>
<th>computer running time</th>
</tr>
</thead>
<tbody>
<tr>
<td>TDS</td>
<td>4 x 4</td>
<td>8433</td>
<td>6064</td>
<td>4:14:06</td>
</tr>
<tr>
<td>TDM</td>
<td>5 x 5</td>
<td>8325</td>
<td>5995</td>
<td>4:01:27</td>
</tr>
<tr>
<td>TDL</td>
<td>6 x 6</td>
<td>8225</td>
<td>5932</td>
<td>3:57:17</td>
</tr>
</tbody>
</table>

*At least 400 mega bytes of memory were required to run each of these models.

*The height of roadways was 3.0 m

<table>
<thead>
<tr>
<th>Type of Rock</th>
<th>thickness (m)</th>
<th>E (GPa)</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>mudstone (floor)</td>
<td>10.0</td>
<td>16.0</td>
<td>0.22</td>
</tr>
<tr>
<td>coal</td>
<td>3.0</td>
<td>4.0</td>
<td>0.30</td>
</tr>
<tr>
<td>sandstone (roof)</td>
<td>12.0</td>
<td>18.0</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Eight loading conditions were taken into account in this research as summarised in Table 7.3. Loads were applied to the model by means of uniform pressure on the internal free faces. This technique, loading from the inside, helped to reduce the size of the model and also to eliminate boundary effects on the results. In all loading conditions, a linear
solution method was used to analyse the models. Results of this study included 200 figures; only those relating to the $6 \times 6$ intersection model, TDL, are presented in this Chapter. Many attempts have been made by means of regression analysis to draw general conclusions from the huge number of figures and results. The effect of depth (in the form of vertical stress), horizontal stress and width of roadways on the behaviour of pillar, roof and floor are discussed separately in the following Sections.

Figure 7.1 3-D model of the intersection in finite element analysis.
Table 7.3 Various loading conditions considered in 3-D models.

<table>
<thead>
<tr>
<th>Loading conditions</th>
<th>$\sigma_Z$ (MPa)</th>
<th>$\sigma_Y$ (MPa)</th>
<th>$\sigma_X$ (MPa)</th>
<th>$K_Y$ ($K_{\text{min}}$)</th>
<th>$K_X$ ($K_{\text{max}}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SUB1</td>
<td>5.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>SUB2</td>
<td>7.5</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>SUB3</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>SUB4</td>
<td>12.5</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>SUB5</td>
<td>10.0</td>
<td>10.0</td>
<td>10.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>SUB6</td>
<td>10.0</td>
<td>10.0</td>
<td>20.0</td>
<td>1.0</td>
<td>2.0</td>
</tr>
<tr>
<td>SUB7</td>
<td>10.0</td>
<td>10.0</td>
<td>30.0</td>
<td>1.0</td>
<td>3.0</td>
</tr>
<tr>
<td>SUB8</td>
<td>10.0</td>
<td>10.0</td>
<td>40.0</td>
<td>1.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

7.2.1 Effect of depth and roadway width on the behaviour of intersections

Effect of depth was studied by analysing models under four different vertical stresses; $\sigma_Z$= 5.0, 7.5, 10.0 and 12.5 MPa. These values are representative of various depths; 200, 300, 400 and 500 m, respectively, taking the average strata density of rocks 2.5 MPa/100 m. Only the results relating to vertical stress distribution in the roof and pillar and vertical displacement in the roof and floor for the TDL model are presented in this Section.

Vertical stress concentrations at the mid-height of the pillar were studied for various conditions by means of stress concentration contour lines, $r$, as expressed in Equation 7.1.

$$r = \frac{\sigma_i}{\sigma_Z}$$  \hspace{1cm} (7.1)

Where:

- $r$ = stress concentration ratio
- $\sigma_i$ = vertical induced stress (MPa)
- $\sigma_Z$ = vertical virgin stress (MPa)

Figure 7.2 shows the vertical stress concentration contour lines at the mid-height of the pillar for the TDL model. The general pattern for stress concentration at the mid-height of the pillar can be summarised in Figure 7.3, and following points are pertinent:
CHAPTER 7: Stability Evaluation of Intersections (3-D FE Models)

Figure 7.2 Stress concentration pattern at the mid-height of the pillar under various vertical stresses for the 6×6 m intersection, TDL model.

Figure 7.3 General pattern of the stress concentration at the mid-height of the pillar for the 6×6 m intersection, TDL model.
(i) The stress concentration pattern at the mid-height of the pillar is almost the same for various vertical loads. The general and simplified picture of the pattern is illustrated in Figure 7.3 and primary details are given in Table 7.4. It was shown that the maximum stress concentration value increased slightly with an increase in roadway size, but its location remained unchanged. The length of the disturbance zone, due to the intersection, over the individual roadways can be calculated from Equation 7.2.

Table 7.4 Primary details of the general pattern of stress concentration at the mid-height of the pillar.

<table>
<thead>
<tr>
<th>Code</th>
<th>L₁ (m)</th>
<th>L₂ (m)</th>
<th>L₃ (m)</th>
<th>L₄ (m)</th>
<th>L₅ (m)</th>
<th>rₙ (ratio)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TDS</td>
<td>2.78</td>
<td>0.77</td>
<td>4.6 - 5.6</td>
<td>4.8</td>
<td>2.2</td>
<td>2.0</td>
</tr>
<tr>
<td>TDM</td>
<td>2.55</td>
<td>0.91</td>
<td>4.6 - 6.0</td>
<td>6.0</td>
<td>3.1</td>
<td>2.1</td>
</tr>
<tr>
<td>TDL</td>
<td>3.00</td>
<td>1.03</td>
<td>4.6 - 6.0</td>
<td>7.2</td>
<td>4.1</td>
<td>2.2</td>
</tr>
</tbody>
</table>

\[
L₄ = 1.2 \, W₀ \tag{7.2}
\]

Where:

\[ L₄ \quad = \text{radius of influence of the intersection over the pillar (m), (Figure 7.3)} \]
\[ W₀ \quad = \text{width of the roadway (m)} \]

With respect to roof behaviour, the vertical stress concentration on a plane 0.5 m above the roof line is illustrated in Figure 7.4. Comparing the results for various cover loads, a generalised pattern was proposed (Figure 7.5) and the following conclusions were made:

(ii) The stress concentration pattern 0.5 m above the roof line was almost the same for various vertical stresses. The general pattern of stress concentration is depicted in Figure 7.5. Diagonal and longitudinal dimensions of the disturbed zone can be expressed in terms of roadway width as given in Equations 7.3 and 7.4, respectively. The highest stress concentration zone was located over the edge of the pillar and its maximum value varied between 1.4 and 1.6 times the vertical stress; about 20 percent more than that for individual roadways.
Figure 7.4 Stress concentration pattern on a plane 0.5 m above the roof line for various vertical stresses for the 6×6 m intersection, TDL model.

Figure 7.5 General stress concentration pattern on a plane 0.5 m above the roof line for the 6×6 m intersection, TDL model.
$L_3 = 3.6 \ W_0$ \hspace{1cm} (7.3) \\
$L_4 = 1.7 \ W_0$ \hspace{1cm} (7.4)

Where:
$L_3$ = diagonal radius of influence of the intersection over roadways (m)  \\
$L_4$ = longitudinal radius of influence of the intersection over roadways (m)  \\
$W_0$ = width of the roadway (m)

(iii) The 3-D stress distribution in the roof was studied by illustrating the destressed zone over the intersection and individual roadways as shown in Figure 7.6. The stress pattern on both sides of the intersection was the same. The height, $h$, and bottom dimension, $2d$, of the various stress concentration contours are illustrated in Figure 7.7. Comprehensive regression analysis was carried out to find out the relationship between factors governing the magnitude of $h$ and $d$ for various contour lines. Results of the study are summarised in Figure 7.8. In this Figure, $W_0$ is width of the roadway, $h_c$ and $2d_c$ are the height and bottom dimension of the destressed zone at the intersection, and $h_{x/y}$ and $2d_{x/y}$ are those for individual roadways.

![Figure 7.6 Stress concentration pattern in the vertical section along X and Y roadways.](image-url)
Figure 7.7 General shape of destressed contour lines: (a) over the intersection, (b) over the individual roadways.

Figure 7.8 Relationship between width of roadway, $W_o$, height, $h$, and bottom dimension, $2d$, of destressed zone and stress ratio, $r$, for various vertical stresses.
Vertical displacement of the roof was studied by plotting contour lines of the sag at the roof line as shown in Figure 7.9. The following conclusions were made;

(iv) Maximum sag of the roof occurs at the centre of the intersection. It was shown that the maximum value was highly influenced by the vertical stress value. Based on regression analysis, it is proposed that Equation 7.5 may be used to estimate the maximum value of sag at the centre of the intersection for various roadways widths.

\[
d_s = H_0 \times \left[ -0.15 + (0.77 + 0.26W_0) \frac{\sigma_z}{E_{\text{coal}}} \right]
\]  

(7.5)

Where:

- \(d_s\) = maximum sag at the centre of the intersection (mm)
- \(H_0\) = height of the seam (m)
- \(W_0\) = width of the roadway (m)
- \(\sigma_z\) = virgin vertical stress value (MPa)
- \(E_{\text{coal}}\) = elastic modulus of coal (GPa)
(v) Behaviour of the floor at the intersection was also studied by means of vertical displacement on the floor line. Plots of the floor heave contour lines for various conditions are shown in Figure 7.10. General conclusions made for the roof sag could be extended to the floor heave except the fact that values of heave are much less than the corresponding values for sag. The maximum value of floor heave can be estimated by using Equation 7.6. This Equation was derived from regression analysis of the maximum values of heave at the intersection for various roadways widths.

$$d_h = H_0 \times \left[ -0.85 + 0.17W_0 + 0.53 \frac{\sigma_z}{E_{\text{coal}}} \right]$$ \hspace{1cm} (7.6)

Where:

- $d_h$ = maximum heave at the centre of the intersection (mm)
- $H_0$ = height of the seam (m)
- $W_0$ = width of the roadway (m)
- $\sigma_z$ = vertical stress value (MPa)
- $E_{\text{coal}}$ = elastic modulus of coal (GPa)

Figure 7.10 Vertical displacement of the floor (heave) at the intersection for the 6×6 m intersection, TDL model.
7.2.2 Effect of horizontal stress on the behaviour of intersections

The effect of horizontal stress on the behaviour of four-way intersections was investigated by analysing three series of models under four different ratios of horizontal to vertical stress, $K$, as $\sigma_Z = 10.0 \text{ MPa}$ and $K = 1, 2, 3$ and $4$. The lower values of $K$ are very common in Australian underground coal mines and the higher range, $K = 3$ and $4$, is representative of the Ellalong Colliery condition. The behaviour of the pillar, roof and floor under different horizontal stress values have been studied as follows;

(i) Vertical stress concentration patterns at the mid-height of the pillar are presented in Figure 7.11. The following conclusions can be made by comparing these results;

(i-a) When $K > 1$, the vertical stress concentration pattern at the mid-height of the pillar is no longer symmetric. The variation of stress values is more pronounced along the Y-roadway which is perpendicular to the direction of maximum horizontal stress. There are some tensile zones along the rib-side when $K > 2$. The thickness of the tensile zone
increases when $K$ increases. This tensile zone on the surface of the rib may result in severe spalling. Hence, any rib reinforcement system such as horizontal dowels, must have a longer length in the Y-roadway than that in the X-roadway.

(i-b) The general pattern and primary details for stress concentration at the mid-height of the pillar are shown in Figure 7.12. The highest stress concentration zone is located close to the edge of the pillar and it extends along the X-side roadway as $K$ increases (Figure 7.11). In this condition the width of the disturbed zone along the Y-roadway is wider than that along the X-roadway. Also, it can be seen that the dimensions of the total disturbed zone over the pillar is influenced by both the $K$ value and the width of the roadway, $W_0$.

\begin{center}
\begin{tabular}{|c|c|c|}
\hline
$K=1$, & $L_1 = 5.5$, & $L_2 = 5.5$ \\
$K=2$, & $L_1 = 5.6$, & $L_2 = 5.8$ \\
$K=3$, & $L_1 = 5.7$, & $L_2 = 6.7$ \\
$K=4$, & $L_1 = 5.8$, & $L_2 = 7.5$ \\
\hline
$K=1$, & $L_1 = 6.0$, & $L_2 = 6.0$ \\
$K=2$, & $L_1 = 6.4$, & $L_2 = 6.6$ \\
$K=3$, & $L_1 = 6.6$, & $L_2 = 7.3$ \\
$K=4$, & $L_1 = 6.7$, & $L_2 = 7.7$ \\
\hline
$K=1$, & $L_1 = 6.0$, & $L_2 = 6.7$ \\
$K=2$, & $L_1 = 6.6$, & $L_2 = 6.9$ \\
$K=3$, & $L_1 = 6.7$, & $L_2 = 7.7$ \\
$K=4$, & $L_1 = 6.7$, & $L_2 = 8.3$ \\
\hline
\end{tabular}
\end{center}

Figure 7.12 Effect of intersection dimension and horizontal stress on the stress concentration pattern at the mid-height of the pillar ($K \geq 1$).

(ii) Vertical stress concentration patterns on a plane 0.5 m above the roof line are presented in Figure 7.13. The following conclusions can be made;

(ii-a) When $K = 1$, the stress contour line $0.1\sigma_z$ was located over the roadway span while the $1.1\sigma_z$ line was very close to the rib-line.
(ii-b) When $K > 1$, the stress pattern over one roadway was totally different from the pattern over the other. The $0.1\sigma_z$ contour line from both sides of the disturbed zone diminished over the Y-roadway while changes in the stress pattern over the X-roadway was not considerable. This condition implies the possibility of a larger failure zone over the Y-roadway, because there is a higher stress concentration contour close to the roof line.

In addition to the above analysis, the 3-D stress pattern within the roof, on the x-side and y-side of the intersection were studied by plotting the destressed zone over the intersection and individual roadways as shown in Figure 7.14 (small discrepancies between the results are due to the accuracy of the plotting package). An attempt was made to determine the correlation between horizontal stress and the height, $h$, and bottom dimension, $2d$, of destressed contour lines. After comprehensive regression analysis, the results for $K = 1, 2$ and $3$ have been summarised in Figures 7.15 to 7.17. In these figures, $W_0$ is the width of the roadway, $h_c$ and $d_c$ are the height and half of the bottom dimension of the destressed zone at intersection, $h_x$ and $d_x$ are the same parameters for the X-roadway and $h_y$ and $d_y$ are those for the Y-roadway.
Figure 7.14 Stress concentration pattern on the X and Y-sides of the roof for the 6×6 m intersection, TDL model.
Figure 7.15 Relationship between width of roadway, $W_0$, height, $h$, and bottom dimension, $2d$, of destressed zone, and stress ratio, $r$, for $K = 1$. 
Figure 7.16-a Relationship between width of roadway, $W_0$, height, $h$, and bottom dimension, $2d$, of destressed zone, and stress ratio, $r$, for $K = 2$.

Figure 7.16-b Relationship between width of roadway, $W_0$, height, $h$, and bottom dimension, $2d$, of destressed zone, and stress ratio, $r$, for $K = 2$. 
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Figure 7.17-a Relationship between width of roadway, $W_0$, height, $h$, and bottom dimension, $2d$, of destressed zone, and stress ratio, $r$, for $K = 3$.

Figure 7.17-b Relationship between width of roadway, $W_0$, height, $h$, and bottom dimension, $2d$, of destressed zone, and stress ratio, $r$, for $K = 3$. 
Comparing the results depicted in Figures 7.14 to 7.17, the following conclusions can be made:

(ii-c) When $K > 1$, the stress distribution pattern over one roadway is different from that over the other as shown in Figure 7.14. In this condition, the boundary of the roof fall in the Y-roadway is limited to stress contour lines greater than 0.1 $\sigma_z$. Also, the rate of change in the stress distribution across the roof line of the Y-roadway is much more than that over the X-side roadway. The height of the roof fall in each of the roadways should be evaluated using appropriate graphs given in Figures 7.15 to 7.17 (more discussion will follow in a subsequent section).

(ii-d) The radius of influence of the intersection over the individual roadways with respect to the stress distribution in the roof was taken as the distance where the stress contour line 0.5 $\sigma_z$ becomes flat; as shown in Figure 7.18. Tables 7.5 to 7.7 show the radius of influence over each of the roadways under various conditions. These values are in good agreement with that expressed previously in Equations 7.2 and 7.3.

![Figure 7.18 Radius of influence of the intersection over the individual roadways.](image)

![Table 7.5 Primary details of the stress contour line 0.5 $\sigma_z$ for the TDS model.](table)

<table>
<thead>
<tr>
<th>$K$</th>
<th>$h_{x/Y,0.5}$</th>
<th>$t_{x,0.5}$</th>
<th>$t_{y,0.5}$</th>
<th>$L_{x,0.5}$</th>
<th>$L_{y,0.5}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5.0</td>
<td>2.9</td>
<td>2.9</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>1</td>
<td>4.4</td>
<td>2.5</td>
<td>2.5</td>
<td>6.0</td>
<td>6.0</td>
</tr>
<tr>
<td>2</td>
<td>4.1</td>
<td>2.5</td>
<td>2.0</td>
<td>7.0</td>
<td>6.0</td>
</tr>
<tr>
<td>3</td>
<td>3.7</td>
<td>2.5</td>
<td>1.7</td>
<td>7.0</td>
<td>5.0</td>
</tr>
<tr>
<td>4</td>
<td>3.4</td>
<td>2.5</td>
<td>1.4</td>
<td>8.0</td>
<td>5.0</td>
</tr>
</tbody>
</table>
(iii) Vertical displacement of the roof was studied by plotting contour lines of the sag at the roof line as shown in Figure 7.19. The following conclusions can be made from the results;

(iii-a) When $K > 2$, the variation of roof sag across the Y-roadway is more pronounced than that across the X-roadway. A comparison of roof sag profiles on a section 8 m from the centre of the intersection is shown in Figure 7.20. It can be seen that there is a sharp change in the sag value over the rib-line in the Y-roadway. It is also shown that when $K > 2$, there is a tensile stress zone on the side wall of the pillar along the Y-roadway.

(iii-b) Sag changes are a maximum at the corner of the intersection and above the rib-line of the pillars. This situation is an indication of the maximum shear deformation which is the most probable reason for roof guttering at these locations. Field observations at Ellalong Colliery and reports from other researchers (Hanna et al 1986) strongly support this conclusion. Also, in Chapter 6 analysis of results from the 2-D models allowed the same conclusion to be drawn.
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Figure 7.19 Vertical displacement of the roof (sag) for the 6×6 m intersection, TDL model.

Figure 7.20 Sag profiles across the X- and Y- roadways on a section 8 m from the centre of the intersection for K > 2.
(iv) Behaviour of the floor at intersections where $K \geq 1$ were studied by means of plotting vertical displacement, heave, on the floor line as shown in Figure 7.21. It can be seen that floor heave is more pronounced in the X-roadway which is parallel to the major horizontal stress.

![Figure 7.21 Vertical displacement in the floor (heave) for the 6x6 m intersection, TDL model.](image)

7.2.3 Shape and dimension of roof falls at intersections

Based on the field observations at Ellalong Colliery, it was realised that roof falls at intersections occur within an arch zone. Generally, the top boundary of the zone which requires support has a dome shape and its base is located within the diagonal width of the intersection. In cases where the coal was not strong the base extended over the edges of the ribs. The height of roof falls were between 1.5 m and 7.5 m with a medium range between 2.0 m and 4.0 m.

Since the destressed zone over the opening is an indication of the tensile strain zone, this may be the reason that roof falls occur within the destressed zone. However, parameters
such as the width of the opening, virgin stress state and the strength of rocks are the major parameters which control the dimensions of the envelop of potential failure. There have been a few attempts to determine the dimensions of the arch zone over the intersections. It was proposed by Peng and Okubo (1978a) that this boundary is the stress contour line 0.1$\sigma_z$. This figure was proposed for the vertical stress state. Later, Hanna et al. (1985) suggested that although the stress contour line of 0.1$\sigma_z$ could be a criterion for estimation of the roof fall boundaries at intersections, this criterion was sometimes an underestimate of the actual values due to some variations in the width and stress state.

Based on the results of the present research, illustrated in Figures 7.15 to 7.17, it was possible to more accurately estimate the dimensions of the different stress contour lines for various conditions. Based on the regression analysis of numerous data, Equations 7.7 and 7.8 were proposed to estimate the height and bottom diameter of roof falls at intersections. These equations are in a good agreement with the results of the field observations at Ellalong Colliery and those from the above mentioned investigations.

$$h_{c0.1} = 0.4 W_0 - 0.15 K \quad (7.7)$$

$$d_{c0.1} = 0.7 W_0 - 0.1 K - 0.8 \quad (7.8)$$

Where:

- $h_{c0.1}$ = height of the roof fall at the centre of the intersection which is equal to the height of the stress contour line 0.1$\sigma_z$ (m)
- $d_{c0.1}$ = half of the average bottom diameter of the roof fall at the intersection (m)
- $W_0$ = width of the roadway (m)
- $K$ = the ratio of horizontal to vertical stress, $K < 2$

When $K > 2$, $h$ and $d$ for the contour line 0.1$\sigma_z$ decrease significantly. In this condition it is more likely that the arching zone within which the roof fall may occur at the intersection is defined by a higher contour line. If it is assumed that the base of the roof fall is limited within the diagonal span of the intersection, then the contour line 0.3 $\sigma_z$ will be the criterion for estimating the dimensions of the roof fall. Hence, it is suggested that the following equations be used for these conditions:
\[ h_{c0.3} = 0.9 W_o - 0.2 K - 0.7 \]  
\[ d_{c0.3} = 0.7 W_o - 0.4 \]

Where:

- \( h_{c0.3} \) = height of the roof fall at the centre of intersection which is equal to the height of the stress contour line 0.3 \( \sigma_z \), (m)
- \( d_{c0.3} \) = half of the average diameter of the roof fall bottom at the intersection (m)
- \( W_o \) = width of the roadway (m)
- \( K \) = ratio of the horizontal to vertical stress, \( K \geq 2 \)

It should be mentioned that although there are many different combinations of rock type in strata columns in the real situation, it was not practical to consider a variety of strata columns in the models. However, the strata column used in the general model was one of the most common ones appearing in coal mines. In addition, the linear property of rocks were taken into account to differentiate effects of other parameters rather than the property of rocks. It is also important to note that the computer running time and required memory for non-linear analysis of a certain model would sometimes be more than eight to ten times that for the same model using a linear solution. Hence, the study of so many models with 3-D features would not be possible with the limitation of the computer facilities. Of course, in the next Section of this Chapter a site-specific model including more features of the real situation is constructed and analysed to determine the most realistic behaviour of the intersections.

### 7.3 Stability analysis of a site-specific model of a four-way intersection

This part of the research describes the application of 3-D FE modelling for a site-specific condition. The main purpose of the study was to apply the new approach of support design into a four-way intersection at Ellalong Colliery. The site-specific model was constructed based on the geotechnical and site information which were comprehensively explained in Chapter 5. A 3-D view which shows the strata column and dimensions used in the model are illustrated in Figure 7.22. The model consisted of 6450 Brick elements and included 11308 grid points. Construction of the model took about 4 hours. Analysis was carried out in two stages. In the first stage the peak strength of the rocks were taken into account. In the second run, residual properties were considered for those elements that failed during the first stage. The computer running time was over 15 hours, and more than 700 mega bytes of memory were engaged during the analysis.
Results of stress and displacement throughout the model had almost the same pattern as those obtained from the 3-D general models. Some discrepancies arose due to the variations in the rock properties in the strata units and dimensions of the structure. In general, there was a significant correlation between the results from the site-specific and general models, and from field data; as will be discussed later in this section. Since a complete discussion was presented on the results of the general 3-D models, the results from the site-specific model were prepared and presented in a manner such that they can be used for designing an optimum support system. The following is the evaluation of major stability aspects of the intersection:

(i) The vertical stress concentration at the mid-height of the pillar is shown in Figure 7.23. In addition, stability of the pillar was checked against the spatial stress distribution in the model. Safety factor contour lines at various levels in the pillar are shown in Figure 7.24. It can be seen that there is a tensile zone along the X-side roadway resulting in a significant reduction in the pillar strength. However, the failure zone extends more into the pillar from the Y-side roadway. The most severe failure zone in the pillar is located at the corner of the intersection; around 1.20 m x 1.0 m.
CHAPTER 7: Stability Evaluation of Intersections (3-D FE Models)

Figure 7.23 Stress concentration at the mid-height of the pillar (site-specific model).

Figure 7.24 Safety factor contour lines in the pillar at various levels (site-specific model).
By comparing the failure zones at various levels in the pillar, it can be concluded that toe failure is more pronounced than top failure in the pillar. This mechanism was also observed in the 2-D site-specific model, and conformed with observed pillar failures in the mine. To maintain the integrity of the pillar, 2.0 m long dowels at 1.0 m spacing in conjunction with polymer grid structures were suggested at the intersection and extending 2.0 m into each of the roadways. However, shorter dowels, 1.2 m long, were sufficient in the rib-side of the individual roadways outside the zone of influence of the intersection. When the confining part of the pillar is kept stable, the centre part would safely carry the maximum stress concentration of 1.4 $\sigma_z$ as shown in Figure 7.24.

(ii) Stability of the roof, with respect to arch failure, was analysed by calculating the safety factor contour lines in the roof as shown in Figure 7.25.

![Figure 7.25 Safety factor contour lines in the roof at various levels (site-specific model).](image-url)
From the above it can be seen that the bottom dimension of the roof fall is limited to the width of roadway and the diagonal span of the intersection. The height of the failure zone is about 1.25 to 2.0 m, if guttering over the edges and sliding on the bedding planes do not occur. The predicted failure height using Equations 7.7 and 7.9 is between 1.4 m and 2.8 m and the values measured in the field were between 1.5 m and 3.0 m.

(iii) Shearing of the strata on the bedding planes was examined by calculating the shear safety factor on the 1st and 2nd bedding planes above the roof line. Figure 7.26 shows shear safety factor contour lines on the bedding planes located 1.0 m and 3.5 m above the roof line. The results shown in Figure 7.26 are in agreement with those obtained from the 2-D site specific model. It can be seen that the second bedding plane failed in shear mode. The most critical locations due to sliding are along the edges of the individual roadways and at the corner of the intersection. Therefore, fully grouted bolts with 4.0 m length should be installed at these locations.

(iv) To examine the possibility of guttering, the shear stress contour lines on the vertical sections along the edges of the X and Y-roadways were studied. The results are plotted in Figure 7.27. The maximum shear stress value was between 6 MPa and 7 MPa and was located over the rib-lines. The critical location where the maximum shear stress occurred was above the corner of the pillar up to 1.0 m into the roof at the intersection. The maximum shear stress zone along the X-roadway was greater than that along the Y-roadway. This suggests that guttering is more likely to occur in the X-roadway.
(v) The structural behaviour of the floor at the intersection was also studied by calculating the safety factor contour lines in the floor. Results of the spatial stress distribution were examined against the strength parameters of the rock in the floor. Figure 7.28 illustrates the safety factor contour lines in the floor. It is shown that the immediate floor, mudstone stratum, failed below the floor line at the intersection. This condition also occurred in the 2-D roadway models.
Moreover, the vertical displacement in the roof and in the floor were calculated based on the FE analysis. Figure 7.29 shows the sag and heave contour lines for the site-specific model. The predicted maximum values of sag and heave using Equations 7.5 and 7.6 are 21 mm and 6 mm. These values are in good agreement with the results shown in Figure 7.29.

Figure 7.29 Vertical displacement (mm) at the roof line, sag, and at the floor line, heave, for the site-specific model.

7.4 Design of reinforcement system at four-way intersections

As described in Chapter 6, there are three potential failure mode (structural failure criteria) which must be tested for the design of an efficient and safe support system for a given roadway condition. Those criteria must be practiced with more attention for intersections as follows:

(a) The first criterion tests for a dome failure zone over the intersection. In this case, the height, \( h_c \), and bottom dimension, \( 2d_c \), of the zone can be calculated from appropriate destressed contour lines over the structure taking into account the major parameters, \( W_0 \) (the width of the roadway) and stress state as explained in previous section.

(b) The second criterion tests for sliding of the bedding planes. This happens when the horizontal shear stress exceeds the frictional strength of the bedding plane. To check for this, the best technique is to prepare shear safety factor contour lines on the bedding planes.

(c) The third criterion tests for guttering of the roof at the corners of the intersection, particularly when the ratio of horizontal to vertical stress is equal or greater than two
(K ≥ 2). For this criterion, changes of the vertical displacement over the rib-line or vertical shear stress contours in the roof must be examined against the tensile or direct shear strength of the rocks in the strata units.

Since the pattern of support would be different for each of the above criteria, it is crucial to identify the dominant criterion. In the first case where a dome roof fall over the structure is predicted, longer and greater number of bolts must be installed at the centre of the span. In the second case, more bolts need to be located near the side walls where the maximum shear stress occurs on the bedding planes. In the last case, inclined long bolts should be installed over the abutments to prevents guttering. By comparing the results of the structural failure criteria with the in-situ properties of the strata and bedding planes surrounding the structure, a reliable and efficient support system can be designed.

Particular attention should be paid to the fact that the roof bolting pattern at and close to an intersection should be varied depending on the structural features existing at this location. The state of stress, width of roadways and properties of rocks have appreciable effect on the behaviour of the structure. In a high horizontal stress field, guttering is more pronounced than the dome shape failure, whilst in a low horizontal stress field, it would be vice versa. On the other hand, the thickness of roof strata and properties of the bedding planes determine whether sliding or dome shape failure will occur. Failure is more due to sliding of the bedding planes in thin laminated strata, but dome shape failure is more possible in thick strata. The dimensions of roof falls have a direct relationship with roadway width, ie as the width of opening increases so does the height and bottom dimension of the roof fall.

Each of the structural failures; dome shape failure, guttering and sliding of the bedding planes may occur in certain locations at the intersection as shown in Figure 7.30. Guttering usually occurs over a 0.5 m to 0.7 m wide stripe parallel to the rib-line and extends vertically into the roof. The most probable location for initiation of sliding of the bedding plane is somewhere between 0.2 W₀ to 0.4 W₀ from the centre-line. The dome or arch shape failure usually covers the whole width of the opening and its height is about 0.5 W₀ to 1.0 W₀.
Dimensions of the zone of influence around an intersection are related mainly to the width of the opening and the stress state. This was determined by means of the parametric analysis presented in Section 7.2.2. The intensity and mechanism of disturbance resulting from an intersection varies from area to area around the structure. The whole region can be divided into three major areas. Figure 7.31 illustrates an intersection divided into distinct zones according to the intensity of disturbance.
Type I areas have normal conditions like a single roadway. The design of a support system is carried out in accordance with the procedure described for main entry roadways (Section 6.3.3). Areas denoted Type II, compared to that in a single roadway, experience higher stress concentrations because of the intersection. For these areas it is suggested that the bolt length be increased by 25% and the spacing reduced by the same percentage. The central area in Figure 7.31, Type III area, is the most disturbed zone at the intersection. The design of reinforcement in this region must be related to the type of structural failure predicted (associated with the state of stress). The discussion presented in Sections 7.2 and 7.3 must be taken into account in this regard. Normally, bolts in this area should be 50% longer than bolts in region II, and also their carrying capacity must be correspondingly higher.

It is suggested, particularly in high horizontal stress fields, that where practical, the roadway running parallel to the major horizontal stress be driven first, install the specific support system at the location of the proposed intersection and then cut through the other roadway. This practice will help to prevent excessive movement due to construction of the intersection which can initiate various types of structural failure. Another measure which can help to maintain the stability of the intersection is to use inclined roof bolts to prevent roof guttering. These bolts must be long enough to reach the high vertical stress concentration zone over the pillars. Also, W-straps parallel to the rib-sides should be installed before driving the second roadway. Figure 7.32 shows the position of inclined roof bolts in the form of a roof truss, and parallel W-straips.

Figure 7.32 Particular support system which should be implemented before the intersection is cut through.
7.5 Stability analysis of three-way (T) intersections

This part of the research was carried out to determine the general behaviour of three-way intersections under various loading conditions. Assuming a symmetric condition around the three-way intersection, half of the structure was taken for modelling. 8-node solid elements were used to build up the model. The model consisted of 7190 elements and 11597 grid points. The computer running time was about 17 hours and around one gigabyte of memory was engaged during this time. The general view of the model is illustrated in Figure 7.33 and the properties of rocks are tabulated in Table 7.8.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Thickness (m)</th>
<th>E (GPa)</th>
<th>V</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandstone (m.g.)</td>
<td>4.0</td>
<td>10.0</td>
<td>0.20</td>
</tr>
<tr>
<td>sandstone (f.g.) + mudstone</td>
<td>3.0</td>
<td>6.0</td>
<td>0.25</td>
</tr>
<tr>
<td>sandstone (c.g.) + shale</td>
<td>2.0</td>
<td>3.0</td>
<td>0.20</td>
</tr>
<tr>
<td>top coal</td>
<td>1.0</td>
<td>3.5</td>
<td>0.30</td>
</tr>
<tr>
<td>coal</td>
<td>3.0</td>
<td>3.5</td>
<td>0.30</td>
</tr>
<tr>
<td>mudstone</td>
<td>1.0</td>
<td>8.0</td>
<td>0.25</td>
</tr>
<tr>
<td>sandstone (c.g.)</td>
<td>4.0</td>
<td>12.5</td>
<td>0.20</td>
</tr>
<tr>
<td>sandstone (m.g.)</td>
<td>5.0</td>
<td>10.0</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Four loading conditions were taken into account in this analysis as summarised in Table 7.9. Loads were applied to the model by means of uniform pressure on the internal free faces. This technique, loading from the inside, helped to reduce size of the model and also to eliminate boundary effects on the results. In all loading conditions, a linear solution method was used. Results of this study included 150 figures and data files; only selected results are presented in this Section.

<table>
<thead>
<tr>
<th>Loading conditions</th>
<th>$\sigma_Z$ (MPa)</th>
<th>$\sigma_Y$ (MPa)</th>
<th>$\sigma_X$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>case 1</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>case 2</td>
<td>10.0</td>
<td>10.0</td>
<td>10.0</td>
</tr>
<tr>
<td>case 3</td>
<td>10.0</td>
<td>10.0</td>
<td>20.0</td>
</tr>
<tr>
<td>case 4</td>
<td>10.0</td>
<td>20.0</td>
<td>10.0</td>
</tr>
</tbody>
</table>
Figure 7.33 3-D and plan view of general three-way intersection model.

Figure 7.34 shows the induced vertical stress distribution throughout the 3-D model for a litho-static condition. However, to better understand the behaviour of the structure, the vertical stress concentration on various horizontal and vertical planes was studied for different loading conditions by plotting the stress concentration contour lines, where $r$ is the ratio of induced to virgin stresses.
7.5.1 Pillar behaviour at three-way intersections

Vertical stress concentration patterns at the mid-height of the pillar under various loading conditions are shown in Figure 7.35. Comparison of the results led to the following conclusions:

(i) When $K_x = K_y = 0$ or 1, the stress concentration pattern has a symmetric shape at the mid-height of the pillar. The disturbed zone along the rib-sides in both roadways has a width of 2.5 m, equal to half the roadway span. The maximum stress concentration is about 1.4 times the virgin stress, and extends in a similar manner along all rib-sides when $K_x$ and $K_y = 0$. For loading conditions $K > 0$, the maximum stress concentration value occurs over a limited zone at the corner of the pillar.
Figure 7.5  Vertical stress concentration pattern at the mid-height of the pillar (three-way intersection).
(ii) When $K > 1$, the vertical stress concentration pattern at the mid-height of the pillar is no longer symmetric. The variation of stress values is more pronounced along the roadway which is perpendicular to the direction of maximum horizontal stress. No tensile zone along the rib-sides was detected. Based on previous studies (7.2.2) this was because $K \leq 2$. The maximum stress concentration zone is located close to the edge of the pillar and extends along the roadway perpendicular to the major horizontal stress.

### 7.5.2 Roof behaviour at three-way intersections

The vertical stress concentration pattern for planes 0.5 m, 1.5 m and 2.5 m above the roof line were studied. Only the results for the plane 1.5 m above the roof line are presented in Figure 7.36. In addition, those results on the vertical plane at the mid-span of the main roadway and cut-through are given in Figure 7.37 and 7.38, respectively. The following conclusions can be made by comparing these results:

(i) There is a stress concentration of $1.1\sigma_Z$ over the edge of the pillar for locations very close to the roof line (less than 0.5 m above the roof line). This stress contour line diminishes farther into the roof. This pattern illustrates a semi-dome shape destressed zone over the three-way intersection. Moreover, when the ratio of horizontal to vertical stress ($K$) increases, the stress contour line $0.2\sigma_Z$ moves towards the centre of the roadway while the $1.0\sigma_Z$ line moves further into the pillar.

(ii) When $K_x \neq K_y$, the stress pattern varies over the individual roadways. The $0.2\sigma_Z$ contour line has limited dimension in the roadway perpendicular to the major horizontal stress. In this condition, the boundary of the roof fall in this roadway will be limited by stress contour lines greater than $0.2\sigma_Z$. However, the rate of change in stress distribution across the roof line of the other roadway is more significant. The height of the roof fall in each of the roadways might be evaluated by using the appropriate destressed contour line plotted in Figures 7.37 and 7.38.

(iii) The radius of influence of the intersection over the individual roadways with respect to the stress distribution in the roof is estimated to be one roadway span from the centre of the intersection.
Figure 7. 36 Vertical stress concentration pattern on a plane 1.5 m above the roof-line (three-way intersection).
Figure 7. 37 Vertical stress concentration pattern on the vertical plane at the mid-span of the main roadway (three-way intersection).
Figure 7.38 Vertical stress concentration pattern on the vertical plane at the mid-span of the cut-through (three-way intersection).

(iv) The shear stress patterns along the left and right sides of the main roadway and along the rib-side of the cut-through were also studied. It is obvious that there would not be much shear stress over the rib-sides when $K = 0$, but in the case where $K_x = K_y = 1$, the shear stress pattern was similar over the rib-lines in both roadways with a maximum value of about 4.0 MPa. When $K_x \neq K_y$, the roadway perpendicular to the major horizontal stress had a higher shear stress concentration over the rib-line. The maximum shear stress was more than 6.0 MPa and this zone extended 1.0 m into the roof over the rib-line.

(v) Figure 7.39 shows the vertical displacement on the roof-line under various loading conditions at the three-way intersection. The maximum sag occurs at the centre of the intersection. The maximum value remains unchanged, 12 mm, under various horizontal stress states, but the roadway parallel to the major horizontal stress will show more roof sag than the roadway perpendicular to the horizontal stress.
Figure 7. 39 Vertical displacement (mm) on the roof-line, sag, at a three-way intersection.
7.5.3 Floor behaviour at three-way intersections

Behaviour of the floor at the T-intersection was also studied by analysing vertical displacement on the floor-line. Figure 7.40 shows floor heave contour lines under various loading conditions. General conclusions made for roof sag can be extended to floor heave except that the maximum heave is much less than the corresponding value for sag.

7.5.4 Guide-lines for designing the support system at three-way intersections

Results of investigation on three-way intersections showed that the general pattern of stress and displacement are somehow similar to that around four-way intersections. Based on the study presented in this Section and those given for four-way intersections, the following conclusions can be made:

(i) The maximum vertical stress at the mid-height of the seam occurs at the corner of the pillar and increases with the roadway width and seam depth. The disturbed zone over the pillar extends along the roadway perpendicular to the major horizontal stress. A uniform pattern of horizontal dowels in conjunction with some sort of mesh would be necessary to maintain the stability of pillars. A bolt length equal to or longer than 50% of the entry width at 1.0 m spacing is suggested. This pattern should be implemented on the edge of the pillar extending along the roadways for a distance equal to one roadway span. The rest of the pillar in the individual roadways should be reinforced (if necessary) according to the single roadway condition.

(ii) The three potential mode of failure explained in previous sections should be taken into account where trying to design the optimum roof bolt pattern at three-way intersections. The following notes are pertinent:

(ii-a) The first mode is a semi-dome shape failure zone over the T-intersection. One side of the zone is parallel to the left side of the main roadway, and the base is a semi-circle. When \( K_X \neq K_Y \), the base of the zone will have different length in each roadway, with the longer length perpendicular to the major horizontal stress. Although the properties of roof strata have significant effect on the stability of the roof, the stress contour lines \( 0.1 \sigma_z \) and \( 0.3 \sigma_z \) can be used to define the boundary of the failure zone above the T-intersection for \( K < 2 \) and \( K > 2 \) respectively.
Figure 7.40 Vertical displacement (mm) on the floor-line, heave, at a three-way intersection.
(ii-b) The second mode is shear on the bedding planes. This happens when the shear stress exceeds the frictional strength of the bedding planes. Note should be taken that the required length of fully grouted bolt depends on the cohesion, coefficient of friction and location of the bedding planes. Thus, a general roof bolt pattern cannot be made for all conditions. However, the most probable location for sliding of bedding planes is closer to the rib-side than to the centre line. For an accurate design, it is necessary to construct and analyse site-specific models based on accurate field data.

(ii-c) The third potential mode of failure is guttering along and over the rib-sides and also at the corners of the intersection. This is more likely to happen when the horizontal stress is greater than the vertical stress. Inclined roof bolts passing through this zone and anchored over the pillars are strongly recommended.

(iii) The three-way intersection causes specific disturbance at and around its location. The area around the structure can for all practicality be divided into two regions as shown in Figure 7.41.

![Figure 7.41 Different regions around the three-way intersection with respect to the support system.](image)

The design of the support system in Region I is carried out based on the procedure for individual roadways. In Region II, special adjustments to roof bolt length and location must be considered. The bolt length must be increased by 50% and the spacing be reduced by 25% in comparison to bolts in Region I. The load carrying capacity of the bolt must then be calculated accordingly. It is suggested that the main roadway be driven
first, implement the specific support system at the proposed intersection and then drive the cut-through.

7.6 Case study of three-way intersections

The overall objective of this particular study was to validate the results of 3-D FE model of T-intersections by comparing them with those obtained from field measurements. The site was an underground coal mine in Southern Coal Fields of the Sydney Basin, Australia. An investigation into the mechanism of roadway deformation in problem areas, particularly at intersections located in the tailgate of a longwall panel was investigated. Field measurements included roof sag, floor heave and rib deformation monitoring ahead and behind of a longwall face. The in-situ measurements were conducted under a contract between the mine and Strata Control Technology, Pty. Ltd., SCT. The results of the field investigation (SCT 1993) were compared with those predicted by the finite element modelling.

7.6.1 General information on the mine site and the FE models

Details of longwall panels, gate roadways and intersections at the site of this investigation are shown in Figure 7.42. Panels are about 200 m wide and 2000 m long. Gate roads are double entry system. Each roadway is 5.0 m wide and pillars are 55 m × 40 m from centre to centre. The height of extraction varies between 2.4 m and 2.6 m. The problem areas between 35 and 36 Cut-throughs (C/T), and 9, 35 and 36 intersections of Longwall 24 Tailgate were chosen for site investigation. The vertical stress was approximately 10 MPa at a depth of 420 m. The major horizontal stress, $\sigma_{h1}$, was 25.0 MPa oriented parallel to the gate roads and the minor horizontal stress was 10.0 MPa. Figure 7.43 illustrates the lithology profiles of the strata units at the particular investigation areas. Accordingly, the mechanical properties of the strata units are tabulated in Table 7.10.

Based on the above information a number of 2-D and 3-D FE models were constructed and analysed to simulate the existing conditions around the investigation spots. FE model of roadways and intersections were analysed for induced stress and displacement around the structures. The results of the FE study are grouped and presented in such a way that would allow easy comparison with the values obtained from field measurements.
Figure 7.42 General plan view of the mine site.
Figure 7.43 Lithology profiles of the strata column at:

Table 7.10 Mechanical properties of rocks in the strata units.

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>$\sigma_0$ (MPa)</th>
<th>UCS (MPa)</th>
<th>$\phi$ (°)</th>
<th>$E$ (GPa)</th>
<th>$v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>sandstone m.g.</td>
<td>4.0</td>
<td>50</td>
<td>4.0</td>
<td>9.0</td>
<td>0.2</td>
</tr>
<tr>
<td>broken shale</td>
<td>1.5</td>
<td>10</td>
<td>2.8</td>
<td>1.0</td>
<td>0.28</td>
</tr>
<tr>
<td>shale + sandstone</td>
<td>3.0</td>
<td>30</td>
<td>3.3</td>
<td>5.0</td>
<td>0.25</td>
</tr>
<tr>
<td>coal</td>
<td>1.5</td>
<td>15</td>
<td>3.0</td>
<td>3.5</td>
<td>0.30</td>
</tr>
<tr>
<td>shale + sandstone + claystone</td>
<td>3.5</td>
<td>35</td>
<td>3.5</td>
<td>6.0</td>
<td>0.23</td>
</tr>
<tr>
<td>sandstone m.g. &amp; c.g.</td>
<td>4.2</td>
<td>53</td>
<td>4.0</td>
<td>12.0</td>
<td>0.2</td>
</tr>
</tbody>
</table>
7.6.2 Field instrumentation

A series of roof, rib and floor extensometers were installed at and between 35 and 36 Cut-throughs and at 9 Cut-through ahead of Longwall 23. The objective of the installation was to determine the pattern of deformation around the problem areas. In addition, the mechanism of failure around these locations was surveyed. The layout of the extensometers and typical failure mechanisms are depicted in Figure 7.44.

Figure 7.44 Layout of the extensometers at various spots in Longwall 24 Tailgate (SCT 1993).
7.6.3 Comparison and interpretation of the field and FE results

Longwall 23 had passed 35 and 36 C/Ts before all the extensometers were installed (SCT 1993). Monitoring continued till the face approached and passed 9 C/T and reached the panel end. No work was done when Longwall 24 approached and passed the investigation areas. All readings were regularly taken over 45 days, but for the sake of simplicity, only the initial and final readings are presented in the following figures. However, the time dependent deformation of the roof, rib and floor can be determined by comparing the initial and final readings, particularly at 9 C/T.

(i) Roof behaviour

The roof sag measurements at different locations were compared with values predicted by FEA (Figure 7.45). In all cases, the difference between the initial and final readings was very little. Therefore, it can be concluded that time dependent deformation of the roof was negligible. In addition, given the good visual condition of the roof, it was likely that softening had not occurred on drivage, and subsequent roadway deformation had not adversely affected the roof integrity (SCT 1993).

As there was a gap between construction of 35 and 36 C/Ts and installation of the extensometers, displacement measurements at these locations did not show the real total values. Consequently, the FE predicted values are greater than the measured values. In contrast, the field results relating to 9 C/T are in good agreement with those predicted by FEM. Comparing the results of deformation at 9 C/T before and after Longwall 23 passed, it can be seen that tailgate behaviour is significantly affected by the adjacent face.
Figure 7.45 Roof sag, measured and predicted values, (t₀ and tᵣ: initial and final readings, B and A: readings before and after LW passed the monitoring site); (a) at 35 and 36 C/Ts, (b) between 35 and 36 C/Ts, (c) at 9 C/T (field data after SCT 1993).
(ii) Rib behaviour

The results from rib extensometers and those predicted by FEA are presented in Figure 7.46. The results indicate a time dependent deformation of approximately 0.4 mm/day. As the time dependent behaviour of the seam could not be considered in the FE analysis of the structure, the FE predicted values are only the total deformation after complete relaxation; and are less than measured values.

The important aspect of the chain pillar between 35 - 36 C/T is the nature of the rib movement. The extensometer indicates that softening has occurred in a fairly uniform manner to a depth of approximately 3.5 m. This contrasts with the rib behaviour monitored at 9 C/T where the deformation into the pillar rapidly abates from the rib-line. The reason for this could be that the pillar between 35 - 36 C/T was not able to generate confinement (SCT 1993). The lack of confinement might be the result of either a low friction surface in the roof and floor or horizontal stress relief which necessitates a relatively large lateral movement in order to build up enough confinement in the pillar to sustain the vertical abutment.

(iii) Floor behaviour

The floor extensometer results and values predicted by FEA between 35 and 36 C/T and at 9 and 35 C/Ts are presented in Figure 7.47. The floor heave plots at 35 C/T and between 35 and 36 C/Ts show that the deformation is particularly evident between 1 m and 2 m into the floor. Referring to the lithology profiles presented in Figure 7.43, it can be seen that there is a very broken shale unit which behaves as a softened zone 1 m below the floor line. Although the shale unit is surrounded by a laminated fine sandstone/shale, it can generate an uplift stress to the immediate floor when failure occurs within it. This phenomenon was also detected during the investigations at Ellalong Colliery. As the floor core obtained from 9 C/T shows a uniform sequence of laminated sandstone/shale without any softened zone, the significant floor heave at 9 C/T is mainly due to the horizontal stress state and the effect of the longwall face. It was comprehensively demonstrated earlier in this Chapter and in Chapter 6 that roadways parallel to the major horizontal stress where $K > 1$ will have greater floor heave and roof sag when compared to roadways parallel to the minor horizontal stress.
Figure 7.46 Rib displacement, measured and predicted values, \( t_0 \) and \( t_f \): initial and final readings, B and A: readings before and after LW passed the monitoring site; (a) between 35 and 36 C/Ts, (b) at 35 C/T, (c) at 9 C/T (field data after SCT 1993).
Figure 7.47 Floor heave, measured and predicted values, (t₀ and t_f: initial and final readings, B and A: readings before and after LW passed the monitoring site); (a) between 35 and 36 C/Ts, (b) at 35 C/T, (c) at 9 C/T (field data after SCT 1993).
(iv) General conditions of the investigated sites

Drivage conditions were significantly influenced by: the orientation of the roadways with respect to the major horizontal stress, effect of longwall extraction and the presence of the softening zone in the floor. Field investigations indicated that drivage conditions parallel to the major horizontal stress were very good with minimal roof shear evident. In contrast, the drivage perpendicular to the major horizontal stress contained considerable shear zones particularly at locations close to the intersections. An increase in stress magnitude resulting from local geological structures accelerated the shear failure. These conditions were well illustrated and predicted during FE analysis of roadways and intersections under various conditions. Due to a number of reasons, comprehensively discussed in previous chapters, there are still some differences between field measurements and FE results. Uncertainty about the in-situ properties of the strata units and stress state as well as variations in geological features are examples of these reasons. On the other hand, there are a few short-comings in FE modelling of underground structures such as lack of consideration of time dependent behaviour of rocks and dynamic loading resulting from an approaching longwall face. In general, the significant agreement achieved between field and FE results proved that the FEM can be used as a very promising tool for stability analysis of underground structures.

7.7 Conclusions

In this Chapter the application of the finite element method to the stability analysis of four-way and three-way intersections was demonstrated. The study included comprehensive investigations on the behaviour of strata surrounding the intersections under various conditions. FE models were constructed based on data acquired from laboratory testing and field investigations (Chapter 5). A number of computer programs were written to obtain, group, process and plot the results of FEA in the form of various graphs and figures.

The research had two major parts; stability assessment of four-way and three-way intersections. Three major parameters, depth of cover (vertical stress), horizontal stress and the width of openings were taken into account in a parametric analysis of intersections. Moreover, site-specific models were constructed and analysed with consideration of more realistic conditions existing around the structures. The results
achieved from 3-D models of intersections supported those conclusions made from stability evaluation of 2-D roadway models (Chapter 6). The specific points from investigations presented in this Chapter can be summarised as follows:

(i) The stress concentration pattern around an intersection is almost the same for various depths of cover; however, the maximum values of resultant stress increase with an increase of depth. The dimensions of the disturbed zone around the intersection can be expressed based on the state of stress and roadway width. An overall estimation for the radius of influence of a four-way intersection is twice the opening width (measured from the intersection centre).

(ii) The highest stress concentration at the mid-height of the pillar for four-way intersections is between 2.0 $\sigma_Z$ and 2.2 $\sigma_Z$, depending on the roadway width. The diagonal diameter of the disturbed zone at the mid-height of the pillar is equal to the opening width (measured from the pillar's edge).

(iii) The destressed zone in the roof of the four-way intersection has a dome shape which extends further into the roof with increasing roadway width. This in turn results in a higher potential arch failure zone at the intersection. The height of arch zone decreases as $K$ increases. However, when the absolute value of the major lateral stress is significant other failure mechanisms may result in a higher failure zone.

(iv) Maximum sag and heave occur at the centre of the intersection. It was shown that the maximum values were highly influenced by the vertical stress, width of the opening and mechanical properties of rocks. Two Equations were developed to estimate the maximum sag and heave at four-way intersections. On the other hand, the maximum rate of change of sag and heave occurs over the rib-lines.

(v) The horizontal stress (magnitude and direction) has significant effect on the behaviour of the intersection. In a high horizontal stress field, guttering is more likely to occur in the roadway perpendicular to the major horizontal stress. On the other hand, sag and heave will be greater in the other roadway. When $K > 2$, there will be a vertical tensile zone on the rib-side in the roadway perpendicular to the major horizontal stress, and the stress distribution around the structure will be non-symmetric. A number of
figures and equations were proposed to estimate the dimension of the failure zone and the maximum values of sag and heave at four-way intersections.

(vi) It was suggested that the area around a four-way intersection be divided into three regions. The design of the support system in Region I (outside the radius of influence) is independent of intersection condition, but the support system in Regions II and III (within the radius of influence) must be enhanced, at least, by a factor of 1.2 and 1.5 respectively, in comparison to the individual roadways. In high horizontal stress fields it would be better to drive the roadway parallel to the major horizontal stress first, implement the specific support system at the proposed location of the intersection and then drive the cut through.

(vii) To design an optimum support system at the intersection, three potential failure modes; dome shape failure over the intersection, sliding of the bedding planes and guttering over the edges of the rib-lines should be examined against the mechanical properties of rocks in the strata units. It is also important to apply some measure, such as horizontal dowels, to help maintain the integrity of the confining part of the pillar. If the integrity of the pillar is not maintained, instability of the pillar may lead to instability of the whole structure.

(viii) Results of investigations on the stability of three-way intersections showed that the general stress and displacement patterns around the three-way intersection are to some extent similar to that of four-way intersections. In general, three-way intersections experience less severe stability problems than four-way intersections in similar field conditions. The depth of cover, horizontal stress and width of the opening have significant effect on the behaviour of the structure. The trend of influences is the same as that for four-way intersections.

(ix) The zone of influence around the three-way intersection has a radius equal to the opening's width. The design of the support system within the zone of influence should be enhanced, at least, by a factor of 1.3 in comparison to that of the individual roadways.
CHAPTER EIGHT

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH
CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

8.1 Summary

Stability of roadways and intersections has been comprehensively investigated during the course of this research. The thesis has presented methods and results of laboratory testing, field investigations and finite element analyses carried out over three years. The main theme and philosophy followed throughout the work was: (i) to obtain the most realistic information on the mechanical properties of rocks and features of the structures (roadways and intersections), (ii) to assess the behaviour and response of the roof, pillar and floor to various conditions, (iii) to identify major parameters governing the failure mechanisms of the structure, and finally (iv) to further the knowledge for designing an optimum, safe and economic support system to maintain the integrity and stability of roadways and intersections. This research was conducted to consider more realistic in-situ features with special reference to Australian underground coal mine conditions. The following conclusions were drawn on the basis of this research.

8.2 Conclusions

Theoretical approaches such as beam and plate theories, voussoir beam theory and the tributary area concept incorporate assumptions relating to the properties of rocks, loading conditions and in-situ features of the structure. The degree of uncertainty inherent in the problem is overcome by employing large factors of safety. Hence, these methods usually lead to an over estimate in design. In addition, there are some cases such as intersections in high horizontal stress fields with active bedding planes which are beyond the capability of theoretical methods.

8.2.1 The new modelling techniques in FEM

Although it has been shown that the Finite Element Method (FEM) can be a reliable tool for quantitative estimation of stress and displacement around underground structures, there is still some concern about the accuracy of the results. As a consequence, one major part of the present research was devoted to develop new techniques which take into account more realistic parameters during the construction and analysis of a FE model. High lights of the work in this part are as follows:
(i) The FE method is conceptually applicable to the continuum media. It basically assumes that all parts of the structure are tied together making one unit, and no separation in the form of either structural failure or sliding is encompassed in the model. On the other hand, bedding planes and major joints in real situations, may allow strata to separate from or slide over each other, and as such, the behaviour of the structure may be significantly affected by the behaviour of the bedding planes. This problem was overcome by employing and modifying a special element called the GAP element. The structural and load-displacement curves of GAP elements were adjusted and applied to FE models in such a way that they reasonably represented the properties and characteristics of the bedding planes and allowed separation and or sliding of the strata. The results of the study showed that so long as the bedding plane remained closed, there was not much difference between results obtained from models with GAP elements or models without GAP elements. Conversely, for conditions where the bedding plane was active the results from both models were significantly different; those from the GAP model were more realistic. Moreover, modelling the support system did not influence the results unless bedding planes were considered by using GAP elements.

(ii) Since the application of many GAP elements is relatively complex and cumbersome, particularly in 3-D models, back analysis was, for the first time, introduced to consider bedding plane effect indirectly. In this approach, the model was constructed without any bedding planes. After analysing the model for stress and displacement, a shear safety factor was calculated based on the shear and normal stresses induced on, and properties of, the bedding plane. Results were then presented in the form of shear safety factor contour lines on the bedding plane showing critical situations due to sliding of the bedding planes.

(iii) To consider the post-failure behaviour of rocks, a new technique was developed and utilised in finite element analysis. In this method which is a combination of the 3-D FE code, NASTRAN, and the SAFETY program developed during this research, a relationship between post-failure strength and post-failure stiffness was established and employed in an iterative procedure. During the solution phase material properties including the stress-strain curve were updated in order to simulate the failure of individual elements and progressive increase in the size of yield zone around the opening. Analysis was terminated when a steady state condition was reached. Any failure criteria such as the Mohr-Coulomb, Bieniawski, Hoek and Brown and even an experimental criterion can be employed in this procedure based on the users choice. This procedure was used in both 2-D and 3-d models of roadways and intersections. The results from this investigation were very promising.
Based on the achievements of the present research, it was demonstrated that loading technique affects the results of finite element analysis. While gravitational loading is an appropriate loading technique for surface or shallow structures, uniform loading is more suitable for deep structures. It was shown that the new loading technique, the Internal Loading Technique (loading from inside the opening), produced more accurate results of stress and displacement, eliminated boundary effects on the results and made it possible to reduce the size of the model. The latter advantage was most significant for 3-D modelling, as minimising computer memory required and CPU time that was critical when running these models.

8.2.2 Laboratory and in-situ properties of rocks

This thesis also indicated the necessity of using accurate and representative data for the mechanical properties of rocks in the stability of roadways and intersections. Extensive laboratory testing was carried out on six major coal measures rocks. Mechanical properties of the rocks, including the complete stress-strain curve, were obtained by means of uniaxial and triaxial compressive, direct shear, point load and Brazilian tests. Conventional failure criteria, the Mohr-Coulomb, Bieniawski and Hoek and Brown criteria were examined against the triaxial compressive results. Some problems were encountered with the above criteria and as a result a new Experimental criterion was proposed.

Attention was drawn to the effect of scale on the mechanical properties of a rock mass. An attempt was made to use available methods to predict the in-situ properties of strata units from laboratory results. The CSIR classification system (RMR index) was used to assess the geotechnical parameters of the strata units. Moreover, the NGI system (Q index) and CMRS system (R index) were examined using equations which expressed the relationship between these classification systems. The results of in-situ strength of strata units after the Hoek and Brown criterion were used to calculate the in-situ safety factor around a FE model of a roadway. The results were significantly far from reality. It was demonstrated that available approaches for estimating the in-situ strength of rock masses cannot be used in FEA. Therefore, a new equation and procedure was suggested and used in conjunction with the Experimental criterion. Application of this approach into the model used in the previous analysis gave a very realistic prediction of the failure zone around the roadway.
8.2.3 Field investigation

This research has once again shown that results obtained from any type of simulation or modelling technique are only as reliable as the data on which the model was based. Therefore, the most important stages in using geotechnical modelling for mine design is firstly preparation of the input data, and then validation of the output results. This thesis has reviewed various in-situ measurements and roof fall surveys conducted in two underground coal mines in NSW, Australia. The field investigation included stress measurements, displacement monitoring, roof fall surveys and observations from problem sites. During this investigation, it was also indicated that horizontal stress values of several times the vertical stress are common in Australian underground coal mines. At one of the sites, the ratio of major horizontal to vertical stress was 3 to 4 and that of minor component was about 1.5 to 2. Variations were primarily due to rock stiffness, depth and the effect of local structures.

It was identified that the height of the softening zone was the height where considerable roof displacement initiated. This was, in most cases, the maximum height of roof falls. Variations in roof displacement and height of the softening zone were due to changes in the horizontal stress state, width of the opening and properties of rocks. Roof fall surveys in roadways and at intersections indicated that an arch or dome shaped failure occurred over the opening, the bottom diameter of which was limited to the roadway span or the diagonal span of the intersection. Guttering usually initiated from one side and propagated into the roof causing an arch or dome shaped failure. Based on the parametric analysis of the intersection models, a number of equations were derived to estimate the height and bottom diameter of the roof falls in roadways and at intersections. Major parameters in these equations are span of the opening, stress state and properties of strata units.

8.2.4 Stability of roadways

This work also involved a comprehensive study of the behaviour of roadways and intersections using 2-D and 3-D finite element models. Many models were constructed and analysed by changing various parameters such as loading conditions, properties of rocks and dimension of openings. This study was designed to define the behaviour of the roof, pillar and floor under different conditions as well as to determine the degree of influence resulting from individual parameters on the stability of roadways and intersections. The input data were obtained from laboratory tests and field investigations. Models were constructed and analysed using new FE optimisation techniques.
The research on the stability of 2-D roadway models targeted three major factors which may significantly affect the stability of roadways. These factors are high horizontal stress, post failure behaviour of rocks and abutment pressure from longwall faces. Based on the results of this study, it can be concluded that:

(i) High horizontal stress has significant effect on the behaviour of underground structures particularly when $K$, the ratio of horizontal to vertical stress, is bigger than 2. In this condition, the destressed zone in the roof extends further into the roof and over the pillars, and also a tensile zone appears in the immediate roof. In addition, there would be a vertical tensile zone on the rib side, in the confining part of the pillar, causing severe spalling. Guttering is more likely to occur over the rib-lines in roadways perpendicular to the major horizontal stress while sag of the roof and heave of the floor will be more pronounced in roadways parallel to the major horizontal stress.

(ii) Properties of rocks, particularly the strength softening behaviour of weak rocks after the peak strength, significantly affects the behaviour of underground structures. Results obtained from softening models were very close to measured values.

(iii) Abutment pressures from longwall faces have appreciable effect on the stability of gate roadways, particularly at locations close to the finish position of the face. The simulation technique used in this research showed the stress and displacement patterns at various locations around a longwall face.

8.2.5 Stability of intersections

Further application of the Finite Element Technique has been extended to assess the stability of four-way and three-way intersections. In this research, the major parameters, depth of cover, horizontal stress and width of opening were included in parametric analysis to predict the most critical condition that may occur around the intersections. Based on the achievements of this research, a number of equations were derived to express the relationship between the above parameters and dimension of the disturbed and yield zones. In addition to parametric analysis of general models, site-specific models of intersections were constructed and analysed based on real data and in-situ conditions. The results achieved from 3-D models of intersections supported the conclusions made from stability evaluation of roadways using 2-D models. The specific points from the stability investigation of intersections can be summarised as follows:
(i) The stress concentration pattern around an intersection is almost the same for various depth of cover; however, the maximum value of the resultant stress increases with an increase of depth. Dimensions of the disturbed zone around an intersection can be expressed as a function of the state of stress and roadway width. An overall estimation for the radius of influence of an intersection is twice the opening width (measured from the intersection centre).

(ii) The highest stress concentration rate at the mid-height of the pillar at four-way intersections is between $2.0 \sigma_z$ and $2.2 \sigma_z$, depending on the roadway width. The diagonal dimension of the disturbed zone at the mid-height of the pillar is equal to the opening width (measured from the pillar's edge).

(iii) The destressed zone in the roof at four-way intersection has a dome shape and at three-way intersection a semi-dome shape. The destressed zone expands further into the roof with increasing roadway width. In turn, resulting in a higher potential failure zone at the intersection. The height to the top boundary of the failure zone decreases when $K$ increases.

(iv) The maximum sag and heave occur at the centre of the intersection. It was shown that the maximum values are highly influenced by the vertical stress, width of the opening and mechanical properties of rocks. Two Equations were developed to estimate the maximum sag and heave at four-way intersections. On the other hand, the maximum rate of change of sag and heave occurs over the rib-lines which in turn induces tension at these locations.

(v) The area around a four-way intersection can be divided into three regions. The design of the support system in Region I (outside the radius of influence) is independent of intersection, but the support system in Regions II and III (within the radius of influence) must be enhanced, at least, by a factor of 1.2 and 1.5, respectively, in comparison to the individual roadways. In high horizontal stress fields, it would be better to drive the roadway parallel to the major horizontal stress first, implement the specific support system at the proposed location of the intersection and then drive the cut through.

(vi) Results of investigations on the stability of three-way intersections showed that the general stress and displacement patterns around a (T) intersection are to some extent similar to that at four-way intersections, but with less severe stability problems. The depth of cover, horizontal stress and width of the opening have significant effect on the
behaviour of the structure. The trend of influences is the same as that for four-way intersections.

(vii) The zone of influence around a (T) intersection has a radius equal to the opening's width. The design of support system in the zone of influence should be enhanced, at least, by a factor of 1.3 in comparison to that of the individual roadways.

(viii) To design an optimum support system, three potential modes of failure (structural failure criteria); dome shaped failure over the opening, shearing along the bedding planes and guttering over the edges of the rib-lines should be examined against the mechanical properties of rocks in the strata units. It is also important to apply some measures, such as horizontal dowels, to maintain the integrity of the confining part of the pillar. If the integrity of the pillar is not maintained, the instability of the pillar may lead to instability of the whole structure.

Laboratory testing, in-situ monitoring and finite element analysis undertaken during the course of this research have successfully shown that stability problems in roadways and at intersections can be quantified and as a result alternative support systems may be designed in such a way as to overcome these problems. In particular, the role of the Finite Element Method using newly developed techniques, was clearly demonstrated as both a planning and an investigation tool for many aspects of strata control in roadways and at intersections. Finally, it can be said that all objectives of the project have successfully been achieved by the end of the research.

8.3 Recommendations for future research

The influence of various parameters on different aspects of the stability of roadways and intersections were investigated and presented in this thesis. The following lines of future research are recommended based on the achievements of the present study.

Mechanical properties of rocks in the strata units are important as realistic input data for the FEA of underground structures. The stress-strain curve including the post-failure behaviour and strength parameters of rocks obtained from laboratory tests should be processed in such a manner so as to get the most realistic in-situ characteristics of strata units. Therefore, it is recommended that further investigation be conducted, particularly in relation to coal field strata units, to develop reliable techniques for considering an appropriate scale factor.
The application of 3-D FEM using newly developed techniques should be extended to include dynamic or wave shape loading imposed on the underground structure as a result of adjacent working areas. Further refinement of the analysis should also be performed to include time effect on the material properties during stability analysis of underground structures. In addition, the benefits from advanced computer technology should be used to fully automate all procedures of modelling and analysis as well as to construct models which consider more details of the structure.

Variations of the in-situ properties of strata units and stress state as well as variations of geological features are reasons why stability problems occur at one site but not at another mine or even at an adjacent site. Therefore, from the practical point of view, it is suggested that sampling and testing of rocks and in-situ instrumentation should periodically be performed to indicate any possible changes which may affect the behaviour of underground structures. Then, the support system should be adjusted to meet the requirements of the new conditions. It is also recommended that more specific investigations be conducted to assess the performance of the current support systems and determine whether any changes are needed for different situations.

Although the results obtained from the present research were very promising, the true value of the work cannot be substantiated until the predicted results are validated by more field instrumentation under different conditions. It is believed that the data available from in-situ measurements and monitoring in Australian underground coal mines is very limited, and also difficult to access, in comparison to that in the USA and European countries. However, it is recommended that the results of this research be employed as a base for stability assessment of new sites, and to validate the accuracy of the predictions more in-situ measurements be carried out.
REFERENCES
AND
BIBLIOGRAPHY
References

Adler, L. and Sun, M. C. 1968, "Ground Control in Bedded Formations", Bulletin 28, Research Division, Virginia Polytechnic Institute, USA.

Australian Coal Industry Research Laboratory 1989, "Ellalong Colliery chain pillar design and floor heave investigations", Australian Coal Industry Research Laboratories Ltd., ACIRL Report No. CMU 1855A

Australian Coal Industry Research Laboratory 1987, "Rock mechanics investigations at Ellalong Colliery to assist future secondary support requirements", Australian Coal Industry Research Laboratories Ltd., ACIRL Report No. 08/1774

Australian Coal Industry Research Laboratory 1972, "Examination of roof and floor strata obtained from Ellalong borecores", Australian Coal Industry Research Laboratories Ltd., ACIRL Report No. 411, 6th June


Bieniawski, T. Z. 1984, "Rock Mechanics Design in Mining and Tunnelling", A. A. Balkema, Rotterdam


Chase, F. E. and Mark, C. 1990, "The effect of hazardous geologic structures on gateroad stability", 9th Int. Conf. on Ground Control in Mining, Peng (ed.), West Virginia University, Morgantown, WV, USA, pp 218 - 229


Frith, R. C. 1993, "Strata Control of Gate Roadways", Strata Control from Principals to Practice, University of Wollongong, November 1993, Wollongong, Australia

Frith, R. C., Reddish, D. J., and Watson, T. P. 1990, "Roadway support and design in the new UK coal industry", The Mining Engineer, October 1990, pp 124 - 127

Fuller, P.G. et al 1987, "An assessment of factors relevant to secondary roof support design for longwall gate roads at Ellalong Colliery, Cessnock, NSW", Consulting Engineers and Geologist, Barrett, Fuller & Partners, BFP, August 1987


Hanna, K. et al 1986a, "Effect of high horizontal stress on coal mine entry intersection stability", Proc. 5th Conf. on Ground Control in Mining, Khair and Peng (eds.), pp 167 - 182


Hanna, K., Haramy, K. and Conover, D. 1985, "Field investigations of roof and pillar stability in coal mine intersections", Proc. 2nd Conf. on Ground Control Symp. in Illionis Coal Basin, USA, pp 76 - 83


Hematian, J. 1989, "Investigations on Roof Control Techniques in Longwall Mining", MSc Thesis, Amir-Kabir Univ., Tehran, Iran, (Persian)


Hematian, J. and Porter, I. 1993(b), "Stability Analysis of Roadways and Intersections at Ellalong Colliery", Report No. 1, Department of Civil and Mining Engineering, University of Wollongong, NSW, Australia
Herget, G. 1988, "Stresses in Rock", A. A. Balkema, Rotterdam


King, H. J., Whittaker, B. N. 1970, "A review of current knowledge on roadway behaviour, especially the problems on which further information is required", Proc. Symp. on Strata Control in Roadways, April 1970, Nottingham, pp 73 - 87

Krese, J. M. 1988, "An overview of the national roof evaluation accident prevention (REAP) program", 7th Int. Conf. on Ground Control in Mining, Morgantown, WV, USA, August 1988, pp 62 - 65


Kropotkin, K. N. 1972, "The state of stress in the Earth's crust as based on measurements in mines and geophysical data", Phys. Earth Planet, Interior 6, pp 214 - 218

Lackey, S. F. 1973, "Numerical investigations into the significance of rock properties and support on the stability of underground roadways in coal mines", Australian Coal Industry Research Laboratories Ltd., ACIRL, PR 84 - 12


Mark, C. and Bieniawski, Z. T. 1986, "Field measurements of chain pillar response to longwall abutment loads", Proc. of 5th Conf. on Ground Control in Mining 1986, Khair and Peng (eds.), pp 114 - 122


McNabb, K. E. and Wardle, L. J. 1986, "Numerical modelling of development roadways", German Creek Central Colliery, Central Queensland, Report NO.63, CSRIO, Division of Geomechanics


MSC/NASTRAN Theoretical Manual 1981, NASA SP, 221 (06), Los Angeles, California, USA


NSW Coal Industry Profile 1993 and 1994, Incorporating Joint Coal Board Statistical Supplement, NSW Department of Mineral Resources, St Leonards, NEW, Australia


Patrick, W. C. and Aughenbaugh, N. B. 1979, "Classification of Roof Falls in Coal Mines", Mining Engineering 31(3), March, pp 279-284


Peng, S. S. and Okubo, S. 1978a, "Roof bolting patterns at four-way intersections", West Virginia Univ., Coal Research Bureau, Report No. 150, May, 30 pp

Peng, S. S. and Okubo, S. 1978b, "Roof bolting patterns at three-way intersections", West Virginia Univ., Coal Research Bureau, Report No. 158, May, 35 pp

Pothini, B. R. 1978, "Roof fall prediction at Island Creek Coal Company", Stability in Coal Mining, Brawner and Dorling (eds.), Proc. of the 1st Symp. on Stability of Coal Mining, Vancouver, British Colombia, pp 214-227

Price, D. 1994, "Longwall gateroad development systems: What to watch for this year", Australia's Longwalls, Australia's Mining Monthly, Central No. 1, Australia, pp 47-51


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Shepherd, J. et al 1987, "Geotechnical methods for the evaluation of underground roadway conditions and support requirements", Australian Coal Industry Research Laboratories Ltd., ACIRL, PR 87 - 2


Steele, J. M. 1989, "Applied Finite Element Modelling, Practical Problem Solving for Engineers", Publisher M. Dekker, New York


Vervoot, A. 1990, "A statistical analysis of falls of ground in South African Collieries", 9th Int. Conf. on Ground Control in Mining, Peng (ed.), West Virginia University, Morgantown, WV, USA, pp 285 - 292


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**Wu, Y. H. and Salamon, M. D. G. 1992**, "Modelling the behaviour and strength of pillars and ribsides", Joint Coal Board Strata Control Project, School of Mines, University of NSW, Australia


Bibliography


Bieniawski, Z. T. 1987, "Strata Control in Mineral Engineering", A. A. Balkema, Rotterdam


Dixon, J. D. and Thompson, R. R. 1990, "Analysis and field testing of pre-support application in coal strata", Proc. 9th Int. Conf. on Ground Control in Mining, WV Univ., pp 42 - 51


Galvin, J. Anderson, I. and Stothard, R. 1991, "Report of Study Tour Investigations into Pillar Extraction at Shallow Cover, South Africa and the USA", The School of Mines, University of New South Wales, Australia


Guo, L. B. and Peng, S. S. 1990, "A 3-D boundary element program for long-term stability analysis of entry intersection in underground coal mines", Department of Mining Engineering, College of Mineral and Energy Resources, West Virginia University, USA

Hargraves, A. J. and McCoy, A. E. R (eds.) 1974, "Supports in pillar extraction", Proc. of the Colloquium on Support in Pillar Extraction, AusIMM, Univ. of Wollongong NSW, Australia

Hematian, J. and Porter, I. 1993(c), "Stability Analysis of Roadways and Intersections at Ellalong Colliery", Report No. 2, Department of Civil and Mining Engineering, University of Wollongong, NSW, Australia


Jaggar, F. 1976, "The development and applications of finite element computer techniques to mining structures", Australian Coal Industry Research Laboratories Ltd., ACIRL, PR 76 - 10

Jaggar, F. E. 1970, "Model investigation to determine the significance of slotting the roof strata of roadways for stress relief", Australian Coal Industry Research Laboratories Ltd., ACIRL, PR 70 - 5

Jaggar, F. E. 1967, "The importance of shape and width of mine roadways in a stressed environment", Australian Coal Industry Research Laboratories Ltd., ACIRL, PR 67 - 7

Jeremic, M. L. 1987, "Ground Mechanics in Hard Rock Mining", A. A. Balkema, Rotterdam


Quinterio, C. 1992, "Behaviour of strip pillars under tributary load", Joint Coal Board Strata Control Project, School of Mines, University of NSW, Australia


Salamon, M. D. G. 1992a, "A coal seam with strain-softening properties enbedded in a stratified rock mass", Research Report 1/02, Joint Coal Board Strata Control Project, School of Mines, University of NSW, Australia

Salamon, M. D. G. 1992b, "A review of coal pillar mechanics and design", Joint Coal Board Strata Control Project, Research Report 1/01, School of Mines, University of NSW, Australia

Salamon, M. D. G. and Oravecz, K. I. 1976, "Rock Mechanics in Coal Mining", Johannesburg, Chamber of Mines of South Africa


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