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Floor failure mechanisms at underground longwall face

Jan Anton Nemcik
University of Wollongong

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FLOOR FAILURE MECHANISMS
AT UNDERGROUND LONGWALL FACE

A Thesis submitted
In fulfilment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

from

UNIVERSITY OF WOLLONGONG

By

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B. Eng. (Hons), M. Eng. (Hons)

Faculty of Engineering
February 2003
I, Jan Anton Nemcik, declare that the work presented in this thesis is my own unless otherwise referenced or acknowledged and has not been previously submitted for a degree to any other University or Institution. The following is a list of my publications related to this research program:

**Refereed Journals:**


**Refereed Conferences**


JAN ANTON NEMCIK
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ABSTRACT

Floor failure in coal mines has not been studied to the same extent as roof failure despite causing significant delays in production. Understanding floor failure mechanisms when mining tabular deposits is, therefore, extremely important. This thesis reviews current understanding of stone floor failure and also explains several other floor failure mechanisms that can occur at the longwall face. Key objective of this study is to provide geotechnical engineers with comprehensive guidelines for predicting floor failure, and outline possible solutions to this problem.

Most commonly encountered floor problems are associated with floor heave and floor puncturing below the hydraulic supports. Although, there are many variables that can contribute to these problems, identifying the causes is often complex and requires a considerable understanding of failure mechanisms. In this research study, two major floor failure mechanisms were identified, primary floor failure and secondary floor failure. Primary floor failure occurs in response to the triaxial stress state ahead of the coal face, where stresses are high. Secondary floor failure can be attributed to post failure distribution of stress and subsequent floor displacements. Five mechanisms of secondary floor failure were identified. Typically, these failures cause floor heave that may interfere with the longwall operations or affect stability of the powered supports.
The puncture of weak stone floor that often occurs when load is applied onto the weak floor below the powered supports is the only failure mode that has been well understood for some time. This failure which has been researched in both soil mechanics and rock mechanics, resembles a foundation bearing capacity type of failure mechanism. Buckling of bedded floor strata that often occurs between the longwall face and the bases of the powered supports is associated with excessive yielding of the seam deposit. Failed seam moves towards excavated goaf, and if coupled with the stone floor, can shear the floor bedding planes and buckle upper floor strata. Floor failure induced by multiple blocks sliding within the floor can occur when weak bedding planes shear 1-3m below floor level and the sub-vertical fractures split the floor at regular intervals. The floor blocks interact at the corners and can induce large lateral stress at the floor level. If stress exceeds floor strength, shear failure at floor level can be expected, manifesting itself as floor heave, which may interfere with mining operations.

Stress concentrations at the longwall corners depend on the magnitude and direction of the pre-mining stress. At these corners, stone floor buckling is often experienced. Weak bedding planes within the floor can exacerbate this type of floor failure. Large goaf areas redistribute vertical stress that concentrates adjacent to goaf excavations. When the chain pillars that support the roof in the goaf are too small, their failure will transfer the vertical stress forward towards the longwall face where increased vertical load interacts with the floor, inducing complex floor failure.
A floor monitoring program was undertaken to investigate fracture formation and floor movement at the longwall face. The monitoring program indicated formation of near vertical fractures in the floor as well as bedding planes shearing well ahead of the face. These primary fractures were measured ahead of the face and provide the basis for the secondary floor failure mechanisms described in this thesis. Understanding fracture formation and their behaviour is very important in developing an analytical theory of floor failure. Analytical solutions were derived to calculate probable floor failure occurrence, which were then compared with modern numerical modelling to predict possible floor failure under a variety of conditions expected to occur in sedimentary strata.

Each of the described floor failure mechanisms is investigated in detail using an analytical and numerical approach. Towards the end of the thesis, a risk assessment procedure has been prepared with the floor failure mechanisms included, for the practising engineer to follow when investigating the possibility of floor failure in an underground mine.
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<td>A</td>
<td>Area of contact between blocks</td>
</tr>
<tr>
<td>$A_{\text{floor}}$</td>
<td>Area of floor</td>
</tr>
<tr>
<td>$b$</td>
<td>Depth of blocks (in third dimension)</td>
</tr>
<tr>
<td>$C_{EQ}$</td>
<td>Equivalent cohesion</td>
</tr>
<tr>
<td>$C_{\text{bedding}}$</td>
<td>Cohesion along bedding plane</td>
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<tr>
<td>$E$</td>
<td>Young's Modulus of elasticity</td>
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<tr>
<td>$F_s$</td>
<td>Force overcoming shear resistance</td>
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<tr>
<td>FOS</td>
<td>factor of safety</td>
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<td>FLAC</td>
<td>Fast Lagrangian Analysis of Continua</td>
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<td>$G$</td>
<td>Goaf resistance</td>
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<td>$h_i$</td>
<td>Height of force centroid $Q_i$</td>
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<td>Overburden depth</td>
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<td>$H_i$</td>
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<td>$I$</td>
<td>Second moment of area</td>
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<tr>
<td>$L$</td>
<td>Length of buckling beam</td>
</tr>
<tr>
<td>$L_i$</td>
<td>Width of block$_i$</td>
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<tr>
<td>$M$</td>
<td>Dimensionless parameter that depends on characteristics of structure within the rock</td>
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<td>$n$</td>
<td>Number of blocks</td>
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<tr>
<td>$N$</td>
<td>Capacity of longwall supports</td>
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<td>$N_c, N_q$ and $N_r$</td>
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<tr>
<td>$N_T$</td>
<td>Number of triaxial tests</td>
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<td>$P_i$</td>
<td>Vertical load applied on block $i$</td>
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<tr>
<td>$P_{Cr}$</td>
<td>Critical buckling load</td>
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<td>$R_i$</td>
<td>Reaction force below a stone block</td>
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<td>$q$</td>
<td>Uniform loading</td>
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<td>$Q_i$</td>
<td>Lateral force between block $i$ and block $i+1$</td>
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<td>$s$</td>
<td>Dimensionless parameter that depends on mechanical properties of rock such as weathering and fractures</td>
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<td>$S_h$</td>
<td>Shear resistance along failed horizontal bedding plane</td>
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<td>$S_v$</td>
<td>Shear resistance along failed vertical fracture</td>
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<tr>
<td>$T_0$</td>
<td>Tensile strength of rock</td>
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<td>UCS</td>
<td>Uniaxial compressive strength</td>
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<tr>
<td>UDEC</td>
<td>Universal Distinct Element Code</td>
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<tr>
<td>$V_i$</td>
<td>Force along vertical fractures separating stone blocks</td>
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<td>$w$</td>
<td>Longwall panel width</td>
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<td>$W$</td>
<td>Total load</td>
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<td>$W_i$</td>
<td>Weight of stone block $i$</td>
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<td>$y$</td>
<td>Floor lift</td>
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<tr>
<td>$Z$</td>
<td>Thickness of strong stratum</td>
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<tr>
<td>$\alpha$</td>
<td>Floor inclination from horizontal</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Density of rock/coal strata</td>
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<tr>
<td>$\Delta \sigma$</td>
<td>Stress deviator ($\Delta \sigma = \sigma_1 - \sigma_3$)</td>
</tr>
<tr>
<td>$\phi_{bedding}$</td>
<td>Angle of friction along bedding plane</td>
</tr>
<tr>
<td>$\phi_h$</td>
<td>Angle of friction along the horizontal plane</td>
</tr>
<tr>
<td>$\phi_v$</td>
<td>Angle of friction along the vertical fracture plane</td>
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\( \eta_0 \)  Stress distribution coefficient
\( \varphi_{\text{EQ}} \)  Equivalent angle of friction
\( \sigma_c \)  Uniaxial compressive strength
\( \sigma_{\text{ult}} \)  Ultimate strength
\( \sigma_{\text{all}} \)  Maximum allowable stress
\( \sigma_1 \)  Maximum principal stress
\( \sigma_2 \)  Medium principal stress
\( \sigma_3 \)  Minimum principal stress
\( \sigma_n \)  Normal stress
\( \sigma_b \)  Bearing capacity of rock
\( \sigma_T \)  Point of critical energy release
\( \tau \)  Shear stress
\( \tau_{\text{bedding}} \)  Shear stress along bedding plane
Chapter 1

1. INTRODUCTION

1.1 GENERAL BACKGROUND

This thesis presents a study of stone floor failure at the underground longwall coal mining face. The study is based on underground measurements, visual observations, analytical work and numerical modelling. Amongst many methods of mining, underground longwall mining is commonly used to mine tabular deposits of coal, because it offers high rates of coal extraction while winning up to 90% of the reserves, and provides one of the safest mining environments for the workforce.

Longwall mining consists of a large number of advancing hydraulic supports holding the roof strata at the edge of coal extraction while a coal cutting machine moves up and down the pillar edge cutting coal. Cut coal is carried on a steel armoured face conveyor to the maingate roadway to be loaded onto the conveyor belt and transported out of the mine. Powered supports control the edge of the caved overburden (goaf edge) and provide confinement stress to the fractured stone roof to minimise excessive rock displacements until roof caving occurs behind the supports. These supports transfer large loads onto the floor via the base pontoons. The method of longwall mining is depicted in Figure 1.
Economic coal reserves are typically located at depths not exceeding 500m below the surface, although some deeper mines exist where the strength of rock is high or economic mining is not feasible.

Coal production from a typical Australian longwall face can currently earn the mine in excess of A$1,000,000 dollars per day, depending on coal quality, coal seam geometry, strength of strata, magnitudes of stress, type of the equipment, and overall efficiency of the longwall operation. Among many
problems that may affect coal production rates are strata failure that can cause chronic deficiency in the expected rate of financial return of the mine. Therefore, it is sensible to maintain an ongoing strata control management program to minimise the risks associated with roof or floor failure.

To minimise disruption of production rates, most strata control design efforts concentrate on roof failure. Even though floor failure does not occur often, under certain conditions, it can cause serious operational problems. Current understanding of floor failure is limited and may not provide solutions to this problem. This thesis intends to advance the knowledge of floor failure in the longwall area and provide the geotechnical engineer with tools to combat the problem.

Floor failure often inhibits longwall mining operations where large displacements of floor strata known as floor heave, interfere with an armoured face conveyor and powered supports. The extent of floor failure depends on rock type and the magnitude of stress concentrations ahead of the face. Floor failure can be divided into primary and secondary failure. Primary floor failure occurs in response to stress concentrations ahead of the longwall face, while secondary failure occurs as stress redistribution and displacements take place at or behind the face line after the floor is exposed.
Primary Floor Failure at or ahead of the Longwall Face

Primary floor fractures induced by high stress concentrations ahead of the longwall face propagate through virgin rock in response to the advancing face. The extent of primary floor failure usually depends on rock strength, bedding strength and the state of ground stress. Although floor failure at longwall faces has been traditionally associated with a weak floor, this study shows that floor failure can be observed in strong rock where weak bedding planes predominate. Primary rock failure that occurs ahead of the face appears to be of a periodic nature, developing continuously or periodically due to stress build up as the coal is cut, which under certain conditions develops into secondary floor failure characterised by large floor displacements. Even though primary failure does not directly cause floor heave, in many instances it does define how secondary floor displacements occur.

Secondary Floor Failure

Stress fields and displacements constantly change as mining progresses. Initially, floor strata are subject to high abutment stress during which primary fractures tend to occur, but as coal is mined, stress relief follows and unloaded floor strata may experience displacements. At this phase of mining, secondary floor failure often occurs that may consist of significant bending and buckling driven by localised stress concentrations and displacements. Secondary floor failure usually occurs at a later stage of mining when strata
movement is reactivated along pre-existing (primary) fracture surfaces. Under certain conditions, these displacements can lead to excessive floor heave that often disrupt coal production. Secondary floor failure is defined as rock failure that develops after primary fractures occur, it is usually influenced by lithology, bedding planes, face geometry, powered hydraulic supports, or modified by either primary failure or subsequent strata displacements.

For secondary failure to develop, there must be a driving force within the floor capable of inducing large strains because once initial fractures develop, secondary rock failure follows, and strata deformation continues. In many instances where rock failure propagates along horizontal bedding planes and near vertical fractures, numerous beams, or blocks, form which interact with each other along cohesionless surfaces. Continuous deformation will bend strata and induce tensile, compressive and shear stresses within the fractured rock. Once critical stresses are generated, fractured rock can break into smaller components until a stress/strength equilibrium is reached. During this process, wedging, bending, buckling and hinging may occur resulting in large strains and bulking of broken rock that can seriously influence production rates.

Observed secondary types of floor failure at the longwall face are usually caused by:

- A lateral displacement of yielded coal seam towards the goaf where coal movement generates large lateral stresses within the immediate floor. If
weak bedding planes are present below the floor, the upper floor may buckle,

- Excessive stress at floor level induced by interaction among stone blocks that were formed during primary floor failure,
- Development of tensile breaks due to strata bending,
- Shear displacement along broken material,
- Shear failure along bedding planes,
- Compression failure of thin bedding planes within the floor strata,
- Floor buckling of thin floor layers in stratified rock (usually close to the floor surface),
- Hinged failure of fractured floor consisting of bedded rock,
- Wedging of sheared rock causing floor heave,
- Opening of fractures, and
- Complex combination of any of the above failures.

Floor failure at a longwall face appears to be progressive in nature. Both primary and secondary floor failures appear to be cyclic where floor failure initiates either after every shear is cut, or less frequently, as stress build up occurs. Many types of these floor failures have often caused severe production losses. These problems have been neglected for far too long and there is a need to provide the coal mining industry with rational solutions to minimise these problems.

Even though floor movement in fractured rock can be very complex, the proposed post failure models given in this study can be used to evaluate the
likelihood of floor failure. In summary, this thesis presents the principles of primary and secondary floor failure mechanisms which are poorly understood at present. This research work is based on field measurements, observations, analytical work and numerical modelling. A practical approach to estimate risks involved with this type of failure is explained towards the end of the thesis.

1.2 OBJECTIVE OF THE THESIS

The objective of this thesis is to provide new knowledge on floor failure mechanism at the longwall coal mining face. The aim is to differentiate between well known foundation failure of weak floor, and floor failure in strong strata with weak bedding planes, that have not been addressed in the past. Several mechanisms of floor failure proposed in this thesis deviate significantly from the traditional idea that floor failure is connected to floor bearing capacity that can be tested directly by placing heavy loads onto the floor. It is true that very weak floors can fail when loaded from above, however, weak bedding planes within floor strata are often only contributors to floor failure in otherwise strong and competent strata. The progressive nature of longwall mining presents challenging differences between the conventional approach to calculating the safety factors of floor failure and an estimation of actual strata behaviour that may occur as dynamic stress changes take place close to the coal edge at the longwall face. This thesis was
structured to provide the engineer with the tools necessary to evaluate the risk of mining in variable floor strata.

Even though the objective to describe some of the complex and variable floor failure mechanisms has been fulfilled, the author would like to encourage ongoing research in years to come to further advance the knowledge of floor failure at the longwall face.

1.3 ORGANISATION OF THE THESIS

This thesis has been divided into 8 chapters, each representing an important part of the overall floor failure mechanisms that can occur at the longwall face.

Chapter 1 introduces the thesis topic in summary form and outlines the importance of this study. Literature review in Chapter 2 describes up-to-date knowledge of floor failure mechanisms and how they are currently analysed. The author has attempted to discuss much of the relevant literature but it is noted that available literature in floor failure analysis is very limited.

Chapter 3 presents underground measurements and observations of progressive floor failure at and ahead of the longwall face that have been measured in the mining roadway driven towards the operating longwall. Various instruments including sonic extensometry, strain-gauged shear strip instruments and several observation holes were used to measure displacements
of floor strata during the approach of the longwall face. This data forms the basis for the analyses that has been developed within this thesis.

Chapter 4 describes the mechanisms leading to floor failure at the face. This chapter uses the "global approach" to explain how stress develops at a typical longwall face. On the basis of stress regime, primary fracture mechanisms in the mining floor are proposed. These mechanisms are discussed along underground measurements and the computational numerical model that was formulated to validate results.

Chapter 5 explains how an excessive yield of coal can expand and buckle bedded floor strata despite the floor being strong and competent rock. If the coal-floor interface is strong and a weak bedding plane exists below the floor, coal movement towards the goaf may shear the bedding plane. Pinning action below the powered supports will restrict upper floor movement and buckling can occur. This failure will manifest itself as floor heave ahead of the powered supports, a condition that can seriously affect longwall production.

Chapter 6 presents an analytical model of floor failure mechanism at a longwall face based on the multiple sliding block model. Under certain conditions, lateral and near vertical fracture planes will develop ahead of the face forming stone blocks within the floor. These blocks displace during mining, interact at the fractured surfaces and concentrate large stresses at the floor surface, often resulting in floor failure which, if large enough, seriously disrupt mining operations. The analytical solutions derived for the moving
stone blocks within the floor are supplemented with numerical models (FLAC) that confirm such movement takes place. The stresses generated at floor level are then compared with both the analytical and computational solutions.

Chapter 7 summarises the types of known floor failure mechanisms, explains the methods required to identify them and gives guidelines to assist the geotechnical engineer with the risk assessment of floor failure. Failure mechanisms are based on the overall content of this thesis. An illustrated example of risk assessment is given at the end of the chapter where a step by step procedure identifies possible floor failure mechanisms of various floor types, evaluates level of risk and suggests remedial actions.

Chapter 8 presents conclusions that have been drawn on the basis of this doctoral study. Evaluation of floor failure mechanisms has been unified to form a comprehensive package for the mining industry. The conclusions highlight the benefits of this work, establishes a new approach to risk assessment of floor failure and outlines recommendations for future work to encourage further research in this important field.
Chapter 2

LITERATURE REVIEW

2.1 INTRODUCTION

Traditionally, floor failure at a coal mining longwall face has been associated with rocks of low bearing capacity, where failure was thought to occur due to floor puncture below the base of powered supports, or below an excessively loaded coal pillar edge. Floor failure mechanisms were taken from soil mechanics principles where the probability of failure in a weak floor was approximated using standard equations for the bearing capacity of soils. Equations dealing with the bearing capacity of the foundations work well when dealing with very weak floor strata, but, may not be applicable for failure in a strong, bedded floor. Historically, the bearing capacity of stronger rocks or bedded rocks are based on similar assumptions, where the vertical loads exceed floor strength, resulting in floor failure. These predictions rarely matched field observations where floor failure often occurred in strong rocks that were considered adequate to withstand the expected loading.

The factors that influence floor stability are numerous. Some are related to the rock strength criterion where parameters such as floor moisture content or rock sample size versus its strength must be considered, while others are
totally independent of floor properties. For example, parameters such as depth of cover, longwall layout and roof strength (that determines the magnitude and location of peak abutment stress ahead of the longwall face) must also be considered. Most reports on floor instability in underground coal mines relate the problem to three principal mechanisms of floor failure. These include: (a) pillar punching into the floor strata due to foundation collapse, (b) buckling of the upper floor layers provoked by high horizontal stresses and (c) swelling of floor rock when exposed to moisture (Santos, 1989).

Foundation failure mechanisms used for longwall floor failure include:

- Failure of a very weak rock described by equations for bearing capacity of soils,
- The influence of geological structures on floor failure,
- Floor failure of bedded rock,
- Brittle floor failure of rock, and
- Buckling of a bedded floor.

2.2 CRITERIA OF ROCK MASS STRENGTH

The bearing capacity of a mine floor is one of the most important investigations that must be undertaken to assess the stability of floor strata in
underground mines. A strength criterion for rock mass can be used to establish the maximum allowable stress on the mine floor. There have been several failure criteria reported in the technical literature such as Coulomb, (1773), Murrell (1965) and Hoek and Brown (1980). Since an analytical solution of determining the strength of rock mass is difficult due to the complex nature of stratified rock, most criteria proposed so far are of an empirical origin.

The Hoek and Brown (1980) strength criterion for rock mass is:

\[ \sigma_1 = \sigma_3 + \sqrt{m \sigma_c \sigma_3 + s \sigma_c^2} \]

where \( \sigma_1 = \) major principal stress at failure

\( \sigma_3 = \) minor principal stress at failure

\( \sigma_c = \) uniaxial compressive strength of rock

\( m = \) dimensionless parameter that depends on characteristics of structure within the rock

\( s = \) dimensionless parameter that depends on mechanical properties of rock such as weathering and fractures.

According to Hoek and Brown (1980), the parameter \( m \) varies with the lithology and angle of internal friction, while the parameter \( s \) depends on the tensile strength of rock. These parameters are independent and reflect the geotechnical quality of the rock mass. For intact rock the parameter \( m \), can be
computed from the triaxial test data or derived from the table for different rock
types presented by Hoek (1986). The parameter $m_i$ can be calculated using:

\[ m_i = \frac{\sum (\Delta \sigma^2 \sigma_3) - \frac{\sum \Delta \sigma^2 \sigma_3}{N_T}}{\frac{\sum \Delta \sigma^2 - (\sum \Delta \sigma)^2}{N_T} \sigma_c} \]  

(2.1)

where $\Delta \sigma = \text{stress deviator}$,

$N_T = \text{number of triaxial tests}$

and $\Delta \sigma = \sigma_1 - \sigma_3$  

(2.2)

It has been reported (Santos, 1989) that the use of Equation (2.1) may sometimes yield unrealistic results, particularly for weaker rocks. An explanation for that fact is that the influence of the angle of internal friction can be higher when low confining pressures are used during the triaxial testing regardless of the geotechnical quality of the rock.

2.2.1 Shear Strength Parameters

The shear strength parameters summarised by Santos (1989) indicate three independent methods of derivation. Bray (in Hoek, 1983), Ucar (1986) and Londe (1988) have worked independently on the determination of shear strength parameters of rock masses based on Hoek and Brown’s criterion. The determination of the ultimate shear strength given by Equations (2.3),
(2.4), (2.5) and (2.6) is a combination of three methods with special consideration to the floor layers and time dependent rock behaviour. The shear strength parameters are expressed in terms of equivalent cohesion and an equivalent angle of internal friction which are parameters of a linear envelope tangent to the Mohr-Coulomb failure criterion for a given pair of principal stresses. The shear stress is expressed as:

\[ \tau = mT_0 \left( \frac{1}{\tan \varphi_{EQ}} - \cos \varphi_{EQ} \right) \]

where

\[ \varphi_{EQ} = \tan^{-1} \left( \frac{1}{\sqrt{4\lambda \cos \left( \frac{\pi}{6} + \sin \frac{\zeta}{3} \right)}} - 1 \right) \]

for

\[ \lambda = \left( 1 + \frac{16m \sigma_N + s \sigma_T}{3m \sigma_T} \right)^3 \]

and

\[ \zeta = \frac{1}{\sqrt{\lambda^3}} \]

where \( \sigma_N \) = effective normal stress,

\( \varphi_{EQ} \) = equivalent angle of friction

\( \sigma_T \) = point of critical energy release

\( T_0 \) = tensile strength
If maximum compressive stress occurs in the horizontal direction and the stress deviator is high, then the issue of floor instability becomes the instability of the mine roadway itself rather than a typical foundation problem.

The value of the equivalent cohesion is taken as twice the tensile strength (Hoek and Brown, 1980) given by:

\[ C_{EQ} = 2T_0 \]  \hspace{1cm} (2.7)

Figure 2.1 shows the equivalent shear strength parameters in the failure criterion.

![Figure 2.1](image.png)  
*Figure 2.1 Equivalent cohesion and angle of internal friction for Hoek and Brown’s strength criterion (after Santos, 1989)*
While the cohesion remains fixed irrespective of the stress level, equivalent angle of internal friction is inversely proportional to the normal stress. In general, a higher normal stress would reduce the angle of internal friction.

2.2.2 Ultimate Stress on the Floor Strata

As reported by Santos (1989), the value of the ultimate strength of the rock can be determined once the shear stress on the floor strata is known. It can be also expressed as a curve between the two principal stresses in terms of parameter $m$ and the equivalent angle of internal friction of the rock mass. The ultimate stress that corresponds to the floor strength as a foundation member can be calculated using:

$$
\sigma_{ult} = mT_0 \int \left( \frac{1}{\sin^2 \varphi_{EQ}} - \sin \varphi_{EQ} \right) \frac{\partial \varphi_{EQ}}{\tan \varphi_{EQ}}
$$

(2.8)

that translates to:

$$
\sigma_{ult} = mT_0 \left( \frac{1}{2 \sin^2 \varphi_{EO}} + \sin \varphi_{EO} \right) + K
$$

(2.9)
The constant $K$ is associated with the boundary conditions of the Mohr shear strength envelope shown earlier in Figure 2.1, where the envelope becomes orthogonal to the normal axis. In that case the angle of internal friction approaches 90° and can be mathematically represented by:

$$\varphi_{EO} \to 90^\circ, \quad (2.10)$$

$$\tan \varphi_{EO} \to \infty, \quad (2.11)$$

$$\lim \sigma_3 \to \frac{\sigma_T}{m} s, \quad (2.12)$$

$$\frac{\sigma_T}{m} s = m T_0 (\varphi_{EO} + 1) + K, \quad (2.13)$$

and

$$K = \sigma_T \left( \frac{3m - s}{16m} \right) \quad (2.14)$$

The term $\frac{s}{m}$ can be neglected in floor analysis because of its low numerical value. The $K$ value can also be approximated using:

$$K = \frac{m \sigma_T}{5} \quad (2.15)$$

Finally, the ultimate stress on the floor can be calculated for any confining stress if the parameters $m$ and $s$ are known. The equivalent angle of internal friction can be computed from Equation 2.4. The maximum allowable stress
is equal to the ratio of the ultimate axial stress and a chosen safety factor where:

\[ \sigma_{ult} = mT_0 \left( \frac{1}{2\sin^2 \varphi_{EQ}} + \sin \varphi_{EQ} \right) + \frac{m\sigma_T}{5} \]

and

\[ \sigma_{all} = \frac{\sigma_{ult}}{FS} = \frac{1}{FS} \left\{ mT_0 \left( \frac{1}{2\sin^2 \varphi_{EQ}} + \sin \varphi_{EQ} \right) + \frac{m\sigma_T}{5} \right\} \]

The angle between the failure plane and the direction of the major compressive stress (usually horizontal) can be determined from Figure 2.1 where this angle \( \alpha \) varies with the increase of principal stresses. The geometrical relation between the angle \( \alpha \) and the equivalent angle of friction can be described by:

\[ \alpha = 45^\circ + \frac{\varphi_{EQ}}{2} \]

According to Hoek (1988) the parameters \( m \) and \( s \) are related to the rock mass rating (RMR) of the Geomechanics Classification (Bieniawski, 1979). The equations that show this relation have been introduced by Brown, (1983):

\[ m = m_i^{(RMR-100)/28} \]

\[ s = e^{(RMR-100)/9} \]
It should be noted that the RMR values exclude adjustments related to the orientation of the discontinuities (Hoek, 1986). Pore pressure should not be used in calculations and an appropriate rock mass strength criterion has to be applied to both isotropic and anisotropic rock masses. This makes the adjustment for orientation of discontinuities unnecessary.

2.3 GUIDELINES FOR FLOOR DESIGN BASED ON FLOOR BEARING CAPACITY.

The mining floor design described in many publications is usually based upon the rock mass strength criterion such as the one described in the previous section. The safety factor based on the floor bearing capacity is applicable to the roadway design and is also used to predict floor stability below the longwall powered supports. Mine roadways adjacent to large goaf areas where vertical stress concentrations are likely to occur, do need to be investigated. Extensive numerical modelling work has been undertaken by many researchers such as Hsiung, (1986) to quantify vertical stress distribution adjacent to pillars and goafs. Special attention must be given to the area in the gate roadways adjacent to the longwall face where high lateral stress concentrations often occur. Mark (1987) and more recently Gale (1996) presented the determination of pillar load, which influences the stability of the floor in gate roadways. Santos (1989) devised a step by step
method for the analysis of floor stability that is summarised in the flow chart and presented in Figure 2.2.

Figure 2.2  Flow chart procedure for mining floor design (after Santos, 1989)
2.3.1 Failure of Very Weak Floor at the Longwall Face

When a longwall powered support is set against the roof and floor, loads ranging between 600-1100 tonnes may be applied to the floor. Uneven loading conditions often occur below the support base and if the floor rock is weak, the supports may puncture the floor causing heave just ahead of the longwall supports.

Floor failure at the longwall face was assumed to occur either below the highly loaded coal face edge or below the base of the hydraulic longwall support. A typical hydraulic support standing at the longwall coal face would carry maximum loads of up to 1100 tonnes. The pontoon area at the support base varies and is approximately 3-5m², and the average floor pressures below the support range up to 3.3 MPa. Under normal operating conditions the distribution of support pressure below the base is similar to that shown in Figure 2.3a, however, during extreme roof loading conditions, canopies often tip causing the front toe of the hydraulic supports to exert very high pressures onto the floor (Figure 2.3b). If the floor is weak the toe of the powered support penetrates the floor and causes heave (Figure 2.4).

The pressure exerted by the toe of the hydraulic base can be in excess of 6 MPa, a value that exceeds the strength of many weak strata in coal measures.
Base of the 980 tonne 2 Leg Powered Hydraulic Supports

Pressure distribution under normal operating conditions

(a)

Stress distribution below base during adverse loading conditions (no lateral load at yield)

5.7 MPa

(b)

Figure 2.3 Typical floor loading distribution below the 2 leg powered supports under (a) normal operation and (b) adverse loading conditions

If the toe of the supports penetrates the floor surface, a localised floor failure would occur. Most modern hydraulic supports are equipped with hydraulic lifting jacks designed to lift the toe of the base before the support is advanced forward, thus reducing floor puncture problems during face advance. The strength of very weak floor strata can be estimated using the bearing capacity of soil foundations that are described in many standard soil mechanics text
Figure 2.4  Puncture of weak floor below the longwall support base.

books. The theory assumes that the bearing capacity of weak rock mass should be similar to that of soil because their Mohr envelopes are similar (Sowers, 1979). The bearing capacity of rock is controlled by local shear that typically develops at the perimeter of the loading area. There are usually three stages in weak floor failure:

- Weak rock beneath the loading area is sheared in the shape of a cone or a wedge and forced downward (Figure 2.5a), forcing the weak rock below the wedge outward away from the loaded area,

- Weak rock around the foundation perimeter moves away from the loading area and the shear surfaces begin to propagate from the tip of the wedge as shown in Figure 2.5b,
a) Stage of elastic distortion

b) Stage of local shear and cracking (after Vesic and Berezantzev)

c) Stage of general shear failure (after Terzaghi)

Figure 2.5 Development of shear failure in weak floor (after Sowers)

- General failure reaches the surface adjacent to the loaded surface and floor heave occurs (Figure 2.5c).

A number of foundation failure equations have been developed and can be used to estimate the strength of the weak floor. Most popular are Terzaghi’s bearing capacity equations that can be used to calculate strength of the mine floor (rough footing) and are discussed below:

For a square base:
\[
\sigma_b = 1.3CN_c + qN_q + 0.4yBN_r
\]  

(2.21)

For a rectangular base:

\[
\sigma_b = \left(1 + 0.3\frac{B}{L}\right)CN_c + qN_q + \left(1 - 0.2\frac{B}{L}\right)\frac{B}{2}N_r
\]

(2.22)

where:

\( \sigma_b \) = Bearing capacity of rock

\( C \) = Cohesion of the floor rock

\( \gamma \) = Density of floor rock

\( B \) = Width of the pressure footing

\( L \) = Length of the pressure footing

\( \phi \) = Angle of the internal friction

\( q \) = uniform loading on both sides of the base

\( N_c, N_q \) and \( N_r \) = Bearing capacity factors

Values of the bearing capacity factors are given in Table 2.1 below.

<table>
<thead>
<tr>
<th>( \phi )</th>
<th>( N_c )</th>
<th>( N_q )</th>
<th>( N_r )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>5.7</td>
<td>1.0</td>
<td>0.0</td>
</tr>
<tr>
<td>5</td>
<td>7.0</td>
<td>1.6</td>
<td>1.14</td>
</tr>
<tr>
<td>10</td>
<td>9.5</td>
<td>2.7</td>
<td>0.7</td>
</tr>
<tr>
<td>15</td>
<td>13.0</td>
<td>4.5</td>
<td>2.0</td>
</tr>
<tr>
<td>20</td>
<td>17.0</td>
<td>7.5</td>
<td>4.8</td>
</tr>
<tr>
<td>25</td>
<td>24.0</td>
<td>13.0</td>
<td>9.8</td>
</tr>
<tr>
<td>30</td>
<td>37.0</td>
<td>23.0</td>
<td>20.0</td>
</tr>
<tr>
<td>35</td>
<td>58.0</td>
<td>42.0</td>
<td>43.0</td>
</tr>
<tr>
<td>40</td>
<td>98.0</td>
<td>77</td>
<td>98.0</td>
</tr>
</tbody>
</table>
When considering the cross-section across the longwall face, the bearing capacity factor $q_N$ can be ignored.

If the immediate floor is a strong stratum of thickness $Z$ underlain by soft rock, the applied load is spread over a larger area, thus reducing the bearing pressure on the weak stratum, where strong stratum at floor level would increase the bearing capacity of the floor. The strength of the floor below the base of a rectangular powered support can be calculated using Equation 2.23 (Peng, 1984) below:

$$\sigma_b = \left[ \left( 1 + 0.3 \frac{B}{L} \right) C N_c + \gamma_1 Z N_q - \gamma_1 Z + \left( 1 - 0.2 \frac{B}{L} \right) \frac{\gamma_2 B}{2} N_r \right] \frac{1}{\eta_0}$$  \hspace{1cm} (2.23)

where:

$\gamma_1$ = Density of strong floor stratum

$\gamma_2$ = Density of weak floor stratum

$Z$ = Thickness of strong stratum

$\eta_0$ = Stress distribution coefficient given in Figure 2.6
Figure 2.6 Nomograph for determining the stress distribution coefficient (after Peng, 1984).

Other equations that can be used to calculate strength of soil foundations are not described here, but they can be accessed in many soil mechanics books dealing with foundation design.
2.4 INFLUENCE OF GEOLOGICAL STRUCTURES ON FLOOR FAILURE

2.4.1 Jointed floor

If a weak layer of rock is present below the stronger floor surface, then several modes of floor failure can occur depending upon geological structures present within the floor. If the rock is jointed, the floor failure mechanism can differ from simple shear failure. The size of the loaded area, joint spacing, joint opening and location of the load with respect to the joints can influence floor strength, with possible cases presented in Figures 2.7a, to c. In the first case (Figure 2.7a), where open joint spacing is a fraction of the load bearing area B, the foundation area is supported by an unconfined rock column. In this case, floor strength would have a value of an in-situ uniaxial compressive strength (UCS) of the weakest layer within the floor rock.

If joints are closed so that pressure can be transmitted across them without movement (Figure 2.7b), floor failure mechanism is identical to the shear failure described by Bell-Terzaghi analysis (Sowers, 1979). The cone shaped zone that forms below the foundation (Figure 2.7c), splits the block of rock formed by the joints. This floor failure was first analysed by Meyerhof (1956) and extended by Sowers (1979). When floor strata consists of a strong and rigid upper layer and a weak layer below, two types of floor failure can occur depending on the thickness of the rigid floor portion (H) and the width of the
fractured/jointed floor (S). If the thickness of the rigid layer is larger than the joint/fracture spacing (S), the floor flexure failure will occur as shown in Figure 2.7d. If the thickness of the upper rigid floor is lower than the load bearing area, floor-punching failure (Figure 2.7e) is more likely to develop. A narrow soft zone within the rigid floor will not seriously reduce floor capacity.

Figure 2.7 Bearing capacity of the rigid floor overlying weak strata (after Sowers, 1979)
2.5 INFLUENCE OF BEDDING PLANES ON FLOOR FAILURE

Sedimentary rocks are usually non-homogeneous with properties that vary greatly when tested at different angles to the bedding planes (Gale, 1996). In general, sedimentary rock is weaker along the bedding planes while their strength increases in other directions. When testing rock samples, the failure plane in many instances coincides with the maximum shear direction that develops at an angle of $\pi/4 - \phi/2$ from the sample axis where $\phi$ is the angle of the internal friction. If the angle of internal friction varies from 20° to 40° then the plane of maximum shear would be between 25° to 35° from the direction of stress application. When the direction of maximum compressive stress is within 25° to 35° from the bedding planes, the maximum shear stress develops along the bedding planes. Failure along the bedding planes is a common occurrence when subject to high lateral stress.

Physical models constructed by Jiang Yaodong (1993) indicate typical floor failure mechanisms in bedded strata as shown in Figure 2.8.
2.6 INFLUENCE OF HIGH LATERAL STRESS ON MINE FLOOR FAILURE

Many advances in data analysis obtained from field monitoring of stress and ground behaviour during mining operations have been made over the past 10-15 years. Underground stress measurements in Australia (Nemcik, 1998) and other parts of the world (Fairhurst, 1986) indicate the maximum principal stress $\sigma_1$ is predominantly horizontal and increases with depth of cover, whereas dominant source of lateral stress is attributed to plate tectonics (Cox, 1973). In most Australian mines, both principal stresses ($\sigma_1$) and ($\sigma_2$) are usually horizontal, while the smallest stress ($\sigma_3$) is normally vertical. The horizontal stresses in the ground are typically very directional. The major
principal stress ($\sigma_1$) in Australia (Nemcik, 1998) is often oriented in the general direction of continental movement while at times geological structures or topography can change stress direction.

To study a general case, we can assume that when the roadway is mined, the tangential stress about the opening would increase while the radial stress at the excavation surface would diminish to zero. Prior to mining, the rock is subject to triaxial stress loading where its strength under these conditions can be represented by Mohr-Coulomb criterion (Brady and Brown, 1985) given by:

$$\sigma_{crit} = \sigma_{ucr} + \sigma_3 \frac{1 + \sin \phi}{1 - \sin \phi} \quad (2.24)$$

where:

- $\sigma_{crit}$ = critical stress at failure of triaxially loaded rock
- $\sigma_{ucr}$ = critical stress at failure of uniaxially loaded rock
- $\sigma_3$ = confining stress (minimum principal stress)
- $\phi$ = angle of internal friction
When excavating the mine roadway, the radial stress at the excavation surface would decrease to zero and in most cases would become the lowest compressive stress ($\sigma_3$). In other words, the confinement stress at the surface is lost and the equation 2.24 would become $\sigma_{crit} = \sigma_{uct}$. If the maximum compressive stress $\sigma_1$ exceeds $\sigma_{uct}$, failure would occur. Measurements of strata movement in a failed roof, rib, and floor (Gale, 1987) adjacent to a typical rectangular roadway, indicates that the failure zone around the roadway takes an oval shape (Figure 2.9).

![Figure 2.9](Figure 2.9) Typical stress relieved zone about the rectangular roadway
Generally, horizontal stress plays a major role in roof and floor strata stability, whereas vertical stress has a major effect on ribside and pillar deformation. Pillar behaviour can significantly influence the stress state of roadways should loads be sufficient to cause the roof and floor materials to fail. Roadways aligned parallel to the maximum horizontal stress attract least horizontal stress across the roadway and suffer the least amount of roof and floor deformation. Roadways aligned perpendicular to the maximum horizontal stress attract maximum horizontal stress across the roadway, hence, suffer the greatest amount of roof and floor deformation. Roadways aligned at an acute angle to the maximum stress field suffer roof and floor deformation bias to that side of the roadway where stress concentrations occurred. The percentage of strata failure about the roadway increases with the angle of maximum principal stress to the direction of the roadway driveage. This is depicted in Figure 2.10 where measurements at two specific underground sites indicate that roadways driven at angles greater than 35-45° to maximum stress experience shear failure in the roof and floor. This failure is most common in roadways driven at a high angle to the maximum horizontal compressive stress.
Longwall panels are essentially wide roadways with surrounding stress concentrations similar to stress concentrations about the roadway face during its advance. Monitored stress concentrations and relief about longwall panels (Siddall, 1992) are shown in Figure 2.11. Predicting the general deformation behaviour of gate roadways is important in assessing the requirements of roadways driven in different directions. This directional influence on roof and floor failure is very important in optimising the mine layout. Gate roadways aligned with maximum horizontal stress will provide the most stable roof and floor conditions that require least reinforcement and maintenance during its lifetime. If gate roadways are aligned with the maximum horizontal
stress, then cut-through roadways, joining the maingate, service roadways, and the longwall installation roadway itself, would be exposed to full lateral stress.

It would be expected that in these roadways roof and floor damage would be greater than in the maingate or service roadway.

Figure 2.11 Stress Distribution about the longwall panel (Siddall, 1992)
Horizontal and vertical stress changes occur about longwall extraction panels in response to coal removal and deformation of caved zones. Stress monitoring has established a general concept of stress redistribution in which horizontal stress is redirected about the goaf area rather than wholly transferred through cracked and caved ground. Vertical stresses are redistributed within the solid coal pillars and within the goaf area depending upon extraction geometry. The stress field is influenced by proximity to other excavated areas and caved zone geometry. Stress field in the vicinity of gate roadways can vary over time due to mining activity, yet still be directly related to the mining layout, therefore roadways must be able to cope with variable stress induced deformation resulting from mining activities.

Stress concentrations that occur about the mine opening induce shear, bedding shear or tension fractures in rock. The type, orientation and severity of fractures depend on the mine opening, magnitude and direction of stress. In high lateral stress fields, floor failure can occur that may lead to excessive floor heave. This failure is commonly seen about the corners of moving longwall panels where high lateral stress concentrations are often experienced. Stress change measurements at longwall corners indicate stress concentrations of up to 2.5 times virgin (pre-mining) stress (Matthews, 1992). If floor heave occurs below the stage loader (Maingate roadway), it becomes very difficult to control. In such situations, floor failure tends to occur continuously just ahead of the moving longwall face and can interfere with a permanently installed stage loader and conveyor belt.
2.6.1 Stress Relief by Sacrificial Roadways

Excavated roadways have a failed zone around them that is incapable of carrying substantial stress. In the first driven roadway where large roof, rib and floor failure is experienced, the failure zone that cannot sustain high stress is relatively large. In a typical coal mine, this zone may be of up to 10m in diameter (Figure 2.9). Numerical modelling (Gale, 1987) shows that the rate of lateral stress relief increases with distance towards the roadway. The effectiveness of lateral stress relief is strongly dependent upon the height of softening (Figure 2.12) and upon rock mass properties. (Figure 2.13). In the homogeneous strata or thick coal seam, the effective lateral stress relief (less than 70% of the virgin stress) occurs within 15-20m of the roadway. In stratified rock, where weak bedding planes predominate, the lateral stress is relieved over a much longer distance. These conditions are commonly satisfied in coal measure sequences where weak bedding planes of low shear stiffness are present.

Stress relief may be offered by failed sacrificial roadway to any adjacent roadways driven at a later time. This method is often used to protect roadways driven at a high angle to maximum lateral stress. As discussed previously, longwall installation roadways are often driven at a high angle to the major horizontal compressive stress, hence, protection is often needed.
Use of a sacrificial roadway is depicted in Figure 2.14 where the first driven sacrificial roadway is often widened and left to collapse to achieve a much
larger stress relief effect. The longwall installation roadway is then driven parallel to the sacrificial roadway within the stress relieved envelope. Good roof and floor conditions prevail despite a typical installation roadway width of 7-10m.

2.7 STABILITY OF CLAYSTONE FLOOR STRATA

Stability of the claystone floor is directly related to the stability of coal pillars resting on the floor. The mechanisms contributing to claystone floor failure are related to fracture development within the floor, expansion of the unconfined coal pillar resting upon weak claystone, and the swelling factor of fractured claystone. These mechanisms are time dependent and influenced by moisture contained within the floor. It has been reported (Mills, 1997) that for medium to large sized pillars, pillar strength value is effectively halved by the presence of claystone floor strata. 25 pillars resting on a claystone floor were studied using the subsidence method (Mills, 1997). Bieniawski’s coal pillar strength formula (Bieniawski, 1984) was used to calculate the stability index for each pillar using known pillar geometry based on width, length and height. Most pillars were located in the supercritical goaf environment where the size of the adjacent waste area was large enough to assume an infinite goaf. The overburden weight carried by the pillar was calculated using subsidence geometry. The ratio of the calculated pillar strength (using Bieniawski’s formula) to the calculated overburden load was used as a
Figure 2.14 Principle of the stress relief roadway used to protect longwall installation roadway
stability index to compare the long term pillar behaviour above the clay floor.

The stability index plotted against the surface subsidence is divided by the seam thickness, as shown in Figure 2.15.

The data plotted in Figure 2.15 divides into two main groups consisting of the failed pillars (subsidence scale value of approximately 0.3) and stable pillars with subsidence scale between 0 and 0.1. A significant characteristic of the grouping is that there are no examples from the stable pillars group that exist below a stability index of 2.0. By comparison, experience of pillar and subsidence monitoring of similar sized pillars in strong floor strata shows that a high proportion of pillars would remain stable at a stability index
approaching 1.0. Results of the stability index shown in Figure 2.15 confirm that pillar strength observed in the study are significantly lower compared to similar sized pillars under strong roof and floor conditions. For medium to large sized pillars, pillar strength is effectively halved by the presence of claystone floor strata.

A second outcome of back analysis study was the correlation between the width to height ratio of pillars standing on the claystone floor and final subsidence associated with pillar failure. Characteristic width to height ratio was taken to be the average width to height ratio of pillars in panels where significant surface subsidence indicated pillar failure. The results shown in Figure 2.16 indicates the data fell into two main groups, (a) a lower group representing intact pillars where the small subsidence is a function of elastic strata compression, and (b) an upper group representing failed pillars where subsidence is high. The few points plotted between these main groups comprise sites where subsidence is ongoing, which appears to be a transient state that may exist for a considerable time until the pillars gradually fail and full subsidence develops. The results in Figure 2.16 show that pillars in claystone floor strata with a width to height ratio of more than 12 are vulnerable to failure. The general concept of pillars with a width to height ratio of eight being indestructible does not apply to pillars in a weak geological environment.
Figure 2.16 Subsidence as a function of pillar width to height ratio (Mills, 1997).

Behaviour of Claystone Floor

Claystone materials recovered from the drilled core ranged from 'rock-like' through to materials that resemble soft clay. Strength tests of claystone presented in Figure 2.17 indicate dependence on moisture content (Mills, 1997). The results from laboratory testing indicate that claystone comprises a mix of expansive clay minerals encapsulated by a much stronger matrix of silicates and carbonates that gives the claystone its rock-like properties.
These rock-like properties prevail whenever the matrix remains intact. If the matrix is fractured, clay minerals within the matrix expand and demonstrate clay-like properties characteristic of degenerate claystone strata. The expansive clay minerals encapsulated within the matrix appear capable of exerting swell pressures in the 200-1000kPa range. In free swell, these minerals are typically capable of increasing in volume by some 5-10% depending on the particular mix of clay minerals. In their natural state in the ground, the claystone matrix and in situ stresses combine to counteract the swell pressures of expanding clay minerals. While the combination of matrix strength and in situ stresses are greater than swell pressure, the clay is
contained within the matrix and the claystone exhibits rock-like strength. When both of these confining influences are removed, the clay minerals are free to swell and the claystone takes on its degenerate clay-like properties. The confining effect of the rock matrix is removed when the claystone is fractured as a result of either a geological process or mining activity.

In general, the confining effect of stress is low in the vicinity of roadways and goaf areas. When lateral stresses are high, fractures followed by stress relief develop in the floor under the excavation. This is where floor heave and soft clay materials are commonly observed to develop some time after mining. A typical case of modelled claystone floor failure is shown in Figure 2.18. Under the pillars where the confining stresses are great and the rock is not fractured, the matrix remains intact and the claystone remains generally rock-like in its behaviour. The soft clay material develops only in areas where the matrix is fractured and the confining pressure is low enough to allow the clay materials to swell. Tests indicate that mechanical strength of the fractured clay material is negligible in comparison to the strength of the original intact claystone strata.

The numerical model of claystone floor failure in the Southern Lake Macquarie area (Mills, 1997) indicates the potential of claystone softening due to moisture directly under the roadway and pillar edge. Confining pressure developed further under the pillar appears sufficient to restrict claystone degeneration unless the pillars become overloaded. This is an area that
Figure 2.18 Numerical model of a typical failure in clay floor adjacent to the chain pillar (Mills, 1997).

requires further investigation. Primary failure mechanisms appear to be: (a) failure of the claysone rock material due to over-stressing, and (b) failure of bedding planes within the claystone. This failure of the claystone floor strata leads to loss of confinement on the pillar edges that reduces the coal strength. Figure 2.19 shows a profile of vertical stress developed as a function of the distance from the rib side. This profile of vertical stress is typically measured in strong roof and floor strata.
Figure 2.19  Vertical stress profile into the rib for various claystone sections and loading (Mills, 1997)

Figure 2.20 shows the pillar strength estimated from modelling and back analysis for a range of pillar width to height rations. The strength of claystone floor indicated by back analysis and computational modelling are very similar, and both are significantly less than the pillar strength observed in a strong roof and floor strata. Clearly, claystone floor strata has a very significant effect on pillar strength. Back analysis and numerical modelling suggest that it would be entirely inappropriate to consider using a pillar design formula for the claystone floor that was developed for a strong roof and floor strata. The
mechanism for failure is not a strong coal pillar punching into the soft floor strata, but rather the coal in the pillar being unable to develop full confinement due to reduced strength of claystone strata.

Figure 2.20 Comparison of pillar strength (Mills, 1997)

Deep yield zones that develop within coal pillars mobilise rib movement into the roadway opening, while simultaneously unloaded pillar edges encourage the claystone floor to fail, especially in areas of high horizontal stress. Swelling of the fractured claystone floor together with rib movement into the
roadway can induce large floor heave. Even though floor heave can be cut (brushed) away, it may reoccur at a later time, or when exposed to higher stress concentrations adjacent to the longwall face. If structures such as a stage loader and conveyor belts are placed in the maingate roadway, floor heave can interfere with their operation and seriously affect mine production.

According to Afrouz (1990), soft floor closure in gate roads depends on:

(a) Depth of cover and varying strata pressure,
(b) Dimensions and shape of the gate roads,
(c) Method of gate road support and its size,
(d) Direction of roadway advancement relative to the dip of the seam,
(e) Rate of advance,
(f) Geological factors, such as stratigraphy, discontinuities, cementation between the discontinuities, washouts and existence of the tectonic actions,
(g) Type of roadside packs, its dimensions and behaviour as a support,
(h) Barrier pillar, its dimensions and behaviour,
(i) Panel width of the face length
(j) Extracted height of the seam,
(k) Proximity of any mining activity to the gate roads,
(l) Methods of the roadway development and the production technique,
(m) Rock/support interaction properties,
(n) Non-uniform stress distribution in the support system
(o) Bearing capacity of the roadway floor,
(p) Type and quality of the support or reinforcement utilised along the roadway and behind the face,

(q) Support intervals along the roadways,

(r) Influence of water on the floor material,

(s) Physical, mechanical and time-dependent behaviour of the rocks surrounding the roadway,

(t) Influence of any ripping or dinting on the neighbouring supports, i.e. pillar, pack and steel arches, and

(u) Position of the roadways relative to the face line and its direction of advance (or retreat) e.g. advanced heading, behind the face line with ripping lip, or inline with the face.

Most of Afrouz (1990) work deals with the steel arches and prevention of the arches from penetrating the floor and panels with no coal chain pillars. The conclusions of this work include:

(a) Water increases the coal and underclay floor heave by 18-30%,

(b) Solid pillars left on one side of the gate roads contributes towards the asymmetrical floor heave, if the yielding characteristics of the goaf side packs is not similar to that of the pillar,

(c) Pillars left in the overworked area tend to transfer the strata pressure towards the floor of the underlying roadways at an angle between 50° and 66° to the horizontal.
(d) Goaf side packs have no appreciable effect on the floor heave unless they are packed very tightly and their compressive strength surpasses that of the floor,

(e) Installation of the steel base plates increases stability of the arched support and reduces penetration of the support legs into soft floor, and

(f) Floor reinforcement by injection is promising and lends itself to the theoretical and in situ analysis

2.8 SUMMARY

The wealth of information on foundation stability in civil engineering has been partially responsible for overlooking some of the differences characteristic of rock failure in underground mines. The up-to-date review of published knowledge on floor failure in underground coal mines indicates that most of the failure mechanisms are still attributed to pillar puncture of floor strata. Recent literature on pillar failure has proposed that even though the floor has a very high vertical bearing capacity, it can buckle due to expansion of the failed pillar. If weak bedding planes are present within the roof and floor, lateral movement of failed coal within the pillar can shear the bedding planes and buckle the roof and floor strata. This mechanism of roof and floor has been observed many times by the writer during the formation of small coal pillars at
depths greater than 200m, which suggests that this type of failure makes the yield pillar design doubtful.

Stress measurements conducted underground reveal that in many cases, the in-situ lateral stresses underground are greater than the vertical stress induced by overburden weight. Such observations led to investigations of roof and floor failure mechanisms in mines where high lateral stresses predominate. Numerous articles have been published on lateral stress concentrations about roadways and longwall corners, and the effect of stress direction with respect to mine roadways. Even though roof failure is discussed in detail, and the same mechanism applied to floor failure, the distinct differences between roof and floor failures are rarely analysed.

There have been numerous problems of floor failure along the longwall mining face, but no satisfactory explanations have been proposed to deal with these problems. Predominant problems related to roof failure along the longwall face have absorbed much of the past research effort and funding with limited attempts to study floor failure mechanisms. Although floor failure at the longwall face itself may not occur often, when it does, the impact on production losses is considerable.

The following chapters of this thesis present underground monitoring of floor failure ahead of the longwall face, examine several types of floor failure
mechanisms that may occur along the face, provide an overview of influencing factors on floor failure, and propose guidelines for geotechnical practitioners.
Chapter 3

UNDERGROUND MONITORING OF FLOOR DEFORMATION AT LONGWALL FACE

3.1 INTRODUCTION

This chapter presents underground measurements taken by the author of floor deformation ahead of a longwall coal mining face. The primary aim of these measurements was to develop an understanding of fracture formation and floor behaviour at the longwall face. Sonic extensometers, straingauged shear strip, observation holes and visual monitoring of the floor fractures were used to measure displacements in the floor, and to determine the mode of floor failure at the centre of the longwall face. Monitoring was specifically designed to determine how floor failure develops, and what fracture modes are contained within the rock mass below the stone floor.

Underground measurements and observations in stratified coal measures indicate that sub-vertical fractures and failure of bedding planes occur in the floor at the longwall face. Sub-vertical mining induced fractures appear to be oriented parallel to the face and spaced at frequent intervals, while bedding plane failure occurs ahead of the longwall face together with lateral floor movement towards the longwall goaf. Orientation of these floor fractures
and measured displacements appear to be consistent with numerical simulation of floor failure (Nemcik, 1998).

3.2 LOCATION

Tower Colliery selected for the field measurements is located in the Southern Coal Fields of Illawarra Coal Measures, NSW Australia (Figures 3.1 and 3.2). The colliery mines high quality coking coal from 2.7m thick Bulli seam at a depth of 500m below the surface. The instruments were installed in the floor of the longwall recovery roadway. This location enabled monitoring the progressive floor movement as the longwall face approached the site.

3.3 GEOLOGY

Illawarra coal measures consist of sandstones, with beds of shale, siltstone or mudstone. The formations below the Illawarra coal measures consist of sills, flows and tuffaceous sediments. A number of coal seams that were mined in this region include Bulli, Balgownie, Wongawilli, and Tongarra seams; the Bulli and Wongawilli seams are still mined today.
Floor monitoring was undertaken in the longwall recovery roadway that intercepted the 150m wide longwall finish line. The monitoring site shown on the plan in Figure 3.3 was 36m from the maingate roadway.

This location enabled measurements of progressive floor failure as the longwall face approached the site. The site was ideal for the measurements since the recovery roadways approaching the longwall face are not commonly used.
Figure 3.2 Map of the Illawarra District mine locations
3.5 FLOOR GEOLOGY

The stone floor was cored vertically down to investigate floor strata, the core was logged, and the details are presented in Figure 3.4. The bedded floor consisted mainly of siltstone, mudstone, sandstone, and coal with the first 1.8m consisting of laminated siltstone/sandstone. The small number of defects within the core, and the resistance to knife scratch indicated a moderately strong immediate floor, while the coal parting observed 1.8m below the floor surface indicated a weak discontinuity plane.

A 0.7m thick mudstone layer below appeared slightly stronger than the laminated floor above, while various layers of sandstone and siltstone below contained a few weak coal partings at 4m and 4.4m below the floor level. A 0.8m thick coal seam with a weak bedding plane at the base was located 4.7m below the floor, and lightly laminated mudstone was found below the coal seam base at 5.5 m.
Figure 3.3 Underground location of floor deformation measurements at Tower Colliery
Figure 3.4  Geological log of the floor below the longwall face finish line
3.6 INSTRUMENTATION

The plan and elevation view of the monitored site is shown in Figure 3.5 together with hole inclinations and the location of each instrument. Powered supports and the waste area (goaf) are shown on the right hand side of the diagram.

Figure 3.5 Location of the instruments installed in the floor at the longwall finish line
3.6.1 Sonic Extensometers

The sonic extensometer (Figures 3.6a to 3.6c) measures dilation of strata. For further details of the instrument and its operation, the reader can refer to the instruction manual from Geokon (1998). Three 21 anchor type sonic extensometer displacement measuring devices were installed into 55mm diameter holes drilled in the floor strata.

![Sonic extensometer readout unit and probe](Figure 3.6a)
Figure 3.6b  Extensometer magnetic anchors
Figure 3.6c  Sonic extensometer – magnetic anchor string
Figure 3.6d Sonic extensometer diagram showing configuration of the magnetic anchor string within the hole.
3.6.2 Strain gauged Shear Strip

This instrument (Figures 3.7a to 3.7c) is manufactured by SCT Operations Pty Ltd and is designed to measure the lateral shearing of bedded strata. The shear strip consists of a large number of linear strain gauges bonded to the sides of a 40mm wide by 10mm thick metal strip. Strain gauges were bonded in pairs on the opposite side of the metal strip to measure the instrument bending. Thirty nine strain gauge pairs were spaced at 50mm intervals, to measure lateral bending of the vertically positioned metal strip. The shear strip was grouted into the vertical hole in the floor with the side facing the approaching longwall face. When weak bedding planes fail and displace laterally, the metal strip deforms. From the readings of closely spaced strain gauges it possible to calculate the overall shape of the deformed metal strip and thus estimate the location and amount of lateral displacement of the floor strata.

Figure 3.7a Shearstrip – strain gauge arrangement
3.6.3 Observation Holes

The purpose of these holes is to visually observe fractures within the floor strata. The sonic extensometer holes were accompanied with visual observation holes. One of the holes, together with the sonic extensometer tube is illustrated in Figure 3.8. For easy observation, the holes were 80mm diameter and a pump was kept on standby to pump water out of the observation holes.
Shearstrip ready for installation

Plugs to suit strain bridge monitor

Figure 3.7c    Shear strip ready for installation
3.6.4 Visual observations of Floor Surface

This survey was made on the exposed floor, between the hydraulic supports and the coal face near the centre of the longwall panel. Approximately 0.3m of stone floor was cut away while the longwall moved to its finish line so there would be enough clearance to recover the longwall equipment. The floor surface at the longwall face was cleaned at 2 sites close to the centre of the longwall, and the mining induced fractures were studied. The frequency and
characteristics of fractures on the exposed floor were recorded with the results presented in Section 3.7.4.

3.7 RESULTS OF FLOOR DEFORMATION AT THE LONGWALL COAL MINING FACE

3.7.1 Measurements of Floor Displacements using Sonic Extensometry

Locations of the sonic extensometers were shown earlier in Figure 3.5. The extensometers were angled towards the longwall face to detect floor dilation ahead of the approaching longwall face. Floor displacements were read with reference to the distance of each instrument from the longwall face (Figure 3.5).

3.7.1.1 Extensometer 1

The collar of extensometer 1 located at floor level, was 2.2m from the longwall finish line with the hole dipping 80° into the floor (see Fig 3.5). Extensometer 1 results presented in Figure 3.9 indicate that a gradual dilation of upper floor strata has occurred as the longwall face approached the site, with an initial displacement of approximately 1mm when the longwall face was 10.2m from the collar of the extensometer hole.
Figure 3.9  Extensometer 1 – floor displacements measured under the longwall face (Tower Colliery)
Movement occurred at a depth of 1.7m, but no significant strata movement occurred until the longwall face was within 6.2m of the finish line. At that stage 2.4m of the immediate floor strata gradually displaced by 12mm (total movement), and when the longwall face was 4.2m from the collar of the hole, the floor dilated a total of 14mm. Floor displacement increased to 25mm when the longwall face reached the finish position. Over the next three days final floor dilation reached 47mm while the depth of floor failure increased slightly to 2.7m. Extensometer 1 indicated a distinctive zone of movement between 2m and 2.7m into the floor.

3.7.1.2 Extensometer 2

Results from the extensometer 2 (Fig 3.5) installed at an angle of 70° are shown in Figure 3.10. No significant floor displacement occurred until the longwall face was 5.8m from the extensometer collar. At that stage the floor dilated 7mm (in total). Movement occurred 3m below floor level (3.2m at an angle of 70°). When the longwall face was 3.8m from the finish line the floor dilated 9mm. When the longwall stopped at the finish line (1.8m from the collar of the hole), the total floor movement was 22mm. While the longwall was standing at the finish line, excessive lateral movement of floor strata sheared the extensometer hole and prevented further readings.
Figure 3.10  Extensometer 2 – floor displacements measured below the longwall face (Tower Colliery)
Extensometer 3 was the closest to the approaching longwall face. Results from the extensometer 3 installed at an angle of 60° are presented in Figure 3.11. The instrument measured approximately 7mm of displacement when the longwall face was 5.9m from the extensometer collar. The floor dilated 23mm in the first 2m of the extensometer hole, when the longwall face was 5.3m away. Excessive hole failure 1.3m below floor level (1.5m along the extensometer) prevented further displacement measurements deeper down, so shallow displacements (1.3m deep) were superimposed onto the last displacement. Dilation of the immediate floor continued until the last displacement (approximately 45mm) when the longwall face was within 1.5m of the hole. Further readings were terminated when the hole totally failed at a shallow depth. From the failure profile, it is probable that total floor displacement would have been much larger if hole failure had not prevented further readings.
Figure 3.11 Extensometer 3 – floor displacements below the longwall face (Tower Colliery)
3.7.2 Observation Holes

Three observation holes were drilled towards the longwall face as shown earlier in Figure 3.5. Initial observations of the holes indicated mining induced fractures that developed while the mine roadway was driven. These fractures were at random angles and would probably have influenced extensometer results.

When the longwall was within 5.8m of the finish line, minor lateral shearing of the floor bed occurred towards the longwall face, approximately 2.1m into hole 3. As the longwall approached within 3.2m from the finish line the first visible sub-vertical fracture developed 2.5m into observation hole 3. This fracture appeared parallel to the longwall face, dipping steeply downwards, such that inserting the inclinometer into the hole to determine the exact dip was not possible.

When the longwall face advanced further all observation holes filled with excess ground water, which indicated the formation of severe floor fractures that allowed water to flow from the failed floor in the goaf area. An attempt was made to pump the water out, however, the rate of inflow was too great. The water level stabilised approximately 1.4m below the floor (2.6m along the holes drilled at a dip of 33°). The early water inflow indicated that the floor was severely broken just ahead of the longwall face.
Despite protecting the holes, mine machinery had filled two observation holes with broken coal that was impossible to remove. Hole No.2 stayed open and was monitored down to the water table level. When the longwall face reached the finish line, lateral shearing of the floor bed (estimated at 30mm) was visible 2.1m into hole 2. Sub-vertical fractures were also visible 1.2m and 2.5m into the hole. Sketch of the hole profile is given in Figure 3.12.

Figure 3.12 Observed displacements in observation hole No2
3.7.3 Strain gauged Shear Strip

A strain gauged shear strip instrument was grouted into a vertical hole as shown earlier in Figure 3.5, with the side of the bar placed towards the approaching longwall face to measure lateral strata movement. It was assumed that the bottom of the shear strip would move the least amount, and it was used as the reference point.

Shear strip displacements calculated from measured strains along each side of the metal bar (Figure 3.13), indicated that the top portion of the floor gradually displaced towards the approaching longwall face. A minute displacement of floor strata was generated when the longwall face was 25m away, small displacements under 1mm were measured when the longwall was 10.7m from the site, and the last displacement was measured when the longwall face was 6.5m from the instrument. At that stage, the instrument measured bedding plane shear 1.3m below the floor, with a lateral movement of approximately 12mm towards the longwall face. Excessive floor strain damaged the shear strip and prevented further monitoring of the instrument.
3.7.4 Visual observations of floor surface

Observations at two exposed floor sites revealed sub-vertical fracture patterns spaced at frequent intervals, parallel to the longwall face and where it was possible, dips of these fractures (towards the goaf) were measured using the inclinometer. It was also possible to chip the floor with a geological hammer to expose some of the fractures and then measure their dip, but most accurate dip was measured at the longwall face where the fractures were actually exposed. All visible fractures at both sites were logged, with the information being presented in Tables 3.1 and 3.2 below. A photograph of the mining...
induced sub-vertical fracture exposed at the longwall coal face is shown in
Figure 3.14.

**TABLE 3.1** OBSERVED FREQUENCY OF THE MINING INDUCED SUB-VERTICAL FRACTURES AT A LONGWALL MINING FACE

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<th>SITE 1</th>
<th>SITE 2</th>
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<tr>
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<tr>
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<tr>
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<td>Mining Induced</td>
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TABLE 3.2 DIP OF MINING INDUCED FLOOR FRACTURES EXPOSED AT THE COAL FACE

<table>
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<th>DIP</th>
<th>PARALLEL TO LW FACE</th>
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<th>DIP</th>
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<td>0m</td>
<td>81°</td>
<td>Yes</td>
</tr>
<tr>
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<td>Mining Induced</td>
<td>0m</td>
<td>87°</td>
<td>Yes</td>
</tr>
<tr>
<td>Mining Induced</td>
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<td>Yes</td>
<td>Mining Induced</td>
<td>0m</td>
<td>88</td>
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</tr>
<tr>
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<td>Mining Induced</td>
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<tr>
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<tr>
<td>Mining Induced</td>
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<tr>
<td>Mining Induced</td>
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<td>Mining Induced</td>
<td>0m</td>
<td>83</td>
<td>Yes</td>
</tr>
</tbody>
</table>
3.8 DISCUSSION

The successful measurements show that floor movement just ahead of the longwall face occurred 2.6 to 3m below the floor surface. Extensometers 1 and 3 indicated a similar movement of up to 47mm, however the last three readings of extensometer 3 were read to 1.5m only. Extensometer 2 sustained
damage earlier, recording initial movement only. Closure of extensometer 2 and 3 holes ahead of the longwall face shows extensive floor damage just ahead of the coal face. Large quantities of ground water inflow into the observation holes located ahead of the longwall face confirm measurements of extensive floor failure and water ingress via fractures.

Even though not many mining induced fractures were visible in the holes drilled for instrumentation before monitoring started, extensometry results may have been influenced by fractures that developed during roadway driveage, however they clearly indicated that floor dilation was larger in the upper floor. Although the observation holes filled with water and coal at early stages of the monitoring, observation hole No.2 indicated that lateral shearing of floor strata and sub-vertical fractures dominated floor failure.

The strain gauged shear strip instrument clearly indicated early lateral movement along the bedding planes that had developed in the upper floor, 10.7m ahead of the coal face, and also measured a significant lateral movement 6.5m ahead of the face. This movement occurred simultaneously with floor dilation being measured by the extensometer instruments. The shear strip is sensitive to early lateral movement of floor strata, however, at a later stage of monitoring, excessive floor strain damaged the instrument and prevented further monitoring of lateral floor movement.

Observations of the floor surface at the longwall face clearly indicate the existence of sub-vertical fractures in floor strata. Recorded observations
show that mining induced fractures are parallel to the longwall coal face and dip towards the goaf at a steep angle. Observations and measurements however could not clearly reveal the magnitude of displacements along these fractures, however their frequent occurrence on the floor surface may have been related to floor strength and the stress state ahead of the coal face (Peng, 1984).

3.9 SUMMARY

Underground observations and measurements taken by the writer indicate the following actions taking place at, or just ahead of the longwall face:

- Sub-vertical fractures and movements along floor bedding planes appear to dominate floor failure at the face,
- Mining induced sub-vertical fractures occur within the zone of high stress just ahead of the coal face,
- Near-vertical mining induced fractures in the floor:
  1. dip at steep angles towards the goaf,
  2. are parallel to the coal face, and
  3. occur at frequent intervals,
- Magnitudes of displacements along the sub-vertical fractures were not measured, however, fracture surfaces located on the goaf side displace upwards in response to floor movement (heave), when the floor surface is exposed.
• Shear displacement failure along the bedding planes occurs ahead of coal face and subsequent displacements along the fractured planes continue to grow as the distance between them and the face diminishes.

• Upper floor surfaces move towards the goaf at greater rate than the lower substrata.
Chapter 4

REVIEW AND INTERPRETATION OF PRIMARY FLOOR FAILURE MECHANISM AT A LONGWALL COAL MINING FACE BASED ON UNDERGROUND MEASUREMENTS AND NUMERICAL ANALYSIS

4.1 INTRODUCTION

This chapter describes the conceptual model of primary rock failure mechanisms in the floor ahead of the longwall coal mining face, based on underground measurements, observations and numerical modelling. During longwall mining, stress fields and strata displacements are constantly changing. High stress concentrations ahead of the longwall face can generate fractures in the rock floor that can, under certain stress conditions, develop into secondary floor failure and displace large sections of strata.

Underground measurements and observations indicate that sub-vertical shear fractures and shear failure along bedding planes in stratified rock are the common fracture types in the floor beneath the longwall face. To explain how these fractures occur, basic principles of rock failure mechanisms that are applied to an overstressed rock subject to stress concentrations typically found at the longwall face are discussed here.
Floor failure can be divided into primary failure generating fractures within virgin rock and secondary failure that occurs as stress redistribution and displacements take place at, or behind, the coal face line where the floor is exposed (Indraratna, Nemcik and Gale, 2000). It is difficult to measure floor failure at the coal face (Peng, 1984) due to the nature of longwall mining operations however, floor displacements ahead of the face line were measured in a mining roadway that was driven towards the longwall. The writer feels that at present, no reliable instrumentation is available to measure 3-dimensional stresses below the floor in an undrained down hole situated or placed ahead of the longwall face.

From the measurements described in Chapter 3, it is assumed that fracture formation takes place ahead of the face where the stress state satisfies the failure criterion of rock (Hoek & Brown, 1980). Since advances in computer modelling techniques now provide an acceptable degree of accuracy in simulating complex strata behaviour, this technique was applied to investigate longwall floor failure. The Fast Lagrangian Analysis of Continua (FLAC, ITASCA, 1993) computer program was selected to simulate floor failure modes at the longwall face, and the Mohr-Coulomb criterion of rock failure was used to simulate varying rock strength and discontinuities to study stress distribution and floor failure.

The computational simulations of progressive longwall mining operations shown here were taken in stratified rock, to model complex stress distributions ahead of the longwall face, and to simulate rock failure in the mining floor.
The model was set up to simulate conditions as monitored underground (Chapter 3). Measured floor displacements were compared with the numerical predictions and the results are presented here. The numerical model indicates that shear fractures and bedding plane failure dominate the floor failure ahead of the moving longwall face as was observed underground.

Two types of primary floor failure are investigated here: (1) the development of shear failure at angles of $\pi/4 - \phi/2$ to the direction of maximum compressive stress and (2) shear failure along weak bedding planes (Brady & Brown, 1985; Hoek & Brown, 1980). The origin of stress within the floor in the vicinity of the longwall face and its influence on initial fracture development is discussed. The periodic development of fractures that were measured, observed and modelled appear to be a dominant feature of strata behaviour at the longwall face. Sequential excavation of the seam was modelled to observe progressive failure mechanism during longwall advance. Rock strength, bedding strength and the state of ground stress were the key elements investigated to explain the mechanism of primary floor failure.

4.2 EFFECT OF STRESS FIELD ON FLOOR FAILURE

Since overstressing causes rock failure, it is important to understand how post-excavation stresses develop and concentrate about the longwall face. Many stress measurements in Australia (Nemcik et al., 1998) and around the world (Fairhurst, 1986) indicate that major principal virgin stress is, in most cases,
horizontal, and increases with the depth of cover, while the vertical stress is predominantly a function of overburden weight (Herget, 1988). A dominant source of lateral stress is attributed to plate tectonics (Cox, 1973). The origin of virgin stresses and development of stress concentrations in the vicinity of the underground openings are briefly explained below.

4.2.1 Vertical Stress at Longwall Face

If surface topography is reasonably level and strata do not vary in composition, pre-mining vertical stress underground is equal to overburden weight. If a typical rock found in the coal measures strata weighs about 2.5 tonnes /m³ than pre-mining vertical stress would increase by 1 MPa per every 40m of depth. When excavating a mine opening, concentrations of stress occur near the excavation as shown in Figure 4.1.

The longwall mining face is located at the edge of the goaf (caved rock area). Concentrations of vertical stress ahead of the longwall face occur when cavity formation (goaf) takes place behind the longwall supports. The undermined overburden strata tend to overhang the excavation edges while caving occurs further away from the edge. The caving angles at which rock separation takes place in the roof at the goaf edge varies with the rock properties. These caving angles indicate that directly above the seam, weak materials such as laminite, mudstone and siltstone cave at angles of 60° to beyond 90° from the horizontal, while stronger sandstone generally caves at relatively shallow
angles of 30°- 70° to the horizontal. A typical caving of strata behind the moving longwall supports is illustrated in Figure 4.2.
Even though the caving angles described above do not necessarily represent the angles at which fracture zones propagate to the surface, the overburden overhang above the caved goaf can be estimated from the subsidence profiles. The surface subsidence profiles form a relationship between subsidence expressed as a fraction of seam thickness and panel width, expressed as a fraction of the overburden depth (Peng, 1992). The surface subsidence measurements indicate, as expected, that subsidence increases towards the centre of the panel.
Surface subsidence versus extracted panel width for the Illawarra and Newcastle regions (Holla, 1985, and 1997), shown in Figure 4.3, indicate that subsidence does not increase any more if the longwall panel width exceeds approximately 1.2-1.4 times depth of cover. The longwall is said to be supercritical in width if the maximum possible surface subsidence occurs at the panel centre. The magnitude of vertical stress varies along the length of the coal face, and is approximately at its maximum near the centre of the longwall. In supercritical panels, pillars located at the sides of the longwall panel, often called chain pillars (Peng, 1984), do not influence vertical stress at the centre of the longwall face. In the case of supercritical panel width, the coal face along the centre of the longwall face will carry a maximum possible vertical load consisting of original in-situ virgin stress and the weight of overhanging goaf strata behind the longwall face (Indraratna, Nemcik and Gale, 2000). Transfer of the vertical load towards the coal face and goaf area is shown with the aid of arrows in Figure 4.4. It indicates how a portion of the overhanging strata will induce additional vertical load onto the coal seam ahead of the longwall face.
Figure 4.3  Relationship between maximum subsidence and the ratio of longwall width to depth for regions in NSW Australia (after Holla): (a) Newcastle region; (b) Illawarra region
Figure 4.4  Rock overhang inducing vertical stress on coal face

The overburden zone supported by the coal pillar, and the zone supported by the goaf floor in relation to the depth and extracted width of a typical longwall panel, is shown in Figure 4.5.
Figure 4.5  Deep longwall panels (subcritical in width), medium and shallow panels (supercritical in width)
The maximum additional load carried by the coal ahead of the mining face at the longwall centre is due to strata overhang. The equations that approximate these loads for supercritical and sub-critical longwall geometry are discussed below (Indraratna, Nemcik and Gale, 2000).

For a supercritical longwall that is wider that 1.2 times the depth of cover, the maximum load per 1m of the face (Fig 4.6) can be approximated by:

$$W = \gamma \frac{0.6h^2}{2}$$  \hspace{1cm} (4.1)

Figure 4.6  Maximum load carried by the coal ahead of the longwall face (Supercritical longwall width)

For the sub-critical longwall width (Figure 4.7) the loads can be calculated using the Equation 4.2.
\[ W = \gamma (0.5hw - 0.21w^2) \]  \hspace{1cm} (4.2)

where:  
- \( W \) = Weight of the overhanging strata per metre of longwall face
- \( \gamma \) = Average density of overburden strata
- \( h \) = Depth of cover
- \( w \) = Width of the longwall

Figure 4.7  Maximum load carried by the coal ahead of the longwall face (Subcritical longwall width)
The shape of stress distribution ahead of the longwall face depends on strata properties and roof failure. Numerical models (Gale, 1998) and microseismic measurements (Kelly, 1998) show the two factors controlling peak stress concentrations ahead of the longwall face are depth of cover, and the modes of roof failure. Typically, when mining in weak ground (UCS<30MPa, Gale, 1998), roof failure tends to develop far ahead of the longwall face, reducing peak stress concentrations at the excavation edge and redistributing the stress further ahead of the face. In general, both vertical and lateral stress would increase the risk of floor failure with depth (Herget, 1988), and therefore, most overstressed longwalls would be sub-critical in width.

4.2.2 Lateral Stress

Underground stress measurements in Australian coal mines and around the world indicate that in most cases, virgin lateral stresses are larger than the vertical stress (Herget, 1988). The driving mechanism of the tectonic movement (Cox, 1973) is depicted in Figure 4.8.
Vast, slow convection currents moving deep within the earth mantle, propel the crust sideways, inducing collisions at the boundaries of the tectonic plates and generating high lateral stresses. Typical magnitudes of maximum lateral stress $\sigma_1$ (maximum principal stress) measured in Australian coal mines (Indraratna, Nemcik and Gale, 2000) are plotted against the depth and shown in Figure 4.9.
The measured increase of lateral stress with depth (Fig 4.9), is contributed partially by the vertical stress increase via Poisson’s ratio effect, however, the major contribution comes from tectonic loading of faulted strata. Constant lateral strain generated by the earth’s crustal movement is partially relieved when movement (slip) occurs along pre-existing fault planes. This increase in lateral stress with depth, is similar to the triaxial testing of broken rock (post-failure triaxial tests on the large scale), where maximum stress $\sigma_1$ (lateral) is the upper limit that the faulted ground is able to sustain while confined by minimum stress ($\sigma_3$). Stress measurements in Australia and overseas indicate that in most cases minimum stress ($\sigma_3$) is vertical stress. Thus the magnitude of lateral stress ($\sigma_1$) the faulted ground is able to sustain is, in most cases, dependent upon the vertical stress ($\sigma_3$) that increases with depth.

It is usual practice to orient longwall access roadways at a low angle to the maximum principal horizontal stress $\sigma_1$ to prevent damage to mine roadways and avoid large stress concentrations at the longwall corners (Matthew et al., 1992). However, this orientation can maximise concentrations of $\sigma_1$ stress in the floor acting perpendicular to the longwall face and contribute to floor failure at some depth below floor level.
The underground stress change measurements (Matthews et al., 1992) indicate that average lateral stress relief towards the goaf cavity extends over 100m from the longwall face. The probable variation of lateral stress acting in the floor below the longwall is schematically illustrated in Figure 4.10. Underground observations indicate that rock failure at floor level appears to
occur mainly at or near the centre of the longwall face. Although strata failure can take place by stress relief alone, the stress deviator ($\sigma_1-\sigma_3$) is the highest ahead of the mined coal face, increasing the chance of shear failure (Brady, 1985). Concentrations of near vertical stress ($\sigma_1$) and stress relief towards the longwall opening ($\sigma_3$) are the greatest near the centre of the longwall coal face.

4.3 DEVELOPMENT OF FRACTURES IN FLOOR

4.3.1 Observations of Floor Failure at Longwall Face

From underground observations (Chapter 3), there is increasing evidence to show that fractures at the longwall face develop along weak horizontal bedding planes, while sub-vertical fractures forming parallel to the longwall face are often visible ahead of the longwall shield supports. Sub-vertical fractures are seen to dip at steep angles of 70° to 90° towards the goaf, therefore it is logical to assume that these fractures probably occur ahead of the longwall face where the stress deviator ($\sigma_1-\sigma_3$) is assumed to be at its maximum.
4.3.2 Types of Primary Rock Failure

Two main types of primary floor failure are investigated:

- General over stressing of rock mass, where stresses within exceed the Mohr-Coulomb strength envelope and induce shear failure (Figure 4.11a),
Shear failure along undisturbed bedding plane, where overstressing of the rock occurs in the direction shown in Figure 4.11b.

Figure 4.11 Types of primary failure mechanism in floor ahead of longwall face

Primary failure may be either continuous, when the longwall face advances after each slice of coal is cut, or periodical as stress builds up after the coal face advances a specific distance. The types of primary rock failure mechanisms illustrated schematically in Figure 4.11, are addressed in the following discussion.
4.3.3 Shear Failure of Triaxially Loaded Rock

Abutment stresses may increase to such an extent they lead to failure of intact rock material below the floor of the longwall panel. This type of failure depends on the magnitude of triaxial stress and strength of rock, and can be represented by the Mohr-Coulomb criterion (Brady and Brown, 1985) given by:

$$
\sigma_{\text{crit}} = \sigma_{\text{ucS}} + \sigma_3 \frac{1 + \sin \phi}{1 - \sin \phi}
$$

where:

- $\sigma_{\text{crit}}$ = critical stress at failure of triaxially loaded rock
- $\sigma_{\text{ucS}}$ = critical stress at failure of uniaxially loaded rock
- $\sigma_3$ = confining stress (minimum principal stress)
- $\phi$ = angle of internal friction

Typical triaxial strength tests of rock under laboratory conditions indicate that rock usually fails in shear, as opposed to pure compression (crushing). Conventional Rankine analysis indicates that conjugate shear fractures occur at an angle equal to $\pi/4 - \phi/2$ from the major principal stress (Brady & Brown, 1985). Insitu propagation of fractured zones can be estimated once the triaxial stress field and rock strength parameters are known. This failure mechanism is schematically depicted in Figure 4.11a. In shallow mines, floor
failure can occur in mudstone and weak rocks, but as the depth of cover increases, failure may propagate into the stronger rocks.

4.3.4 Shear Failure along Undisturbed Bedding Plane

In general, stratified rock is weaker along the bedding planes. Under certain stress conditions, fractures can propagate along the bedding plane in preference to other directions (Hoek and Brown, 1980). The Mohr-Coulomb relationship shown in Equation 4.4 below describes the shear stress required to induce failure along the bedding plane.

\[
\tau_{c(\text{bedding})} = c_{(\text{bedding})} + \sigma_N \tan \phi_{(\text{bedding})}
\]  

(4.4)

where:

- \( \tau_{c(\text{bedding})} \) = critical shear stress at which fracture is mobilised along the bedding plane
- \( c_{(\text{bedding})} \) = cohesion along the intact bedding plane
- \( \sigma_N \) = stress normal to the bedding plane (usually vertical stress)
- \( \phi_{(\text{bedding})} \) = angle of friction of the bedding surfaces

The cohesion and friction angle along the bedding planes can be obtained from triaxial testing of samples that are prepared to contain the bedding planes at low angles to the sample axis (Indraratna, 1990). The risk of failure increases
when the maximum principal stress direction is approximately $\pi/4 - \phi/2$ to the bedding plane. Shear failure will move along the bedding plane if the applied shear stress component exceeds the shear strength of the bedding planes (Figure 4.11b). Similarly stratified rock may consist of many rock types of various properties and strength. When the triaxial stress field exceeds the strength of the weakest rock layer, failure would move along the weak rock unit to influence strata behaviour in a similar way to failure along a weak bedding plane.

4.4 NUMERICAL PREDICTIONS OF FLOOR BEHAVIOUR

A numerical model was constructed to simulate floor behaviour and conditions at the monitored site ahead of the longwall face, and to show how primary fractures form in the floor. Direct comparison between measured floor displacements (as described in Chapter 3) and the numerical predictions are presented here.
4.4.1 Numerical Model

The true behaviour of strata can be predicted only if the underground mining process is modelled in detail. The in situ stresses are initiated before mining begins. Rock failure develops in response to stress changes during mining while stresses also change with the extent of failure. To simulate progressive strata failure as it occurs underground, coal must be excavated sequentially (slice by slice) to simulate longwall advance, the rock allowed to fail, and the stress redistributed, before proceeding to the next cut.

The area of modelled FLAC element mesh (Figure 4.12) is only a portion of a much larger model mesh that extends 200m below and ahead of the longwall face, 500m to the surface and 100m into the goaf. A model of this size was chosen because it would minimise boundary effects on stress distribution in the area of interest. A typical specific gravity of sedimentary rock equal to 0.025MN/m$^3$ was used to calculate the increase in vertical stress with depth of cover while the horizontal stress was increasing at a rate of 1.5 times the vertical stress (based on stress measurements in the Illawarra Region shown in Figure 4.9).

Two numerical models were constructed to investigate whether the mechanisms of rock failure in the field can be predicted. The first model was constructed to simulate the behaviour of monitored underground floor strata (discussed in Chapter 3), while the second model used homogeneous floor strata to investigate parameters contributing to floor failure mechanism.
The longwall face was modelled using Fast Lagrangian Analysis of Continua FLAC (Itasca, 1993). The immediate floor portion was studied to help identify locations of elevated stress that are related to specific types of floor failure. Typical properties of strata used in the model were based on the
laboratory triaxial tests of overburden rock and coal seams found in the Illawarra region (Nemcik, 1998). Rock properties derived from 45mm diameter core samples for the first model are given in Table 4.1 while properties used in the second model are given in Table 4.2. To simulate in-situ rock strength, the laboratory triaxial strength of 45mm rock core presented in Table 4.1 and 4.2 was reduced by one half (Goodman, 1989) to incorporate the sample size effect (Fig 4.13).

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Figure 4.13 Influence of specimen size on the strength of intact rock
(Hoek and Brown, 1980)

Coal was cut 0.7m deep along the longwall face while the modelled hydraulic supports were initially set at 520 tonnes (80% of yield), and allowed to yield at 650 tonnes after converging 40mm under the roof load. The stone roof behind the supports was allowed to fall freely for approximately 1.5m, to reach the zone where a vertical load was applied to the falling roof that gradually brought the convergence to a halt.
The 200m wide and 500m deep longwall panel was of a sub-critical width (longwall panel width was less than 1.2 times the depth of cover), and the overhanging strata behind the longwall face were partially supported by chain pillars at each side of the face. The model was constructed to allow for partial support of overhanging strata and to simulate sub-critical longwall width in 2-dimensions. This can be approximated when mining only half the distance of the longwall width from the reflective boundary located at the start of the mining panel. The total vertical load increase in the model ahead of the longwall face can be compared with calculations using Equation (4.2) to approximate load increase near the goaf edge.

4.4.2 Modelling of Fracture Mode at Longwall Face

The primary objective of the model is to estimate the likelihood of fracture formation in the floor, and the programmable language 'fish' routines in FLAC were used to manage data within the model while the program was running. Fish routines enable the stress state at any location of the model to be studied and compared with the unique triaxial strength of each rock layer, and the type of probable rock failure evaluated.

The five major types of strata failure that are simulated in the model include: intact shear, intact bedding shear, tension crack, bedding tension and the residual failure along existing fractures. The intact strength of rocks used in
the model was given in Tables 4.1 and 4.2 while the stress state at each zone is constantly monitored and the safety factors evaluated using:

\[ FOS = \frac{\sigma_{\text{crit}}}{\sigma_1} \]  

(4.5)

where, the \( \sigma_{\text{crit}} \) (critical stress) was described earlier in Equation (4.3). When the compressive strength is exceeded at any particular zone, the likelihood of the shear failure is evaluated. Theory predicts that for static equilibrium, there are two potential failure planes (conjugates) on either side of the maximum compressive stress (oriented at an angle of \( \pi/4-\phi/2 \) from the maximum stress direction \( \sigma_1 \)). Underground observations indicate only one orientation of fractures in the roof or floor at the longwall face. To choose the probable failure plane, the model is "back stepped". The over-stressed zone is modelled in an elastic mode for a short time and the displacements are studied. Safety factors are calculated on the displaced (slightly rotated) conjugate planes and failure is assigned to the conjugate plane with the lowest factor of safety. Displacements along the failed conjugate surface occur in the same general direction as the elastic "back step" run suggests. The fractures observed in the model were similar to those observed underground.

Intact bedding shear strength and maximum tensile strength of rock are compared in a similar manner to the stress fields, with the safety factors for each grid zone being constantly evaluated. When any or all of the safety factors fall below unity, the lowest safety factor is chosen and the fracture orientation is calculated. The ubiquitous joint is then placed in the direction of the calculated fracture and the post failure properties assigned to the joint.
Tensile failure is modelled in a similar manner, allowing the rock to separate. Stress calculations and rock properties are continuously updated until equilibrium is reached. When the unbalanced forces and displacements within the grid stabilise, the longwall mining face is advanced further, a process that continues until the longwall face advances to a desirable location.

4.4.3 Comparison of Measured Floor Displacements with Numerical Predictions

The FLAC model was formulated to represent mining geometry and floor strata at the underground monitored site as described in Chapter 3. The model geometry and uniaxial compressive strength (UCS) of the floor in MPa are shown in Figure 4.14. A mine roadway excavation was simulated in the mesh of the model, displacements due to excavation were initialised and the longwall face was advanced towards the roadway (Figure 4.15). The corresponding mode of strata movement (failure) and the vertical stress contours ($\sigma_1$) are shown in Figures 4.16 and 4.17, respectively. A regular floor failure zone associated with large displacements occurred to a depth of approximately 2-3m while smaller displacements were experienced within the failed zone deeper into the floor. A failure zone that propagates at a steep angle into the floor can be seen at the edge of the roadway. As demonstrated later in this chapter (Figure 4.22), these steep failure zones are of a periodic nature and usually develop at regular intervals.
Figure 4.17 shows the vertical stress distribution in the vicinity of the longwall face. The stress contours indicate that the small remnant pillar left between the longwall face and the mine roadway is able to transmit a significant vertical stress into the floor strata while the vertical stress below the roadway is low. Typically, floor heave can occur in a broken and unconfined floor if lateral displacements are present.

Strata displacements shown in Figure 4.15 indicate that during the longwall face approach, floor heave occurs in the roadway. The net floor displacements during the longwall advance towards the roadway were determined. The underground extensometer measurements taken in the roadway during the longwall approach (presented earlier in Chapter 3, Figures 3.9 to 3.11) were compared to the modelled displacements in Figure 4.18. Good correlation of underground measurements and modelled displacements were obtained; both measured and modelled floor displacements ranged from 36mm to 47mm in total except that extensometer 2 shows incomplete displacements due to premature damage.

Lateral floor displacements that were measured below the mine roadway using the shear strip instrument (see Chapter 3) are compared with the FLAC model in Figure 4.19. This comparison indicates good correlation between the measured and the modelled results. Both measured and modelled lateral displacements at floor level were approximately 12mm. A slight anomaly to this displacement was observed within the model 1.6-1.8m below the floor.
where a single grid zone indicated 4mm of displacement away from the longwall face.

**Figure 4.14** Laboratory strength of floor strata (UCS in MPa) used in the FLAC model

**Figure 4.15** Plot of displacements (mm) in the FLAC model
Figure 4.16  Mode of strata failure in the FLAC model

Figure 4.17  Plot of vertical stress contours in the FLAC model (Compressive stress is negative)
The numerical model indicates that a wide variety of floor failure modes are possible. Displacements vary from place to place, they are dependent on deep-seated fracture zones that occur periodically within the model and also underground. Variation in floor failure results need not be surprising when considering the periodic nature of mining. The numerically modelled results indicate reasonable correlation with underground measurements showing that the model can be used to predict complex floor behaviour in an underground
mine. In fact, the occurrence of periodic events and a wide variety of floor behaviour indicates that monitoring floor failure at one location may not be sufficient to explain complex strata behaviour. In this case, a validated and reliable numerical model would be a valuable asset to assist with interpreting field measurements.

Figure 4.19 Comparison of the FLAC model floor displacements with the shear strip measurements underground
4.4.4 Stress Distribution and Fracture Modes in the Floor

Stress fields about the longwall face were simulated for a depth of cover equal to 500m. To interpret fracture modes within the floor the model was constructed using homogeneous floor strata. The longwall face was excavated in small steps from the model boundary for a distance of 250m before examining the stress field and rock failure at the longwall face. As illustrated in Figure 4.20, the vertical and lateral stress concentrations have combined to form a typical high stress zone (hatched area) about 15m below floor level ahead of the failed floor, and in front of the longwall face. Below and above the excavation where rock has failed zones of reduced compressive stress (2-6 MPa) are represented by the dotted or dashed areas in Figure 4.20. Detailed distribution of principal stresses adjacent to the fractured floor is shown in Figure 4.21, where major principal stresses rotate near the tip of sub-vertical fracture zones and become almost horizontal in deeper regions beneath the floor (The intact rock is shaded in a lighter colour).
Figure 4.20 Concentration of maximum principal stress ahead of the fractured floor
The principal stress directions concentrated at the tip of each failure zone indicate that if shear fractures develop at an angle of $\pi/4 - \phi/2$ from the maximum stress direction (Mohr-Coulomb), fractures at a shallow depth below the floor would dip at steeper angles into the floor. The corresponding failure zones based on shear strains (Figure 4.22) appear to propagate deep into the floor, dipping at angles of $70^\circ$ to $77^\circ$ towards the goaf. The results
indicate a periodic development of fracture zones that may occur after every shear, or several shear cuts.

Maximum and minimum principal stresses ($\sigma_1$ and $\sigma_3$) and their orientations were obtained for shallow depths below the floor at close proximity to the longwall face line. The magnitude and orientation of maximum principal stresses ($\sigma_1$) in the immediate floor are plotted in Figures 4.23a and 4.23b, respectively. As expected, the magnitude and direction of the $\sigma_1$ stress approximately coincided with the vertical stress ahead of the longwall face, while behind the face, stresses were affected by floor failure. Not surprisingly, the magnitude of $\sigma_1$ within the failed floor was low. The minimum principal stress $\sigma_3$ (confinement) appeared relatively large ahead of the face but diminished rapidly to smaller values just behind the face line. Tensile stress induced by bending of the immediate floor tends to develop just behind the face at a shallow depth (Figure 4.23c). The magnitude of vertical stress (Figure 4.23d) increased slightly 3m to 6m behind the face, in response to the longwall support loads. While stress was at its maximum close to the face, the original in-situ conditions prevailed further ahead of the face.
Figure 4.22  Propagation of fracture planes based on shear displacement contours
Figure a  Maximum Principal Stress in Floor at Longwall Face

Figure b  Angle of Maximum Principal Stress in Floor at Longwall Face

Figure c  Minimum Principal Stress in Floor at Longwall Face

Figure d  Vertical Stress in Floor at Longwall Face

Figure e  Shear Stress in Floor at Longwall Face

Figure 4.23  Distribution of stress at the vicinity of longwall face

DEPTH = 500m
Numerical analysis indicates that at shallow floor depths deviator stress \((\sigma_1-\sigma_3)\) is at a maximum when close to the longwall face, which increases the risk of floor failure by shear. Corresponding shear stress \((\tau)\) in the floor (Figure 4.23e) indicates the shear stress to be at its maximum 1-2m ahead of the longwall face but quickly diminishes to relatively low values further away from the face.

To study the effect of rock strength, in situ uniaxial compressive strength (UCS) of the floor were varied from 10MPa to 30MPa and the factors of safety (FOS) were calculated using Equation 4.5. The FOS were calculated at depths of 0.25m, 0.5m, 1.15m, 2m and 3m below the floor in the vicinity of the longwall face. The FOS values (Fig 4.24) indicate that when the in-situ strength of unconfined floor rock is reduced to 10MPa (e.g. weak mudstone), floor failure occurs approximately 3.5m ahead of the longwall face. When floor strength is increased to 20MPa the floor fails approximately 1m ahead of the longwall face, while the risk of floor failure at the face is totally eliminated (FOS>1.25) when in-situ floor strength is increased to 30MPa.
Figure 4.24  Factor of safety in the floor based on the Mohr-Coulomb criterion of rock failure
4.4.5 Influence of Weak Bedding Plane on Floor Failure

Computed shear stresses shown in Figure 4.23e taken at various depths below the floor, were used to evaluate the FOS for shear failure along weak bedding planes using the following equation:

\[ FOS = \frac{\tau_{crit}}{\tau_{max}} \]  \hspace{1cm} (4.6)

where: \( \tau_{crit} \) and \( \tau_{max} \) = maximum shear strength and maximum shear stress along the bedding plane.

To calculate the effect of shearing resistance on FOS, the friction angle of the weak bedding plane was varied: \( \phi = 10^\circ, 20^\circ \) and \( 30^\circ \), and the FOS was evaluated at depths of 0.25m, 0.5m, 1.15m, 2m and 3m below the floor at the vicinity of the longwall face. The results plotted in Figure 4.25 indicate that for relatively small friction angles (10° and 15°), the FOS is below unity at a considerable distance ahead of the longwall face line. In contrast, higher friction angles decrease the probability of bedding plane failure ahead of the longwall face. The likelihood of bedding failure increases with depth, which can be explained by an increase in shear stress along the bedding induced by the rotation of principal stresses below the floor.

The FOS values (see Fig 4.25) calculated from this longwall model indicate that weak bedding planes can fail ahead of the longwall face due to high shear stresses (Fig 4.23e) that exist in the floor. Failed bedding planes will lose cohesion and subsequently increase the likelihood of large horizontal floor displacement that can lead to floor heave.
Factor of Safety along Undisturbed Bedding Plane $C = 0.5 \text{ MPa}$

$\phi = 10^\circ$

- $0.25\text{m}$ below floor
- $0.55\text{m}$ below floor
- $1.15\text{m}$ below floor
- $2.00\text{m}$ below floor
- $3.00\text{m}$ below floor

Distance along Face (m)

Figure 4.25  Factor of safety along bedding plane in floor at longwall face
4.5 SUMMARY

This chapter demonstrated that the principles of initial floor failure based on underground measurements can be supported by numerical modelling, with the measured floor displacements compared well with numerical predictions. Underground observations indicate strong evidence of sub-vertical fractures occurring at regular intervals below the floor at the longwall face where the fractures tend to dip towards the goaf (caved rock) area. Displacements along weak bedding planes were also seen in the fractured floor just ahead of the longwall supports, with the FLAC model constructed to simulate floor failure mechanisms supporting the occurrence of these fractures. Underground stress measurements in Australia and subsidence profiles from the Illawarra region were used to estimate typical insitu stress conditions in the numerical model before simulating the longwall excavation. Programmable ‘fish’ routines and detailed material properties enabled the FLAC model to simulate rock failure and stress re-distribution during the longwall face advance exactly as it occurs underground.

Stress distribution at the vicinity of the longwall face was modelled to a depth of 500m. Stress computed by the numerical model indicates that in most cases, mining induced fractures occur ahead and below the longwall face where stresses are high. The factors of safety calculated in the modelled floor decreased as the longwall face approached. During sequential excavation of the modelled seam, propagation of the yield zones into the floor occurred at regular intervals, supporting the evidence for sub-vertical fractures forming at
or ahead of the longwall face. The dip of the sub-vertical failure zones predicted from the model agree with underground observations, where fractures are often seen to dip steeply towards the goaf. Directions and magnitudes of the principal stress ahead of the longwall face also indicated the likelihood of bedding plane failure. The risk of bedding plane failure increased when the maximum shear stress direction was inclined at a low angle to the bedding plane.

The results from FLAC support common underground observations where near vertical fractures and bedding plane failures dominate floor failure at, and ahead of the longwall face. The study suggests that the extent of floor failure depends on stress state and rock strength, while principal stress direction and bedding plane properties determine the failure mode. Primary floor failure appears cyclic in nature, where the near-vertical shear zones and failed bedding planes dominate floor failure geometry.

Based on this study, further theoretical work is developed and presented in the following chapters to predict secondary floor failure mechanisms that commonly interfere with longwall mining operations.
Chapter 5

CONCEPTUAL FLOOR FAILURE MODES INDUCED BY LATERAL STRESS AHEAD OF THE LONGWALL SUPPORTS

5.1 INTRODUCTION

Although floor failure at longwall faces has been associated with weak rock (Peng, 1984, Bieniawski, 1987), there have been many instances when floor buckling has occurred just ahead of the longwall supports, despite the floor being competent rock. Floor failure in strong bedded rock is associated with failure of weak bedding planes and strata movement towards the goaf opening. This chapter presents the concept of floor failure subject to high lateral stress based on numerical modelling and field observations, and outlines a practical approach for estimating the risks involved.

Buckling floor failure at the longwall face is associated with displacement of yielded coal towards the goaf. Roof and coal are laterally unconfined and are free to move towards the goaf, but floor movement is inhibited by pinning action of the longwall supports. If the coal-floor interface is weak, it will allow differential lateral movement between the coal and the floor, however, if the coal-floor interface is strong, the floor will partially restrict coal displacement towards the goaf, and large shear forces will exist close to the coal-floor boundary. If a weak bedding plane exists at a shallow depth below
the floor, the shear force may fail the bedding and large lateral stresses will
develop in the upper floor that resist lateral displacement of coal towards the
goaf. If the upper floor is relatively thin its strength may be exceeded,
resulting in floor buckling or compression failure manifesting itself as floor
heave ahead of the powered supports.

The main objective of this study is to present a conceptual model of floor
failure influenced by insitu stress relief, vertical abutment, and expansion of
the failed coal face. Both theoretical analysis and numerical modelling were
used to provide the design tools needed to predict floor failure.

5.2 MECHANISM OF FLOOR FAILURE INDUCED BY LATERAL
STRESS

Floor heave can be experienced under the following circumstances:

(a) Loading a weak and wet claystone floor,
(b) Loading broken floor, and
(c) Buckling or compression failure due to weakly bedded floor.

The load below the base of the powered support rarely exceeds the bearing
capacity of the floor. While Terzaghi's equations for bearing capacity of soils
(Terzaghi, 1967) can be used to investigate stability of claystone or broken floors, floor buckling mechanism is discussed here.

It is common to see roof and floor buckling when excessive vertical loads fail undersized coal pillars adjacent to the roadway. This frequently occurs to longwall panels when small pillars are exposed to full vertical abutment loads. The failure mechanism that causes roof and floor buckle is shown in Figure 5.1. Floor buckling failure will occur if lateral stress exceeds floor strength, but to induce lateral stress, driving forces and opposing reactions must satisfy static equilibrium where the sum of all active forces and opposing reactions acting on the floor must equal zero.

A significant lateral stress relief towards the goaf is commonly experienced ahead of the longwall face (Matthews, 1992), but lateral strata movement above the floor (associated with the stress relief), induces large shear stresses within the floor (Aggson, 1978).
If a weak bedding plane exists at a shallow depth in the floor, slip along the bedding can occur and induce excessive lateral stress in the upper floor. Numerical modelling indicates that for low angles of friction this stress is larger than the reactions supplied by the 'pinning' action of powered supports. If the floor is strong it will move towards the goaf until the driving force and opposing reaction forces are equal in magnitude. If the driving force required to displace the bedding plane below the support base is excessive, the floor will buckle or fail in compression between the face line and support base.
Figure 5.2  Floor buckling due to excessive coal expansion above the weak bedding plane

Figure 5.3  Floor buckling failure in the diagrammatic form
This failure mechanism can often be seen underground and is shown in Figures 5.2 and 5.3.

Parameters influencing floor failure can be divided into:

(a) lateral stress generated by strata movement towards the goaf,

(b) reaction force opposing floor slip,

(c) effect of friction angle along the bedding on lateral stress in the floor,

(d) effect of bedding plane depth on floor stability, and

(e) strength of the floor strata.

These aspects are described in detail below.

5.2.1 Lateral Stress generated by Strata movement towards the Goaf

Lateral strata movement towards the goaf is complex, depending upon a large number of parameters which include vertical abutment stress, depth of coal failure, the magnitude of lateral stress relief, seam strength, seam thickness, and the location of weak bedding planes. The theoretical analyses of all parameters are not attempted here, however, a numerical model was constructed to investigate some of the variables.
5.2.2 Reaction Force Opposing the Slip within the Floor

Floor movement towards the goaf is opposed by shear resistance induced below the longwall supports combined with the weight of floor strata, conveyor, and goaf. The longwall supports provide a pinning action (normal force) to the sliding floor while the other factors provide additional resistance to floor slip. The maximum shear force $S_{\text{max}}$ resisting the sliding floor (Brady, 1985) can be expressed as:

$$S_{\text{max}} = (N + \gamma h A) \tan \phi + G$$  \hspace{1cm} (5.1)

where:

- $N$ = Capacity of Longwall supports
- $\phi$ = Angle of friction along the bedding plane
- $G$ = goaf resistance
- $\gamma h A$ = self-weight exerted by the part of floor above the bedding plane
- $A$ = area of floor

To satisfy force equilibrium in the floor, lateral force induced by moving strata cannot be greater than the force due to shear resistance ($S_{\text{max}}$), to prevent slip.
5.2.3 Effect of Friction Angle on Displacement along the Bedding Plane in the Floor.

Coal expansion generates a lateral stress that drives the floor towards the goaf. Increased friction along the bedding plane allows some redistribution of lateral stress deeper into the floor whereas the increased angle of friction along the bedding plane would also minimise bedding failure ahead of the longwall face. When the floor is unloaded it will slip along the bedding plane until the forces generating lateral strata displacement are balanced by reaction forces generated along the bedding plane. Reaction forces are the result of the pinning action of the hydraulic supports. Increments in friction angles along the bedding plane would provide enhanced shear resistance against movement, and thereby increase the reaction force generated below the support base.

5.2.4 Effect of the Bedding Plane Depth on Floor Stability.

Forces generating lateral stress in the floor are independent of floor thickness above the weak bedding, but if forces acting on the floor are constant, lateral stress will depend on the cross-sectional area of the floor above the failed bedding plane. This lateral stress is inversely proportional to the cross-sectional area, therefore, thinner floors above the weak bedding plane will carry increased stress and are subject to a greater risk of buckling failure.
Two types of buckling can occur, the Eulers buckling failure or the three-hinge buckling failure (Afrouz, 1992). The Euler formula for column buckling can be used to estimate the floor buckling criterion. The theoretical buckling criterion for a long column is given by:

\[ P_{cr} = \pi^2 \frac{EI}{L^2} \]  

(5.2)

where:

\( E \) = Young's Modulus of Rock

\( I \) = Second moment of area for the cross-section of floor

\( L \) = Unconfined length of floor

It is clear that the greater the floor thickness and the shorter the floor span, the greater is the resistance to buckling. In practice, the Euler formula overestimates the stress needed for buckling because it does not consider any imperfect geometry or uneven distribution of loading, non-homogeneous nature of floor strata, or any presence of confinement.

Three-hinge buckling failure develops when mining induced fractures in the floor and an uneven geometry of the dilating floor form detached blocks that are in contact at the corners, as shown in Figure 5.4. When the centroids of forces (hinges) acting at the corners of the blocks align, the floor becomes unstable. The geometry of thin floor beds indicates a greater potential for this type of failure.
5.2.5 Strength of Floor Strata

In general, the effective strength of laminated strata is reduced if horizontally loaded. The Euler equation indicates that buckling failure is dependent on the elasticity and geometry of the floor rather than its inherent strength, however, heavily laminated floors with weak bedding planes can develop a matrix of thin beams that can lead to a complex floor failure.
5.3 NUMERICAL MODEL

Strata behaviour about the longwall face was modelled using Fast Lagrangian Analysis of Continua (FLAC) to investigate the parameters contributing to floor failure. More details about FLAC modelling are discussed in the FLAC manual Version 3.2 (Itasca 1993). Failure of a weak bedding plane located 0.2m below floor level was modelled and stress concentrations in the floor studied for different depths of cover, seam thickness, and bedding plane strength properties. Two models were constructed to simulate longwall mining. The first model used large scale geometry (Figure 5.5) to obtain the boundary stresses required to incorporate into a smaller model that studied in more detail parameters contributing to lateral stress concentrations in the floor. Details of the near field model depicting the caving zone are illustrated in Figure 5.6, rock properties and other relevant parameters used in the model are given in Table 5.1.
Figure 5.5  FLAC MODEL - Overall element discretization of surrounding rock strata
Figure 5.6  FLAC MODEL – Portion of element mesh in the near vicinity of longwall face

Table 5.1  Rock Properties used in FLAC Model

<table>
<thead>
<tr>
<th>Strata Type</th>
<th>Bulk Modulus (GPa)</th>
<th>Shear Modulus (GPa)</th>
<th>Angle of Internal Friction</th>
<th>Cohesion (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>5</td>
<td>3</td>
<td>30°</td>
<td>3</td>
</tr>
<tr>
<td>Coal</td>
<td>1</td>
<td>0.5</td>
<td>30°</td>
<td>1</td>
</tr>
<tr>
<td>Floor</td>
<td>5</td>
<td>3</td>
<td>30°</td>
<td>3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Bedding Plane</th>
<th>Normal Stiffness (GPa)/m</th>
<th>Shear Stiffness (GPa)/m</th>
<th>Cohesion (MPa)</th>
<th>Friction along Bedding Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interface</td>
<td>3</td>
<td>3</td>
<td>0 - 1</td>
<td>5 - 30</td>
</tr>
</tbody>
</table>
5.3.1 Results of Numerical Modelling

To identify the parameters that elevate lateral stress within the floor, a bedding plane was placed 0.2m below floor level and the floor was numerically fixed to prevent it from slipping towards the goaf. Lateral stress increase within the floor was studied as each parameter changed.

5.3.2 Shear Stress in the Floor generated by Strata Movement towards the Goaf

To enable shear stress to fully mobilise along the bedding plane, relatively high strength properties were assigned to the bedding. Shear stress in the floor near the toe of the face was computed and plotted against the depth of cover and seam thickness. Shear stress values in the floor at different depths of cover are given in Figure 5.7. Failure occurs when mobilised shear stress exceeds the critical shear strength of the bedding plane (based on linear Mohr-Coulomb criterion). The results indicate that floor failure is unlikely if the angle of bedding friction exceeds 30°. Decrease in the angle of friction allows bedding failure to propagate further ahead of the longwall face. For angles of friction of 10°, 20° and 30°, failure propagated approximately 2-3.5m, 1-2m, and 0.5m ahead of the face, respectively, depending on depth of cover.
Figure 5.7 FLAC MODEL – Vertical and shear stress in floor versus distance ahead of longwall face
5.3.3 Development of Lateral Stress in the Constrained Floor above the Bedding Plane

The 0.2m thick floor above the failed bedding plane was restricted at the toe of the longwall support to study how much maximum lateral stress can develop. Lateral stress magnitudes in the floor were studied with respect to depth of cover and the angle of friction $\phi$ along the bedding. The results summarised in Figure 5.8 indicate that lateral stress increases as the friction angle is reduced while it also increases in proportion to seam thickness and depth of cover. Negligible forces in the floor beam were present when the angle of friction along the bedding plane was increased more than 30°.

5.3.4 The Effect of Reaction Force onto Lateral Stress in the Floor.

Reaction forces resisting strata movement are generated by the pinning action of longwall supports, the weight of armoured conveyor, and the floor located in the goaf. If the lateral forces acting on the upper floor are larger than the reaction forces, the floor will slip towards the goaf, relieving any excess force. When the modelled floor above the bedding plane moved towards the goaf the lateral force acting on the floor decreased to approximately the theoretical value given by Equation (5.1).
Figure 5.8 FLAC MODEL – Lateral stress in floor induced by strata movement and powered supports versus bedding friction and seam thickness
Assuming the supports provide the major reaction opposing floor movement, maximum lateral stress in the floor can be computed using Equation (5.1). Theoretical reaction forces plotted against maximum lateral stress in the restricted floor indicated that maximum floor stress occurs when friction angles are between 15°-25°, but with increasing goaf resistance, maximum stress in the floor will grow in magnitude and occur at a lower friction angle.

5.4 TECHNIQUES TO ASSESS PARAMETERS ASSOCIATED WITH FLOOR FAILURE.

5.4.1 Determining the Shear Resistance along Weak Bedding Planes

To assess possible floor buckling, potential weak bedding planes need to be located and tested. If weak bedding planes are located at a shallow depth, coring of floor samples is required at approximately 30° to the bedding planes with laboratory triaxial tests conducted to determine shear resistance along the bedding planes (Indraratna, 1990).
5.4.2 Estimating Maximum Lateral Stress in the Floor

Once the angle of friction along the bedding plane is determined the reaction forces generated by the longwall supports can be calculated on the basis of linear Mohr-Coulomb theory. Additional reactions caused by the armoured conveyor, goaf load, and weight of floor strata, are difficult to estimate, but if the floor in the goaf is broken, these reactions may not be large enough to provide substantial resistance to moving strata. Assuming the supports alone provide the reactions opposing floor movement, maximum lateral stress can be computed using Equation (5.1).

5.4.3 Estimation of Floor Strength

The type of floor failure subject to lateral stress depends on floor thickness (above the failed bedding plane) and unconfined floor length. The unconfined length of exposed floor at the face is measured from the face to the toe of the longwall support. Although gravity provides some stability to the floor slab, this may not be large enough to prevent floor buckling. In this case Euler Equation (5.2) can be used to estimate buckling failure in the absence of confining stress, but floor distortion and mining induced fractures must be considered for a more realistic analysis.
5.4.4 Minimising Floor Failure.

Several actions can be adopted to minimise floor failure:

- Keep longwall supports as close to the face as possible to reduce free floor span,
- Keep the floor dry because water ingress weakens the floor, and
- Mount the pull-out jacks on the longwall hydraulic support bases to minimise base penetration into the fractured floor.

5.5 SUMMARY

The results of FLAC modelling indicate that the major influence on floor failure is the location of weak bedding plane, the angle of friction along the bedding plane, and the magnitude of generated reactions resisting floor movement. This study shows that maximum lateral stress in the floor develops when the friction angle along the bedding plane is between $15^\circ$ and $25^\circ$. Friction angles higher than $30^\circ$ will reduce the magnitudes of lateral stress induced by moving strata, and friction angles lower than $15^\circ$ reduce reaction forces generated by the pinning action of longwall supports.

It is difficult to quantify all of the parameters contributing to floor failure because many types of strata may be encountered during longwall extraction.
Change in bedding plane properties, bedding depth and type of rock affect the face such that only localised floor failures occur. Persistent floor failures are usually associated with thin, clay, or mudstone floors of a low strength, where numerous weak bedding planes allow lateral floor displacements.

Success in estimating floor stability is dependant upon the quality and quantity of geotechnical investigations and analysis of floor stability prior to mining.
Chapter 6

FLOOR FAILURE ANALYSIS AT LONGWALL MINING FACE BASED ON THE MULTIPLE SLIDING BLOCK MODEL

6.1 INTRODUCTION

This chapter presents an analytical model of floor failure mechanism at a longwall coal mining face based on a multiple sliding block model. During longwall mining, stresses and displacements of strata are constantly changing. High stress concentrations can exceed rock strength and initiate strata fractures that can, under unfavourable conditions, lead to large floor displacements and disruption of mining.

Underground observations of the rock floor and computational modelling of the longwall face, indicate that sub-vertical fractures and bedding plane shear dominates floor failure. Extensive lateral shearing of weak bedding planes typically present in the sedimentary strata and sub-vertical fractures that usually occur at regular intervals during face advance give, the floor strata a typical blocky appearance.
Fractures that develop ahead of the longwall face are subject to a 'secondary' movement when exposed ahead of the longwall supports. As coal is mined from above, floor strata moves toward the opening, causing the floor to bend. If the floor fails, blocks displace in response to floor movement and interact at the fractured surfaces. The analysis described in this chapter attempts to explain how stress distribution that develops within broken floors during an active movement of floor strata leads to high stress concentrations at floor level. These stress concentrations can exceed rock strength and induce compression failure of the floor that may interfere with longwall operations.

The multiple sliding block geometry within the floor was developed on the basis of observations presented in Chapter 3, while the analytical formulations presented here are supplemented by numerical modelling to verify that the results are in accordance with numerical predictions.

6.2 PROPOSED MECHANISM FOR THE MULTIPLE SLIDING BLOCK MODEL

The proposed floor failure mechanism based on a multiple sliding block model can develop where geological conditions are favourable (Terzaghi, 1967). These conditions require extensive lateral fracturing that often develop along the numerous weak bedding planes present in sedimentary strata, and sub-vertical fractures that normally form in response to changing stress abutments ahead of the longwall coal face. These failure mechanisms were extensively
modelled (Gale, 1998) and measured underground using microseismic surveys (Kelly, 1998). Lateral and near vertical fracture planes that define stone blocks within the mining floor will interact during floor heave and can induce large stresses at the corners of the blocks.

The analytical solution for multiple sliding blocks was specifically designed to suit floor movement. The analysis assumes a failed bedding plane deep within the floor and near vertical fractures forming at regular intervals that define the geometry of moving blocks (Nemcik, 1998). It also assumes planar and curved floor inclinations on which the blocks move. Progressive floor uplift (Peng, 1984) and the reaction forces generated at the face initiate block movement while continuous floor uplift creates an inclined surface on which the blocks can slide. Analytical equations have been derived to calculate stress magnitudes at the block corners during floor uplift and describe the force generated between free-standing blocks with an additional loading of powered supports.

The computational model was formulated to compare analytical solutions with the numerical results and simulate the interaction of blocks standing on an inclined floor experiencing uplift. This simulation was repeated for a number of planar and parabolic floor inclinations including an additional vertical loading induced by powered supports.
6.3 POST FAILURE BEHAVIOUR OF FLOOR SPLIT BY MINING INDUCED FRACTURES

The following analysis assumes that a single bedding plane fails below the floor and that vertical fractures develop ahead of the longwall face at regular intervals forming blocks, as shown in Figure 6.1. Progressive longwall mining causes continuous floor uplift at the longwall face which initiate an active slip of blocks along the sub-vertical fractures and lateral slip along the failed bedding plane.

The analytical approach and numerical modelling are presented to explain how continuous floor deformation and actively sliding blocks can induce lateral stress concentrations at the floor level. Block behaviour varies according to geometry and angle of friction along the slip surfaces. In most cases, near vertical fracture surfaces dip steeply towards the goaf so vertical surfaces were assumed to simplify calculation of forces at the block sides.
6.4 ACTIVE SLIP OF BLOCKS RESTING ON AN INCLINED BEDDING PLANE

In response to longwall mining, continuous floor uplift is generally experienced as illustrated in Figure 6.2 while Figure 6.3 describes forces of actively slipping blocks standing on an inclined surface subject to:

- weight of block (W)
- lateral interaction force (Q)
- frictional force at block sides \((Q \tan \phi_v)\) where \(\phi_v\) is the angle of friction along vertical fractures, and
- reaction force at the bottom of each block consisting of normal force \((N)\) and shear force \((N \tan \phi_h)\), where \(\phi_h\) is the angle of friction along the horizontal bedding.

Figure 6.2 Schematic representation of block movement in floor

Note that the blocks move upwards, that the shear forces along the sides of the blocks are also in the direction of movement and the friction along vertical fractures increases with the normal force \(N_i\) at the base of each block. The ability to slip either along the vertical plane or the horizontal bedding appears
to be related to floor shape, block geometry and the angle of friction along the slip surfaces.

Three possible cases of block behaviour are shown in Figure 6.4, with each case being described and analysed in the following pages.

Figure 6.3 Schematic representation of acting forces on block sides experiencing floor tilt
Figure 6.4  UDEC model of magnified block displacements
6.4.1 Slip Along Fracture Boundaries, No Block Rotation

This model (Figure 6.4a) assumes that movement occurs along both the vertical and horizontal fractures simultaneously. The analytical solution of block movement is presented below:

A free body diagram presented in Figure 6.3 shows the forces acting upon a single block. The angle of friction $\phi_h$ along the failed bedding plane will depend on the material properties of geological discontinuity along which the failure is developed. For the blocks to slip along the failed bedding plane, the force $F_s$ overcoming shear resistance $S_h$ must exceed the Mohr-Coulomb criterion (Brady and Brown, 1985):

$$F_s > S_h,$$

and

$$S_h = N_i \tan \phi_h$$

where,

- $S_h = \text{shear resistance along failed bedding plane}$
- $N_i = \text{normal force to the bedding plane}$
- $\phi_h = \text{angle of friction along failed horizontal bedding}$

The following analytical procedure shows how to calculate the maximum lateral force $Q$ concentrated at the point of contact at the top corner of each block resting on an inclined surface.

In order to derive an equation to evaluate $Q$ forces at block corners, the first trial assumes that floor surface inclination $\alpha$ is planar, includes self-weight of
the floor and the additional surface load \( P \). Consider the "block\(_1\)" at the edge of the boundary as shown in Figure 6.4.

From the free body diagram in Figure 6.3 it can be observed that the normal force to the bedding plane at the block\(_1\) is given by:

\[
N_1 = W_1 \cos \alpha + \tan \phi_v Q_1 \cos \alpha + \sin \alpha Q_1
\]  \hspace{1cm} (6.1)

The normal force to the bedding plane \( N_2 \) at the block\(_2\) is:

\[
N_2 = W_2 \cos \alpha + \tan \phi_v (Q_2 - Q_1) \cos \alpha + \sin \alpha (Q_2 - Q_1)
\]  \hspace{1cm} (6.2)

Similarly it can be derived that the normal force at block\(_i\) is:

\[
N_i = W_i \cos \alpha + \tan \phi_v (Q_i - Q_{i-1}) \cos \alpha + \sin \alpha (Q_i - Q_{i-1})
\]  \hspace{1cm} (6.3)

The lateral force \( Q_1 \) is equal to:

\[
Q_1 = N_1 (\sin \alpha + \tan \phi_h \cos \alpha)
\]  \hspace{1cm} (6.4)

Substituting Equation (6.1) for \( N_1 \) leads to:

\[
Q_1 = (W_1 \cos \alpha + \tan \phi_v \cos \alpha Q_1 + \sin \alpha Q_1) (\sin \alpha + \tan \phi_h \cos \alpha)
\]
Solving for $Q_1$,

$$Q_1 = \frac{W_1 \cos \alpha (\sin \alpha + \tan \phi_h \cos \alpha)}{1 - (\tan \phi_c \cos \alpha + \sin \alpha) (\sin \alpha + \tan \phi_h \cos \alpha)}$$

Rearranging gives:

$$Q_1 = \frac{W_1 \cos \alpha}{(\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_h \cos \alpha)}$$  \hspace{1cm} (6.5)

The lateral force $Q_2$ is equal to:

$$Q_2 = (N_1 + N_2) (\sin \alpha + \tan \phi_h \cos \alpha)$$  \hspace{1cm} (6.6)

Substituting Equations (6.1) and (6.2) for $N_1$ and $N_2$ gives:

$$Q_2 = (W_1 \cos \alpha + \tan \phi_c \cos \alpha Q_1 + \sin \alpha Q_1 + W_2 \cos \alpha + \tan \phi_c \cos \alpha Q_2 - \tan \phi_c \cos \alpha Q_1 + \sin \alpha Q_2 - \sin \alpha Q_1) (\sin \alpha + \tan \phi_h \cos \alpha)$$

Simplifying the above leads to:

$$Q_2 = (W_1 \cos \alpha + W_2 \cos \alpha + \tan \phi_c \cos \alpha Q_2 + \sin \alpha Q_2) (\sin \alpha + \tan \phi_h \cos \alpha)$$

Solving for $Q_2$ gives:
\[ Q_2 = \frac{(W_1 + W_2) \cos \alpha (\sin \alpha + \tan \phi_h \cos \alpha)}{1 - (\sin \alpha + \tan \phi_v \cos \alpha) (\sin \alpha + \tan \phi_h \cos \alpha)} \quad \text{or} \]

\[ Q_2 = \frac{(W_1 + W_2) \cos \alpha}{(\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha)} \quad (6.7) \]

Further it can be proven that for any force \( Q_n \):

\[ Q_n = \frac{\sum_{i=1}^{n} W_i \cos \alpha}{(\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha)} \quad (6.8) \]

For the blocks of the same weight, the equation is linear and becomes

\[ Q_n = \frac{nW \cos \alpha}{(\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha)} \quad (6.9) \]

Expanding Equation (6.9) gives:

\[ Q_n = \frac{nW (\sin \alpha \cos \alpha + \tan \phi_v \cos^2 \alpha)}{1 - \tan \phi_v \sin \alpha \cos \alpha + \sin^2 \alpha - \tan \phi_h \tan \phi_v \cos^2 \alpha + \tan \phi_h \sin \alpha \cos \alpha} \quad (6.10) \]

If the inclined slope on which the blocks slide is not planar and

\[ \alpha_i \neq \alpha_{i-1} \neq \alpha_{i-2} \ldots \alpha_1. \]
Q force can be calculated as shown below:

\[
Q_1 = \frac{W_i \cos \alpha_i}{(\sin \alpha_i + \tan \phi_h \cos \alpha_i)^{-1} - (\sin \alpha_i + \tan \phi_v \cos \alpha_i)} \tag{6.11}
\]

The \( Q_2 \) is dependent on \( Q_1 \) and becomes:

\[
Q_2 = \frac{W_2 \cos \alpha_2 - Q_1 ((\sin \alpha_2 + \tan \phi_v \cos \alpha_2) - (\sin \alpha_2 + \tan \phi_h \cos \alpha_2)^{-1})}{(\sin \alpha_2 + \tan \phi_h \cos \alpha_2)^{-1} - (\sin \alpha_2 + \tan \phi_v \cos \alpha_2)} \tag{6.12}
\]

and for any force \( Q_n \), it can be proven that:

\[
Q_n = \sum_{i=1}^{n} \frac{W_i \cos \alpha_i - Q_{i-1} ((\sin \alpha_i + \tan \phi_v \cos \alpha_i) - (\sin \alpha_i + \tan \phi_h \cos \alpha_i)^{-1})}{(\sin \alpha_i + \tan \phi_h \cos \alpha_i)^{-1} - (\sin \alpha_i + \tan \phi_v \cos \alpha_i)} \tag{6.13}
\]

6.4.2 Height of the Lateral Force Centroid \( Q_i \)

Assuming that \( W_1 = W_2 = W_3 = \ldots = W \) and \( \alpha_1 = \alpha_2 = \alpha_3 = \ldots = \alpha \)

Taking moments about the normal force interaction point \( N_i \) of the first block: 
\[
\frac{WL}{2} - Q_1 h_1 + LQ_1 \tan \phi_v = 0
\]  \hspace{1cm} (6.14)

Solving for the height of centroid \( h_1 \) gives:

\[
h_1 = \frac{WL}{2Q_1} + L \tan \phi_v
\]  \hspace{1cm} (6.15)

Substituting Equation (6.5) for \( Q_1 \) gives:

\[
h_1 = \frac{L((\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha))}{2 \cos \alpha} + L \tan \phi_v
\]  \hspace{1cm} (6.16)

Similarly taking moments about the normal force point \( N_2 \) of the block\(_2\):

\[
\frac{WL}{2} - Q_1 h_1 + LQ_2 \tan \phi_v - Q_2 h_2 = 0
\]  \hspace{1cm} (6.17)

Solving for \( h_2 \):

\[
h_2 = \frac{WL}{2Q_2} + L \tan \phi_v + h_1 \frac{Q_1}{Q_2}
\]  \hspace{1cm} (6.18)

Substituting Equation (6.5), (6.7) and (6.15) for \( Q_1, Q_2 \) and \( h_1 \) leads to:

\[
h_2 = \frac{L((\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha))}{2 \cos \alpha} + \frac{3L \tan \phi_v}{2}
\]  \hspace{1cm} (6.19)
Similarly it can be proven that:

\[ h_i = \frac{L((\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha))}{2 \cos \alpha} + \frac{(i + 1)L \tan \phi_v}{2} \] (6.20)

Equation (6.20) indicates that as the distance from the block \( i \) increases (term \( i+1 \)), the height of the centroid \( Q_i \) increases until it coincides with the top corner of the block, where \( h_i = H_i \).

The condition of the block rotation can be described as:

\[ Q_{i+1}(h_{i+1} - \frac{H}{2}) + \frac{N_i L}{2} \tan \phi_h < (Q_{i+1} + Q_i) \frac{L}{2} \tan \phi_v + \frac{N_i L}{2} + Q_i(h_i - \frac{H}{2}) \] (6.21)

If the left hand side of the Equation (6.21) is smaller than the right hand side, the block will rotate to the position where the base is in full contact with the bedding plane. Equation (6.21) indicates that if the angle of friction along the bedding plane decreases while the friction angle along the vertical fractures increases, the chance of block rotation will increase. Reducing the geometrical ratio \( H/L \) would also increase the likelihood of block rotation.
6.4.3 Additional Load induced by Powered Supports

An additional load $P_i$ induced on the block $i$ by the powered supports modifies the above equations as given below where Equation (6.3) is modified to:

$$N_i = (W_i + P_i)\cos \alpha + \tan \phi_v \cos \alpha (Q_i - Q_{i-1}) + \sin \alpha (Q_i - Q_{i-1})$$ \hspace{1cm} (6.22)

where, \hspace{1cm} $P_i =$ Load of Powered support on block $i$

Equation (6.9), (6.13) and (6.20) now become:

$$Q_{ai} = \frac{n(W + P)\cos \alpha}{(\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha)}$$ \hspace{1cm} (6.23)

$$Q_{ai} = \sum_{i=1}^{n} \frac{(W_i + P_i)\cos \alpha - Q_{i-1} \{(\tan \phi_v \cos \alpha_i + \sin \alpha_i) - (\sin \alpha_i + \tan \phi_h \cos \alpha_i)^{-1}\}}{(\sin \alpha_i + \tan \phi_h \cos \alpha_i)^{-1} - (\sin \alpha_i + \tan \phi_v \cos \alpha_i)}$$ \hspace{1cm} (6.24)

$$h_i = \frac{L(P_i + \frac{W}{2})\{(\sin \alpha + \tan \phi_h \cos \alpha)^{-1} - (\sin \alpha + \tan \phi_v \cos \alpha)\}}{(P_i + W_i)^2\cos \alpha} + \frac{(i + 1)L \tan \phi_v}{2}$$ \hspace{1cm} (6.25)

Equation 6.25 indicates that the $Q$ forces are largest at the longwall face and the $P$ force induced by powered supports increases the $Q$ forces significantly.
6.4.4 Rotation of Block with Slip along the Bedding Plane

The Equation (6.20) indicates that the centroid \( h_i \) of the lateral force \( Q_i \) is sensitive to the friction angle along the vertical fractures, and the distance towards the face. The friction angle along the vertical fractures \( \phi_v \) is typically \( \approx 35^\circ \) and possibly higher due to an uneven fracture surface (angle \( i \)). Under the conditions described by Equation (6.23), rotation will occur and the base of the block will move to the position shown earlier in Figure 6.4(b). The analytical solution and the computational model indicate that if block rotation occurs during active block movement, normal stress at the base is, in most cases, concentrated close to the uphill corner of each block. For block rotation, the lateral force \( Q \) can be calculated as described in the following section.

6.4.5 Rotation of Block with No Slip along Vertical Fractures

For planar bedding inclination \( \alpha \) (Figure 6.4b), calculations of the \( Q_n \) force at the coal face can be determined as follows. The \( Q \) forces are taken as being parallel to the bedding plane. Taking moments about the face point where the \( Q_n \) force is calculated and considering all "n" blocks at once, the sum of forces is equal to zero. Therefore,
\[
\sum_{i=1}^{n} \{(W_i + P_i)(n - i + 1/2)L + N_i(H \tan \phi_n - 1)\} = 0 \tag{6.26}
\]

where, \( L \) = width of the blocks

\( H \) = height of the blocks

Solving for the sum of normal forces \( N_i \):

\[
\sum_{i=1}^{n} N_i = \sum_{i=1}^{n} \{(1 - H \tan \phi_n) - (W_i + P_i)(n - i + 1/2)L\} \tag{6.27}
\]

Finally, the \( Q_n \) force can be calculated as shown below:

\[
Q_n = \sum_{i=1}^{n} N_i \tan \phi_n \tag{6.28}
\]

**6.4.6 Rotation of Block with No Slip along Vertical Fractures standing on a Curved Slope**

For curved bedding inclination as shown in Figure 6.4c, \( Q \) forces were taken as being parallel to the bedding plane. By taking moments about the top left corner of each block where the \( Q \) forces are transferred through, the normal force at the base of block \( i \) is equal to:
\[ N_1 = \frac{0.5L(W_1 + P_1)}{L - H \tan \phi_h} \] (6.29)

Summing the forces parallel to the bedding plane, the \( Q_1 \) force can be calculated as:

\[ Q_1 = N_1 \tan \phi_h \] (6.30a)

Substituting for \( N_1 \):

\[ Q_1 = \frac{0.5L(W_1 + P_1)}{L - H \tan \phi_h} \tan \phi_h \] (6.30b)

Similarly for block2:

\[ N_2 = \frac{0.5L(W_2 + P_2) - LV_1 \cos(\alpha_1 - \alpha_2)}{L - H \tan \phi_h} \] (6.31)

where, \( V_1 \) is the force along the vertical fracture of Block1 and is equal to:

\[ V_1 = N_1 - (W_1 + P_1) \] (6.32)

Summing the forces parallel to the bedding plane, the \( Q_2 \) force can be calculated as:

\[ Q_2 = N_2 \tan \phi_h + Q_1 \cos(\alpha_1 - \alpha_2) \] (6.33)
Substituting for $N_2$:

$$Q_2 = \frac{0.5L(W_2 + P_2) - LV_1 \cos(\alpha_1 - \alpha_2)}{L - H \tan \phi_h} \tan \phi_h + Q_1 \cos(\alpha_1 - \alpha_2)$$ (6.34)

It can be derived that for any term $N_i$:

$$N_i = \frac{0.5L(W_i + P_i) - LV_{i-1} \cos(\alpha_i - \alpha_{i-1})}{L - H \tan \phi_h}$$ (6.35)

$V_{i-1}$ is the frictional force along the vertical fracture on the right side of block $i$ and must be smaller than $Q_i \tan \phi_v$. Any frictional force along the vertical fracture can be iterated by:

$$V_i = V_{i-1} + N_i - W_i - P_i$$ (6.36)

The general $Q$ force can be evaluated as:

$$Q_i = N_i \tan \phi_h + Q_{i-1} \cos(\alpha_i - \alpha_{i-1})$$ or $$Q_i = \frac{0.5L(W_i + P_i) - LV_{i-1} \cos(\alpha_i - \alpha_{i-1})}{L - H \tan \phi_h} \tan \phi_h + Q_{i-1} \cos(\alpha_i - \alpha_{i-1})$$ (6.37)

$$Q_i = \frac{0.5L(W_i + P_i) - LV_{i-1} \cos(\alpha_i - \alpha_{i-1})}{L - H \tan \phi_h} \tan \phi_h + Q_{i-1} \cos(\alpha_i - \alpha_{i-1})$$ (6.38)
6.4.7 Rotation of Block with Slip along Vertical Fractures

For certain conditions along the non-planar bedding inclination $\alpha_i$ as shown in Figure 6.4 (c), the slip along the vertical fractures will occur. In this case, the Q forces are taken as being parallel to the bedding plane.

Summing the forces perpendicular to the bedding plane, the normal force to the bedding at Block$_1$ can be calculated by:

$$N_1 = (W_1 + P_1) \cos \alpha_1 + Q_1 \tan \phi_v$$  \hspace{1cm} (6.39)

Summing the forces parallel to the bedding plane, the $Q_1$ force can be evaluated to give:

$$Q_1 = N_1 \tan \phi_h + (W_1 + P_1) \sin \alpha_1$$  \hspace{1cm} (6.40)

Substituting for $N_1$ and rearranging gives:

$$Q_1 = \frac{(W_1 + P_1) (\sin \alpha_1 + \tan \phi_h \cos \alpha_1)}{1 - \tan \phi_h \tan \phi_v}$$  \hspace{1cm} (6.41)

Similarly for block$_2$:

$$N_2 = (W_2 + P_2) \cos \alpha_2 + (Q_2 - Q_1) \tan \phi_v$$  \hspace{1cm} (6.42)
Summing the forces parallel to the bedding plane, the $Q_2$ force can be calculated as shown below:

$$Q_2 = Q_1 + N_2 \tan \phi_n + (W_2 + P_2) \sin \alpha_2$$  \hspace{1cm} (6.43)

Substituting for $N_2$ and rearranging gives:

$$Q_2 = Q_1 + \frac{(W_2 + P_2)(\sin \alpha_2 + \tan \phi_n \cos \alpha_2)}{1 - \tan \phi_n \tan \phi_v}$$  \hspace{1cm} (6.44)

For general forces $N_i$ and $Q_n$, it can be proven that:

$$N_i = (W_i + P_i) \cos \alpha_i + (Q_i - Q_{i-1}) \tan \phi_v$$  \hspace{1cm} (6.45)

and

$$Q_n = \sum_{i=1}^{n} \frac{(W_i + P_i)(\sin \alpha_i + \tan \phi_n \cos \alpha_i)}{1 - \tan \phi_n \tan \phi_v}$$  \hspace{1cm} (6.46)

The computational codes to solve Q force Equations derived in Section 6.4 for various slope shapes are given in the Appendix, Section A2.
6.5 COMPUTATIONAL MODEL USING UDEC

The model was used to study interactive forces between the block surfaces and to compare them with the forces derived when using analytical equations. Using the Universal Distinct Element Code UDEC (Itasca, 1993), the model was constructed to represent a fractured floor consisting of interacting blocks sliding on an inclined bedding plane. The free standing blocks used to model the fractured floor were 1m wide 2m high, while the base on which the blocks were sliding represented a failed horizontal bedding plane. To simulate an increase in floor elevation, the bedding was moved upwards at a predetermined rate. The floor was either a planar slope or curved into a parabolic shape as shown earlier in Figure 6.4. The blocks were gravity loaded and a vertical load of 650 tonne was applied onto the 4th, 5th and 6th block from the face, to simulate the pressure induced by powered supports onto the floor. The “face block” shown on the left hand side of Figure 6.2 was fixed, to provide reaction forces to the moving blocks. The properties of fractured surfaces used in the numerical model are given in Table 6.1 (Nguyen, 1981), and the interacting forces between the block surfaces were studied. The computational code used here is given in the Appendix, Section A3.
Table 6.1 Rock and Fracture Properties used in UDEC Model (Nguyen, 1981)

<table>
<thead>
<tr>
<th>Block Properties</th>
<th>Bulk Modulus (GPa)</th>
<th>Shear Modulus (GPa)</th>
<th>Normal Stiffness (GPa/m)</th>
<th>Shear Stiffness (GPa/m)</th>
<th>Angle of Friction along Fractures</th>
<th>Cohesion (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedding Fractures</td>
<td>9</td>
<td>6.7</td>
<td></td>
<td></td>
<td>15-35°</td>
<td>0</td>
</tr>
<tr>
<td>Vertical Fractures</td>
<td>9</td>
<td>6.7</td>
<td></td>
<td></td>
<td>15-45°</td>
<td>0</td>
</tr>
</tbody>
</table>

6.6 COMPARISON OF ANALYTICAL AND NUMERICAL SOLUTIONS

The possible cases of sliding models described earlier have been numerically modelled using UDEC, as shown in Figure 6.4. The analytical predictions for "Q-forces" represented by Equations 6.23, 6.24, 6.28, 6.38 and 6.46 are compared with UDEC numerical results for block rotation and no rotation.
Moving blocks modelled on the inclined planar bedding surface experienced slip along the vertical and horizontal fractures, without rotation. Calculated and modelled Q forces are compared in Figure 6.5 where, for low angles of friction along the fractures, the modelled Q force approximates the calculated force from Equation 6.23 that describes the "no rotation" movement. When angles of friction were greater than 25° the interacting Q force appears to follow a lower path than the calculated force of non-rotating blocks. Reduction of the expected Q force in the model reflects the complex movement of all six modelled blocks, where some blocks were displaced along the fractures only, whilst others were rotated slightly. Magnified displacements from the UDEC model of blocks with no rotation are shown in Figure 6.4a, while slip along the bedding plane with no vertical slip, is shown in Figure 6.4b. Block movement conditions modelled by UDEC were magnified for clarity.
6.6.2 Moving Blocks standing on Curved Bedding Surface

When modelling the parabolic inclination of floor taking the shape of curve $y = x^2/400$, the blocks were approximating the rotations shown in Figure 6.4(c). The Q forces obtained from the model were compared to the forces derived by Equations 6.24, 6.38 and 6.46, and are shown in Figure 6.6.

Figure 6.5 Interacting Q force between floor blocks standing on moving planar bedding plane
The above plots indicate that for the described geometry the modelled Q forces were slightly smaller than calculated Q forces.

When the floor was inclined in the shape of $y = x^2/200$, upward movement of the modelled floor induced block displacement similar to the previous case. The magnified UDEC displacements shown in Figure 6.4c indicated that the blocks rotated and slipped along the bedding plane but did not slip along the
vertical fractures. The exceptions were blocks 1 and 2, which were loaded by powered supports. The interacting Q forces modelled by UDEC were similar to all equations for lower bedding friction. For higher bedding friction, modelled results were similar to Equation 6.38 for block rotation with no vertical slip. The results are compared in Figure 6.7.

For a planar floor, UDEC predictions are close to the condition of 'no block rotation' with slip along all fractures, but for a parabolic floor, curvature varied block behaviour, and slip along the bedding plane definitely occurred.

6.6.3 Magnitudes of Q force

The Q force magnitude acting at the block corners increased with the angle of friction along the bedding plane, and also increased with the angle of friction along the vertical fractures in all cases where movement along these fractures occurred. Figure 6.5 shown that for a given geometry and a low angle of friction equal to 10°, the Q force was approximately 1000kN (100 tonnes) for all calculated and modelled cases. As the angle of friction along the bedding plane increased to 30°, the modelled Q force in the planar floor model varied from 3400kN to 4300kN (340-430 tonnes) depending on the angle of friction along the vertical fractures. When compared to the analytical equations the Q force varied from 2600kN (260 tonnes) for a 'block rotation' condition to a maximum of 6100kN (610 tonnes) for a 'no block rotation' condition. As
expected, Q forces for the parabolic floor inclination were independent of the friction angle along the vertical fractures, where slip did not occur. For the 30° of friction angle of along the bedding plane, calculated Q forces on the curved plane varied from approximately 3000kN to 4600kN (300-460 tonnes),
while the more complex block movement in the model indicated $Q$ forces of approximately 2600kN to 2900kN (260-290 tonnes).

Figure 6.8  UDEC model showing concentrations of lateral stress at floor level
6.7 STRESS AT FLOOR LEVEL

Block interactions occur typically at floor level and as expected, lateral stresses concentrate in the upper edges of the blocks at floor level. Typical concentrations of lateral stress at floor level modelled by UDEC are shown in Figure 6.8.

Calculating the probable stress at floor level is required to estimate the safety factor of the floor. An equation to estimate maximum stress at floor level was derived using the geometry described in Figure 6.9. Contact area at the block corners is usually small, and depends on block movement, geometry, rock stiffness and the magnitude of Q force.

From the geometry shown in Figure 6.9, it can be shown that the contact area A at the block corner can be estimated by:

\[ A = \sqrt{\frac{2bL^2Q}{yE}}, \]  

(6.51)

where: 
- b = width of the block along the face equal of the width of the powered support,
- L = width of the block at perpendicular to the face,
- Q = interacting force at block corner,
- y = floor lift, and
- E = Young's modulus
Calculations of the contact area between the blocks using Equation (6.51) are within 10% of the contact area observed in the UDEC model.

The average stress at the contact area can be calculated using:

$$\sigma_{\text{average}} = \frac{Q}{A} = \sqrt{\frac{yEQ}{2bL^2}},$$  \hspace{1cm} (6.52)
From the geometry shown in Figure 6.9, maximum stress develops at the top of the block (floor level) and is equal to:

$$\sigma_{\text{max}} = 2\sigma_{\text{average}} = \sqrt{\frac{2yE\gamma}{bL^2}}.$$  \hspace{1cm} (6.53)

Even though the planar slope of the floor was included in the study, the non-planar increase in floor elevation would represent a more realistic behaviour (Peng, 1984). Using Equation (6.53), maximum stress $\sigma_{\text{max}}$ was calculated at the face for various floor shapes, bedding friction and powered support loads. These were:

Floor curvature: $\frac{x^2}{400}, \frac{x^2}{200}, \frac{x^2}{100}$ and $\frac{x^2}{50}$

Friction angle of floor: $10^\circ, 15^\circ, 20^\circ, 25^\circ, 30^\circ$ and $35^\circ$

Powered support load: 400, 500, 600, 700, 800 and 900 tonnes

The increase in slope, bedding friction and powered support loads appeared to have a significant influence on the magnitude of calculated stress $\sigma_{\text{max}}$. A reduction of contact area at the block corners and an increase of the Q force was experienced when floor curvature changed from $\frac{x^2}{400}$ to $\frac{x^2}{50}$.

The calculated floor stress levels presented in Figures 6.10 and 6.11 indicate a wide variety of possible stresses at floor level. For friction angles along the bedding plane varying from $10^\circ$ to $15^\circ$, and for low floor curvature ($y = \frac{x^2}{400}$) maximum floor stress varied from approximately 6 to 10 MPa.
depending on the load below the powered supports. For steeper floor curvature approaching $y = x^2/50$ and larger angles of friction ($\phi = 30^\circ$ to $35^\circ$), stress magnitudes increased dramatically, ranging between 32 and 50 MPa. These stress magnitudes are well within the range of the insitu strength of rocks typically found in coal mine floors (Karabin, 1999).
Floor curve shape $y = \frac{x^2}{200}$

Longwall Powered Support Loads
- 900 tonnes
- 800 tonnes
- 700 tonnes
- 600 tonnes
- 500 tonnes
- 400 tonnes

Floor curve shape $y = \frac{x^2}{400}$

Longwall Powered Support Loads
- 900 tonnes
- 800 tonnes
- 700 tonnes
- 600 tonnes
- 500 tonnes
- 400 tonnes

Figure 6.11  Calculated maximum lateral stress at floor level for $y=\frac{x^2}{200}$ and $y=\frac{x^2}{400}$
6.8 SUMMARY

The effect of moving blocks along fractured interfaces and its influence on stress distribution in the floor was studied. The theoretical solutions for interacting Q forces acting at the corners of the moving blocks compared well with the numerical model based on UDEC. From the three types of proposed block motions, block rotation appeared to dominate the movement of the non-planar floor shape in the model. The "no rotation but slip along all fractured surfaces" condition is usually restricted to blocks with high height to width ratios but for severely inclined floor surfaces, complex block movement can occur with all proposed block motions.

Even though the multiple block analyses presented here are similar to some limit equilibrium methods used in slope engineering (Hoek & Bray, 1981), the applications of the writers model are unique for longwall mining conditions. Results from the analytical and numerical model show that high lateral stresses can exist at the top of each moving block, and the magnitudes of interacting forces can be large enough to induce floor failure. The analytical solutions indicate that calculated interaction forces between the blocks increased with the distance towards the face, and as expected, stress transfer locations are usually at the unconfined floor level. The study indicated that maximum stresses at floor level increased mainly with (i) an increase in longwall support loads, (ii) the angle of friction along the bedding plane on which the blocks slide, and (iii) an increase in floor curvature.
The calculated and modelled contact area at the tip of each block appeared relatively small, allowing interacting lateral stress to exceed rock strength and induce rock failure at floor level. The block movement mechanism indicates that if floor failure occurs, it will begin at the surface and propagate lower down. The theoretical equations formulated in this study would give the practising mining or geotechnical engineer a useful tool for predicting floor stability at the longwall face. Even though the mechanism for moving blocks in the floor and the derived equations for interacting forces describe a new approach to analysing stress distribution in the immediate floor, further work is needed to fully assess the applications of this method.
Chapter 7

BRIEF OVERVIEW OF INFLUENCING FACTORS ON FLOOR
FAILURE AND PROPOSED GUIDELINES FOR PRACTITIONERS

7.1 INTRODUCTION

This chapter summarises a perspective that can help the practising mining engineer to determine what type of floor failure may occur and what remedial or preventative measures can be used to minimise potential problems. As part of risk assessment, an example of floor failure investigation of a proposed new mine was prepared to help the geotechnical engineer identify possible floor failure mechanisms at the longwall face. Floor failure types are described with an overview of influencing factors, together with recommended actions which may be taken to minimise the problem and any interference with coal production. This investigation is designed to help assess the risk of floor failure in green fields (proposed future mines), or in existing mines where longwall development panels are already mined.

This chapter covers all possible types of floor failure associated with longwall mining, including floor heave in gate roadways. Floor failure in gate roadways was not discussed in the previous chapters because its mechanisms are not new, having been understood and published for more than two decades (Gale, 1993, Afrouz, 1975). Floor heave in gate roadways is included here to
provide a complete coverage of all the possible types of floor problems a geotechnical engineer may encounter. The floor failure mechanisms presented here include:

(a) puncture of longwall supports into weak floor,
(b) buckling of stratified floor during excessive coal seam movement,
(c) compressive floor failure induced by multiple sliding blocks moving within the floor,
(d) floor failure in gateroads subject to high lateral stress concentrations, and
(e) floor heave adjacent to the goaf areas.

7.2 STRENGTH OF FLOOR – IMPORTANT TESTING PROCEDURES AND INFLUENCING FACTORS

To estimate the likelihood of floor failure, the strength of the floor rock needs to be known. The testing procedures described below must be understood and undertaken, to calculate the safety factors of various floor failure mechanisms that may occur, as well as regular floor core testing to build a database for the mining area.

Strength parameters that need to be known are:

- Uniaxial compressive strength (UCS),
- Triaxial strength properties
- Shear strength of the bedding planes, and
- Tensile strength.
Important parameters that reduce the strength of the above values are:

- Moisture content of rock,
- Reduction of rock strength with size of the rock specimen, and
- Rock texture, laminations or bedding planes.

### 7.2.1 Compressive Strength of Rock

Uniaxial compressive strength (UCS) and triaxial strength are the most common tests performed as part of geotechnical investigations in the mines. In these tests rock core is loaded to failure when the rock is unconfined (UCS) or confined (triaxial test). The tests are described in Figure 7.1. Samples in these tests usually fail along the maximum shear plane oriented at an angle of $\pi/4 - \phi/2$ to the sample axis, where $\phi$ is the angle of internal friction. Compressive rock strength can be presented on the graph of $\sigma_1$ versus $\sigma_3$ (Figure 7.1) where $\sigma_1$ represents axial stress, while the $\sigma_3$ is confinement stress applied to the sides of the samples. Triaxial strength of each rock type can be represented as a curve (or range of curves) on the graph while unconfined strength (UCS) values lay along the “y” axis of the graph.

Compressive strength is related to rock stiffness making it is possible to use geophysical logging (sonic velocity log) to estimate Young’s Modulus of rock.
and to approximate UCS. This is a particularly useful method when interpolating rock strength between tested strata horizons because it is not practical to test large number of samples. Compressive tests quantify the strength of homogeneous rock only, if the rock is anisotropic, other tests must be performed to quantify strength in directions other than vertical coring. Many other variables can influence rock strength if tested under triaxial conditions (Murrell, 1965).

### 7.2.2 Bedding plane Strength

Typically, sedimentary strata are non-homogeneous, where laminated or bedded sedimentary rocks are usually weaker when loaded in directions that enable shear failure to propagate along bedding planes or laminations. There are two ways to conduct strength tests. The first method involves a triaxial test of rock cored approximately 30° to the bedding planes. The plane of maximum shear would coincide with the weak bedding plane enabling cohesion and the angle of friction along the bedding plane (Indraratna, 1990) to be calculated. The second method involves testing the bedding plane in the shear box (direct shear test) with similar results. Examples of these tests are shown in Figure 7.2. All samples of 45-55mm in diameter were tested according to Australian Standards (AS4133.4.2).

Bedding strength may vary greatly, therefore it is important to select the weakest bedding planes for testing, if possible. If bedding planes are
extremely weak and cannot be tested, their strength can be estimated from the lower strength range of similar (tested) bedding planes.

7.2.3 Tensile Strength of Rock

The indirect tensile strength (Brasilion test) is usually conducted to determine the tensile strength of rock. Tensile strength of the bedding planes is expected to be lower than the tensile strength of the rock itself. The sample axis needs to be parallel to the bedding planes if the tensile strength of the bedding plane is to be tested. A summary of the rock properties can be presented in various forms, as shown in Figure 7.3.

7.2.4 Test Integrity

Over-estimation of floor strength often occurs when the laboratory strength of rock is not reduced to the in situ strength. The size effect, moisture and clay content of tested rock samples can often reduce rock strength by more than 100% of its tested value. One of the most common mistakes is to adopt the compressive strength of weakly laminated or bedded strata tested perpendicular to the bedding planes. It is essential that all the possibilities of strength reduction are well understood before evaluating the strength of the mine floor. Typical strength reduction parameters are summarised below.
a) Intact properties.

b) Residual properties.

c) Young's Modulus and strength of samples.

Figure 7.1 Methods of compressive strength
Figure 7.2  Determining strength of the bedding planes
Figure 7.3  Example of rock properties summarized for a particular area in the mine.

7.2.5 Effect of Moisture on Rock Strength (Sedimentary Strata)

The strength of tested rock samples increases as the moisture content is reduced. Recovered rock samples must be protected against moisture loss when transporting to the laboratory and the moisture content should always be measured immediately after the laboratory strength of rock has been
determined. A typical range of moisture content for sandstone, siltstone, mudstone, claystone and coal are given in Table 7.1 below.

Table 7.1  Typical Range of Rock Moisture Content

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Approximate Range of Moisture</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High-Medium Strength(&gt;30MPa)</td>
</tr>
<tr>
<td>Sandstone</td>
<td>0.5-4 %</td>
</tr>
<tr>
<td>Siltstone</td>
<td>2-5 %</td>
</tr>
<tr>
<td>Mudstone</td>
<td>2-6 %</td>
</tr>
<tr>
<td>Claystone</td>
<td>2-6 %</td>
</tr>
<tr>
<td>Coal</td>
<td>3-5 %</td>
</tr>
</tbody>
</table>

If the moisture content of tested samples is not within the specified range, the strength tests should be viewed with suspicion.

7.2.6  Reduction of Rock Strength with size of Rock Specimen

It is important for the geotechnical engineer to understand that the compressive strength of rock specimen decreases as the size of the tested sample increases (Hoek, 1980). An approximate relationship between the
insitu uniaxial compressive strength and sample diameter (for samples ranging 10-200mm in diameter) is approximated by:

\[ \sigma_e = \sigma_{e50} \frac{50^{0.18}}{d} \]  

(7.1)

where \( \sigma_{e50} \) = is the uniaxial compressive strength of a sample 50mm in diameter

\( d \) = diameter of tested sample in mm

This relationship, plotted in Figure 7.4, describes the uniaxial compressive strength (UCS) of rock sample versus its size. The typical size for a tested laboratory specimen consists of a rock core between 45-65mm in diameter. The geometry of floor failure below the support base is usually large enough to have the insitu strength. The calculated bearing capacity of the floor must be reduced to approximately half the laboratory strength of the floor specimen (using strength reduction versus specimen size), failure to observe this would result in overestimating floor strength.
7.2.7 Effect of Weak Laminations on Rock Strength

High lateral stress is one of the most common causes of strata failure in coal mines. High stress at low angles to weak bedding planes promotes shear failure along planes of weakness, which can occur at much lower values than
the strength of the actual rock mass. When estimating the in-situ strength of rock, bedding plane strength and other parameters such as direction of maximum stress must be considered.

7.3 RISK ASSESSMENT OF FLOOR FAILURE

To summarise, if the laboratory strength of floor rock is below 10-15 MPa, or if weak laminations or bedding planes are present within the immediate floor, failure about the longwall face needs to be investigated.

Five major floor failure mechanisms that can occur in a weak floor are:

1. Puncture of a weak floor below powered supports,
2. Buckling of stratified rock floor due to excessive movement of the coal face,
3. Compressive floor failure induced by movement of multiple sliding blocks within the floor,
4. Floor failure in gateroads that are subject to high lateral stress concentrations, and
5. Floor heave at the tailgate end when mining adjacent to large goaf areas.

These mechanisms are described below:
7.3.1 Puncture of Weak Floor below the Powered Supports

This type of failure occurs when peak pressure below the support base exceeds the floor bearing capacity. The probability of floor puncture can be predicted from floor strength tests. This failure (see Chapter 2) commonly occurs in claystone or weak mudstone floors, especially where reduction of floor strength can occur due to water saturation.

Typical modern powered supports consist of 2 leg shields with support capacities from 600 tonnes to more than 1000 tonnes acting on base pontoons with area ranging 3m$^2$ to 4.5m$^2$. A typical profile of stress distribution below the base under normal operations and adverse loading for a 2 leg 980 tonne powered support is shown in Figure 7.5a and 7.5b.

The front of the base can exert 2 to 3 times the average load under the pontoons, depending on type and stability of the supports under variable roof strata. Increased loading at the front toe of the base can cause the support to puncture the floor, as shown in Figure 7.6.
Figure 7.5  Typical floor loading distribution below the 2 leg powered supports under (a) normal operation and (b) adverse loading conditions.
An effective solution to floor puncturing is to use a positive base lift hydraulic pull out ram system that picks up the base toes before the powered support advance and restricts the toes from digging into the failed floor. The hydraulic pull out ram mounted on the base of the powered support in shown in Figure 7.7.
Identification of Floor Puncture:

(i) Puncture of the front support toe into the floor,

(ii) Localised floor failure around the toe of the support, and

(iii) Low angle shear fractures in floor strata ahead of the support toe.

Remedial actions:

1. Using “hydraulic pull out rams” mounted on the front of the support base,

2. Minimising floor saturation during mining by:
   - Reducing water sprays when shearing coal,
• Mining uphill (if possible) to prevent water build up,
• De-watering gate roadways, and
• If gas drainage is used, drill holes into the floor and use vacuum and gas pressure release to drain water from the floor strata.

3. Minimising longwall stoppages in wet areas,
4. Conducting geological mapping and regular floor strength testing to predict weak floor areas, and
5. Minimising longwall stoppages in areas where increased floor failures occur or are expected.

7.3.2 Floor Buckling due to Excessive Yield of the Coal Face.

This floor failure mechanism was described in detail earlier in Chapter 5. The floor failure, driven by lateral displacements of the coal seam towards the goaf is associated with weak bedding planes at a shallow depth below the floor, and excessive coal failure at the face. The immediate roof and coal at the face is laterally unconfined and free to move towards the goaf with only the floor strata, pinned by the powered supports, opposing this movement. If a weak coal-rock interface is present at the base of the coal seam, coal slip will occur with no damage to the floor. However, if the coal-floor interface is strong, then the floor will restrict coal displacement towards the goaf and large shear forces will form close to the coal-floor boundary. If a weak bedding plane
exists at a shallow depth below the floor, the shear force may fail the bedding resulting in large lateral stresses developing in the upper floor that resist the lateral displacement of coal towards the goaf. A relatively thin floor above failed bedding may buckle or fail in compression, manifesting itself as floor heave in front of the powered supports. This floor failure mechanism is shown in Figure 7.8.
Identifying the Problem:

1. Unfavourable strata sequence obtained from geological reports showing:
   - Strong coal–floor interface,
   - Weak bedding plane/planes located within the first 0.2-0.3m of the floor,
   - Weak coal seam, and
   - Strong roof strata causing a high vertical stress peak just ahead of the longwall face.

2. Overburden depth in excess of 200m,

3. Excessive spalling of the coal face,

4. Floor buckling below the armoured face conveyor ahead of powered supports, and

5. A high support load resulting in a pinning action below the support base that restrict floor movement towards the goaf.

Remedial actions:

1. Use “hydraulic pull out rams” mounted on the front of the support base if the base punctures a weak and fractured floor,

2. Conduct geological mapping and regular bedding plane strength tests to predict weak floor areas,

3. Minimise longwall stoppages in areas where increased floor failures occur or are expected, or

4. Excavate weak parts of the floor.
7.3.3 Compressive Floor Failure at Floor Level induced by Multiple Sliding Blocks Moving within the Floor.

This mechanism of floor failure was described in detail earlier in Chapter 6. Floor failure is associated with weak bedding planes and regular formation of mining induced sub-vertical shear fractures deep within the floor. Floor blocks interacting within a rising floor can induce a lateral compression at floor level that can exceed floor strength. The mechanism is depicted in Figure 7.9.

Figure 7.9 Mechanism of floor failure associated with stone block movement within the floor
Identifying the Problem:

1. An unfavourable geological sequence obtained from cored holes that show:
   (i) Weak bedding plane/planes located 1-3m below floor level,
   (ii) Strong roof strata causing a high vertical stress peak just ahead of the longwall face,

2. Large lateral stress relief towards the longwall face,

3. Overburden depth in excess of 200m,

4. Large longwall support loads, and

5. Periodic floor buckling below the armoured face conveyor ahead of the powered supports.

Remedial Actions:

1. Use "hydraulic pull out rams" when negotiating floor heave ahead of the longwall supports, or when the toe of the support base penetrates into weak and broken floor,

2. Conduct geological mapping, identifying bedding plane depth and strength to predict where failure may occur, and

3. Minimise longwall stoppages in areas where increased floor failures occur or are expected.
7.3.4 Floor Failure in Gate Roads exposed to High Lateral Stress Concentrations

If the mine roadway or longwall panel is situated in an undesirable direction, significant lateral stress concentrations can occur at longwall corners. Figure 7.10 shows stress concentrations occurring at the corners of a square hole in a stiff plate exposed to the stress flow, with stress relief being experienced at the top and bottom of the square opening (Figure 7.10).

Figure 7.10 Example of stress concentrations in a solid plate subject to uniaxial stress application.
As stress magnitudes increase with depth (Chapter 4), knowledge of stress directions becomes very important for mine design. In general, roadways driven at a high angle to maximum horizontal stress will experience stress concentration.

**GOOD CONDITIONS**

**BAD CONDITIONS (SAG)**

**Guttering On Left Hand Side**

**Guttering On Right Hand Side**

**Major Horizontal Stress** is the direction of major pressure which usually acts in one main direction.

Figure 7.11 General concept of variation in roof conditions with driveage direction in elevated horizontal stress
concentrations in the roof and the floor during mining severe enough to cause deformation, while roadways cut parallel to the major lateral stress generally experience good conditions. Roadways inclined in the direction of maximum stress will experience stress concentration on the roadway corner that leads into the stress, as shown in Figure 7.11. This can cause roof and floor failure biased to that side (roof gutter and floor heave). Stress concentrations ahead of the roadway face can also be influenced by other factors such as roadway sequencing and coal cutting procedure. Roadway directions, choice of the “first driven roadway”, coal cutting sequence at the face, and good roof/rib reinforcement design are therefore essential to improve strata conditions (Gale, 1993).

An advancing longwall is essentially a “wide” roadway, but if the panels are placed the in the wrong direction, stress concentrations would form either in the maingate roadway area or the tailgate side. If the position of longwall corner leads into the lateral stress flow, high stress concentrations can be expected during mining. This increased lateral stress causes additional damage to the initial roof and floor failure that developed during roadway driveage. Figure 7.12 shows this failure in the tailgate roadway of the first driven longwall block. The roof sag and floor heave indicates that the initial roof and floor failure was biased to one side. Underground stress change measurements at the longwall corner leading into the high stress flow are shown in Figure 7.13. The summary of stress concentrations at the longwall corner for various angles of advance to the maximum stress direction is shown in Figure 7.14.
Once in-situ pre-mining stresses are known, magnitudes of lateral stress concentrations can be estimated using numerical modelling. A 3-dimensional model was constructed to evaluate lateral stress concentrations at longwall corners. These concentrations versus the distance from the longwall corner (graphed in Figure 7.15) can be used together with the floor strength information to evaluate the likelihood of floor failure.

Floor failure caused by lateral stress is most common within the stage loader

Figure 7.12  Tailgate roof and floor failure ahead of the longwall face – biased to one side
area (maingate). Many mines are restricted in the directions they can advance the longwalls, and thus some stress concentrations and floor failure are highly probable. As described in Chapter 4, increase in lateral stress occurs with depth of cover, therefore deeper mine workings would be more likely to experience floor failure.

**Identifying the Problem:**

1. Unfavourable geological sequence obtained from cored holes showing:
   (i) Low compressive strength of the floor, and
   (ii) Weak bedding plane/planes located at shallow depths below floor level.

2. Large lateral stress concentrations at the longwall corner,

3. Overburden depth in excess of 100m, and

4. Floor buckling in the gate roadway just ahead of the moving longwall.

**Remedial Actions:**

1. Tailgate: - Erect wooden cribs and props to confine the floor, Brush floor regularly,
   Maingate: - Consider floor bolting

2. Use "hydraulic pull out rams" when advancing supports in the gate roadway,
3 Conduct geological mapping, identify bedding plane depth and strength to predict where this type of failure may occur,

4 Minimise longwall stoppages in areas where increased floor failures occur or are expected, and

5 Consider stress directions when planning longwall panels.

7.3.5 Floor Heave at the Tailgate End when mining adjacent to Large Goaf Areas.

The probability of floor heave at the tailgate corner depends on floor geology, the depth of overburden, and the size of the chain pillars protecting the tailgate roadway from the adjacent goaf. If the floor is weak, or weakly bedded, the stress required for the floor to fail is lower than it would be in massive floor strata.

If the chain pillars are small and fail some distance past the longwall face (into the goaf), they shed the vertical load towards the longwall face. Increased vertical abutment loads at the tailgate area are detrimental to the longwall itself and cause the tailgate floor to fail. An increased vertical load in the tailgate area can puncture the floor below the chain pillars or buckle the laminations just below floor level. If small pillars yield, the coal pillar will expand and induce large side forces that can buckle the roadway floor.
Figure 7.13 Underground stress change measurements showing elevated stress concentration at the gate corner - undesirable orientation of the longwall (after Matthews, 1992).
Figure 7.14  Summary of stress concentrations at the longwall corner versus angle of stress to the longwall advance (after Matthews, 1992)
Pillar strength formulae have been proposed by many authors on the basis of back analysis of full size pillar behaviour and laboratory testing of large blocks of coal. Wilson (1972) showed that a pillar with a width to height ratio greater than 10 is, for all practical purposes, classified as indestructible. There are many publications on pillar strength and these can be referred to for more information.

**Identifying the Problem:**

1. Unfavourable geological sequence obtained from the cored hole showing:
   
   (i) Weak floor strata,
(ii) Weak bedding planes located 0.1-1m below floor level, and

(iii) Weak coal seam.

2 Small chain pillars indicating excessive yielding,

3 Old goaf adjacent to the tailgate, and

4 Overburden depth in excess of 200m.

**Remedial Actions:**

1. Increase chain pillar size,

2. Test the strength of floor rock and bedding planes,

3. Conduct geological mapping to identify areas where this type of failure may occur,

4. Minimise floor saturation in the tailgate roadway,

5. Erect wooded cribs or props to confine the floor in the tailgate area, and

6. Optional use of Breaker Line Supports (Figure 7.16) or spare old powered supports in the tailgate roadway (ahead of the longwall face) to minimise roof to floor convergence.
7.4 ILLUSTRATIVE EXAMPLE OF FLOOR FAILURE INVESTIGATIONS

A typical example of floor failure investigation has been constructed to demonstrate how to predict and minimise floor failure while mining coal using the longwall method. This example studies a proposed mine site where a number of longwall blocks are to be mined. Typical tools available are core samples and strength tests from exploration holes, geophysical logs measured within the exploration holes, regional stress fields typical of the area, and experience from other mines mining the same seam.
7.4.1 Risk Assessment of Floor Failure in the Mine

The plan is to develop an underground coal mine, using the longwall method, with 12 longwall blocks 250m wide and 2.5km long. Initial studies recommended retreat longwalls towards already completed trunk roadways driven West to East from an existing 350m deep shaft.

Preliminary explorative drilling revealed variable floor strata with a relatively strong siltstone floor in the Northern section, medium to weak mudstone at the centre of the lease, and weak claystone in the Southern section. Weak laminations and carbonaceous beds at a shallow depth into the floor were discovered in strong siltstone strata, while a thin coal seam was present some 2m below a mudstone floor found in the central region. The claystone floor appeared weak and prone to weakening when exposed to moisture, but no bedding planes were seen. Because management was concerned about mining the southern area where a weak claystone floor was found, they appointed a geotechnical engineer to conduct a risk assessment. The geotechnical engineer chose the investigation procedure given in this thesis.
Investigation Procedure

1. INFORMATION STAGE
Compile data base required for the investigation.

Geology

<table>
<thead>
<tr>
<th>Area</th>
<th>Floor type</th>
<th>Depth into the floor</th>
<th>Floor strength (Laboratory)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northern Area</td>
<td>Siltstone containing frequent weak bedding planes at spacing of less than 50mm. Coal/Floor interface is relatively strong. Coal is highly cleated at 20° to the longwall face.</td>
<td>0-0.3m</td>
<td>UCS 60 MPa (perpendicular to bedding) E = 15 GPa. UCS 10-20 MPa (at 30° to the bedding) Bedding properties: c = 0.5-1 MPa φ = 20-25°</td>
</tr>
<tr>
<td></td>
<td>Sandstone – massive</td>
<td>0.3-6m</td>
<td>UCS 80 MPa E = 16 GPa.</td>
</tr>
<tr>
<td>Middle of lease</td>
<td>Massive mudstone</td>
<td>0-2m</td>
<td>UCS 40 MPa E = 14 GPa.</td>
</tr>
<tr>
<td></td>
<td>Thin coal seam</td>
<td>2-2.1m</td>
<td>UCS 15 MPa E = 3 GPa.</td>
</tr>
<tr>
<td></td>
<td>Massive sandstone</td>
<td>2.1-8m</td>
<td>UCS 80 MPa E = 17 GPa.</td>
</tr>
<tr>
<td>Southern Area</td>
<td>Claystone</td>
<td>0-3.5m</td>
<td>UCS 28 MPa E = 9 GPa.</td>
</tr>
<tr>
<td>Coal Seam thickness</td>
<td>Massive sandstone</td>
<td>UCS 80 MPa</td>
<td>E = 16 GPa</td>
</tr>
<tr>
<td>---------------------</td>
<td>-------------------</td>
<td>------------</td>
<td>-------------</td>
</tr>
<tr>
<td>Ranging 2.8-3.1m</td>
<td>Coal Seam Strength</td>
<td>UCS 20 MPa</td>
<td>E = 3 GPa</td>
</tr>
</tbody>
</table>

### Additional Information

<table>
<thead>
<tr>
<th>Depth of cover</th>
<th>Northern Area</th>
<th>Middle of the lease</th>
<th>Southern Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>380-450m</td>
<td>360-380m</td>
<td>300-360m</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Virgin Stress Measurement (Underground)</th>
<th>Pit bottom Area (Middle of the lease)</th>
<th>Stress MPa</th>
<th>Dip/Bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's Modulus E = 15 GPa</td>
<td>Poissons Ratio 0.22 (measured)</td>
<td>σ₁ 16</td>
<td>2°/25°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>σ₂ 11</td>
<td>6°/116°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>σ₃ 9</td>
<td>-88°/255°</td>
</tr>
<tr>
<td></td>
<td></td>
<td>σᵥ 9</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Proposed longwall width</th>
<th>250m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proposed direction of longwall retreat</td>
<td>170°</td>
</tr>
<tr>
<td>Chain pillar size (proposed)</td>
<td>30m wide, 100m long</td>
</tr>
<tr>
<td>Location of old goafs</td>
<td>To the East of retreating longwalls (First longwall – no adjacent goaf)</td>
</tr>
<tr>
<td>Water table</td>
<td>Close to the surface – pore pressure equal to the depth of cover</td>
</tr>
<tr>
<td>Capacity of the proposed Longwall Supports</td>
<td>750 tonnes set</td>
</tr>
<tr>
<td></td>
<td>900 tonnes yield</td>
</tr>
<tr>
<td>Support Type</td>
<td>2 leg shields</td>
</tr>
<tr>
<td>Support base area (pontoons nett)</td>
<td>4m²</td>
</tr>
</tbody>
</table>
## 2. EXAMINE INTEGRITY OF DATA

| (i)  | Check whether the moisture content of the tested rock samples are within normal limits. Moisture content is done immediately after completing the strength tests and should be stated in the report. If the reported moisture is too low, rock strength is overestimated. |
| (ii) | Check the diameter of tested rock sample and estimate insitu rock strength using Equation 7.1. |
| (iii) | Examine rock texture and estimate whether rock strength would be reduced if loaded at a low angle to the bedding planes or laminations |
| (iv) | Note any weak discontinuities (not reported) within the rock core. |
| (v) | Estimate the internal friction angle within the intact rock by either:  
  - Measuring the angle between the sheared plane and loading axis.  
  - Examining triaxial test graphs if available. |
| (vi) | Calculate intact rock cohesion using the angle of internal friction and stresses acting in normal and parallel directions along the shear fractures. |
| (vii) | For bedding plane strength tests; if samples tested at 30° to the bedding planes, calculate the angle of friction and cohesion from triaxial tests. |
If bedding planes are too weak to be tested, the estimated angle of friction and cohesion should be lower than the weakest bedding that was tested.

3. ANALYSIS STAGE

Note: This example is intended to be an introductory guide only. Due to many variables associated with strata and longwall mining geometry, this example does not provide safety factors. It is however possible to conduct an extensive underground measurement program to calculate safety factors that can be used to quantify floor failure risk in mines.

Puncture of Weak Floor below the Longwall Powered Supports

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range of the Powered Support Loading</td>
<td>600-900 tonnes</td>
</tr>
<tr>
<td>Area of base pontoons</td>
<td>4m²</td>
</tr>
<tr>
<td>Range of Maximum Floor Load (available from the manufacturer of the supports)</td>
<td>3-4.5 MPa</td>
</tr>
<tr>
<td>Northern Area:</td>
<td>Compressive strength of intact floor is high (Laboratory UCS 60MPa). Bedding planes do not significantly affect</td>
</tr>
<tr>
<td>Location</td>
<td>Compressive strength of the immediate floor is high (Laboratory UCS 40 MPa).</td>
</tr>
<tr>
<td>-------------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Middle of the Lease:</td>
<td>this type of floor failure.</td>
</tr>
<tr>
<td>Southern Area:</td>
<td>Compressive strength of the immediate claystone floor is low (Laboratory UCS 28 MPa). Size related strength reduction from 50mm sample to 1m indicate insitu strength of approximately 16 MPa (or lower). This strength value is higher than the calculated maximum pressure below the base of the support. If the clay floor becomes saturated, the possibility of floor failure increases. Water management in this area needs to be strict with a continuous monitoring system in place to manage water inflow.</td>
</tr>
</tbody>
</table>
Buckling of the Stratified Rock Floor due to Excessive Movement of the Coal Face towards the Goaf

| Northern Area: | Frequent weak bedding planes present within the immediate floor. Compressive floor strength perpendicular to the bedding planes is high (Laboratory UCS 60MPa). Compressive strength at 30° to the bedding planes is very low (Laboratory UCS 10-20 MPa). That indicates an insitu floor strength is 5-10 MPa.

Bedding properties are low with cohesion and angle of friction equal to:

$c = 0.5-1 \text{ MPa}$

$\phi = 20-25^\circ$

(Bedding spacing<50mm)

Coal/Floor interface is strong with the coal heavily cleated at a low angle to the longwall face. Using the charts in Figure 5.8-Chapter 5, probable lateral stress increase in the immediate floor would range from 5-10 MPa. This level of lateral stress could regularly exceed the strength of highly laminated floor with weak bedding planes.

The probability of floor failure in this area is very high.
<table>
<thead>
<tr>
<th>Middle of the Lease:</th>
<th>The floor rock is massive with no weak bedding planes. Compressive strength of the immediate floor is high (Laboratory UCS 80 MPa). <strong>Risk of this type of floor failure is low.</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>Southern Area:</td>
<td>Compressive strength of the claystone floor is low (Laboratory UCS 28 MPa). No weak bedding planes are present. This floor does not support this type of failure. <strong>Risk of floor failure is low.</strong></td>
</tr>
</tbody>
</table>

**Remedial Actions:**

The weakly bedded floor is only 0.3m thick. The pullout jacks mounted at the toe of powered supports would probably handle floor puncture into the buckled floor. If the longwall supports advance is hindered, the full or partial layer of weak floor can be cut, together with the coal, until better ground is encountered.
### Compressive Floor Failure induced by Multiple Sliding Blocks moving within the Floor

<table>
<thead>
<tr>
<th>Area</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Northern Area:</strong></td>
<td>No bedding planes are present deeper than 1m below the floor. Risk of floor failure is low.</td>
</tr>
<tr>
<td><strong>Middle of the Lease:</strong></td>
<td>A thin coal seam exists 2-2.1m below the floor, which would be expected to fail below the longwall face. The friction angle of the intact coal layer is expected to be approximately 35°, however, this value could fall if severe fracturing develops within this plane of weakness. The mudstone floor above the thin seam is medium - weak in strength (laboratory strength 40 MPa) and formation of the sub-vertical fractures is probable. The charts in Figures 6.10 and 6.11 (Chapter 6) indicate the magnitude of stress at floor level would range from 17 to 50 MPa depending on floor curvature. Floor curvature is not known but a numerical model can be constructed to indicate what the curvature may be. <strong>Risk of floor failure is moderate and subject to further investigations.</strong></td>
</tr>
<tr>
<td><strong>Southern Area:</strong></td>
<td>Compressive strength of the claystone floor is low (Laboratory UCS 28 MPa). No weak bedding planes are present within the floor. The floor does not support this</td>
</tr>
</tbody>
</table>
type of failure.

Risk of floor failure is low.

## Gate Road Floor Failure due to Concentrations of High Lateral Stress

**Northern Area:**

In-situ stress measurements at pit bottom indicate the maximum compressive stress of 16 MPa is horizontal at a bearing of 25° from North. Lateral stress depends on rock stiffness and depth of cover. Depth of cover in this area is 20 to 70m deeper than the pit bottom and therefore lateral stress is expected to increase slightly. Young’s Modulus of floor strata, and the rock where stress measurements were taken are the same.

The proposal to extract the longwall at 170° indicates that a ‘stress notch’ would occur on the tailgate side of the first longwall (see Figure 7.17– Chapter 7). Further longwalls would be excavated in the ‘stress shadow’ so that stress concentrations would only affect the first longwall.

The immediate floor is weakly bedded with low lateral...
Concentrations of virgin stress at the tailgate corner can be estimated from Figure 7.15 – Chapter 7 as 2.1 times the virgin maximum stress. With maximum stress concentrations in excess of 33 MPa floor failure can be expected.

Risk of floor failure is very high.

Remedial Actions:

- No change in direction of longwall mining is recommended due to maingate protection from any excessive lateral stress (Maingate roadway is more important than the tailgate roadway).
- Wooden cribs, props and other support can be used to provide confining stress onto the floor to minimise this type of failure.
- Floor brushing (where possible) can assist to keep the tailgate roadway in a reasonable condition.

Middle of the Lease:

The insitu strength of massive mudstone in the immediate floor (Laboratory test 40 MPa) would be less than 23 MPa. Stress concentrations of 29 MPa are expected, indicating a probable failure of the mudstone.
<table>
<thead>
<tr>
<th>Southern Area:</th>
<th>Compressive strength of the claystone floor is low (Laboratory UCS of 28 MPa with insitu strength less than 16 MPa). Stress concentrations of 18 MPa are expected indicating a probable failure of the claystone floor.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Risk of floor failure is high.</strong></td>
</tr>
<tr>
<td></td>
<td><strong>Remedial Actions:</strong></td>
</tr>
<tr>
<td></td>
<td>As per Northern Area</td>
</tr>
</tbody>
</table>
Figure 7.17 Proposed advance of Longwall Face

Floor Heave in the Tailgate Roadway adjacent to Large Goaf

| Northern Area: | Chain pillar stability needs to be assessed, to determine whether chain pillar failure occurs in the goaf area. If the chain pillar fails, vertical stress would increase at the tailgate corner raising the possibility of floor heave in the tailgate roadway. |
The width to height ratio of the chain pillar is approximately equal to 10. Coal seam tests indicate a strength of 20 MPa (Laboratory test). The vertical stress increase on the chain pillar was estimated using equation 4.2 in Chapter 4. Under maximum depth of cover (450m), the average load on the 25m wide chain pillar located between two fully formed goafs (supercritical width) was 97.5 MPa. Weak bedding planes within the floor may reduce pillar strength. Numerical modelling needs to be carried out to investigate whether the chain pillars would cope with overburden loads.

**Risk of floor failure is moderate.**

**Middle of the Lease:**

Width to height ratio of 10 indicate that the chain pillars are indestructible (Wilson, 1972),

**Risk of floor failure is low.**

**Southern Area:**

Claystone below the seam is stronger than the coal and should not fail. If the goaf is inundated with water, it may be possible the saturated claystone floor could fail.

**Risk of floor failure is low to moderate**

**Remedial Actions:**

Even a small increase in pillar width can dramatically improve stability of the chain pillar. More information on saturated clay strength should be undertaken.
4. RESULTS OF THE INVESTIGATION

The investigation concluded that, contrary to the initial concern of management, the likelihood of floor failure is greatest in the Northern Area, not the Southern area. Even though the rock strength perpendicular to the bedding was high (60 MPa), weak bedding planes were identified as the major cause of probable floor failure. The study provided the remedial actions needed to minimise problems with floor failure and indicated that the company may need to cut 0.3m of weak floor, together with the coal. This action would provide the mine with trouble free production, however, cost of production may increase due to a higher ash content of the run of mine product. Low to moderate risk of floor failure still exists in other parts of the mine, however, appropriate steps can be taken to minimise any problems.

7.5 SUMMARY

This chapter provides the risk assessment of floor failure at the longwall based on the overall knowledge of floor failure given in this thesis. A detailed summary leading towards risk assessment has been constructed to provide guidelines for the geotechnical engineer to follow.
The first part of this chapter conveys information on floor testing procedures crucial to determining the likelihood of floor failure. Many factors that are often neglected are moisture content of tested rock samples, reduction of rock strength with specimen size, or failure to consider bedding plane strength. Such neglect can lead to overestimating floor strength and minimising accurate predictions of floor failure.
8.1 CONCLUSIONS

Knowledge gained from previously published work has been summarised into a comprehensive analysis and discussions on floor failure and extended to develop new theories of failure. Analysis was based on established rock failure criteria, underground measurements, visual observations, analytical formulations and numerical modelling.

Underground monitoring (Chapter 3) provides a basis for evaluating potential fracture formation mechanisms within the floor, while field monitoring provides data that validate the numerical model developed to simulate various floor failure mechanisms near the longwall coal mining face. This analytical work not only describes potential floor failure mechanisms, it also quantifies the parameters associated with individual types of failure mechanisms. The results available from the analytical and numerical (FLAC) models provide data for evaluating floor stability at the longwall face. The factors that influence strata stability have been studied in depth to provide more accurate risk assessment when examining potential floor failure.
Primary conclusions based on this study are as follows:

8.1.1 Extension of Current State-of-the-Art

Traditional views of floor failure were related to rock strength criteria and floor bearing capacity but they did not always represent true behaviour. When a very weak floor is present, soil mechanics equations appear to fit the type of failure and conditions that develop at the longwall face, but there were no other models which could explain floor failure in stronger floors. As far as this writer is aware, this study offers the only analytical and numerical models that describe floor failure in longwall mining.

8.1.2 Floor Design based on Bearing Capacity

Low floor bearing capacity has always been associated with weak floors but severely fractured floor strata can provide low bearing capacity at the floor surface. The failure mechanisms described in this study are characterised by failure at floor level, and quite frequently this softening mechanism reduces floor strength to a fraction of its intact strength, resulting in floor puncture when loaded from above.
8.1.3 Influence of Bedding Planes on Floor Failure

Weak bedding planes typically found in sedimentary strata have probably the greatest influence on floor failure, which should not be surprising since bedding planes are generally weaker than the surrounding rock. Lack of routine testing of bedding strength and scant information on weak bedding discontinuities that are often omitted in geological logs indicate the extent of the problem, particularly for manager who often despair when unexpected strata failure occurs in rock of high strength. Tests of rock samples cored at 30° to the bedding planes (see Chapter 4) reveal the true strength of the rock mass and explain the discrepancies that often exist. Bedding planes that fall apart during drilling can be assumed to have little or no cohesive strength while their angle of internal friction can be determined by laboratory testing.

8.1.4 Influence of High Lateral Stress on Mine Floor

Underground stress measurements in Australia indicate that maximum compressive stress is predominantly horizontal (Indraratna, Nemcik and Gale, 2000). It is no surprise therefore that lateral shearing (mainly along the bedding planes) or buckling of bedded strata is often the primary cause of floor failure in coal mines (see Chapter 4). Floor failure that is often biased to one side of the mine roadway is a typical indication of lateral stress in play.
Even though lateral stress along the longwall face (at floor level) is usually relieved towards the goaf, deep lateral stresses below the floor are usually very high. Numerical modelling shows that high stress concentrations below the face can induce failure deep within the floor, which differ from the common belief that floor failure is a “surface” phenomenon caused by longwall mining.

8.1.5 Relevance of Floor Deformation monitoring at Longwall Face

Underground monitoring of floor deformation at the longwall face was especially conducted for this project to investigate floor failure mode (see Chapter 3). Monitoring was carried out using geotechnical instruments consisting of: multiple anchor sonic extensometers, shear strip instrument and observation holes. Frequency and orientation of fractures visible on the floor surface ahead of the longwall supports were documented, exposed fracture surfaces visible at the longwall face just above the undercut floor were photographed, and their dip and orientation was recorded. Based on the monitoring results, a computational model (FLAC) was constructed to validate measured results (section 4.4.3 in Chapter 4). Modelled floor behaviour was similar to the underground measurements, which proved that computer modelling does provide solutions to complex problems such as mine floor failure.
Several important issues need to be emphasised in the study of floor failure mechanisms:

- Shear fracture zones in the floor occurred ahead of the longwall face in a periodic manner, dipping at steep angles towards the goaf (Figures 3.14 and 4.22);
- The upper floor appeared to move towards the goaf more than the deeper strata, indicating that the top section of floor sheared along bedding planes towards the goaf (Figure 3.13);
- Rapid water inflow into the monitoring holes just ahead of the longwall face indicated severe floor failure in that region (It was not possible to empty water out of the holes using a high capacity pump).

This monitoring program provided excellent data to study the floor failure mechanisms given in this thesis. Further monitoring of floor failure at the longwall face is recommended to confirm the data, and investigate whether similar results can be obtained in different strata, and at a shallower depth.

8.1.6 Interpretation of Primary Failure Mechanism

Several primary fracture mechanisms that occur in the floor ahead of the longwall face were described in this thesis. Factors influencing primary fractures were quantified to enable a more accurate prediction of floor failure. Development of these fractures was based on the Mohr-Coulomb criterion of
rock failure and the stress field that typically occurs in the vicinity of the longwall face.

The following conclusions are made based on the current study:

- Primary floor fractures develop ahead of the longwall face in response to high stress concentrations. They primarily consist of near-vertical shear fractures, failure along weak bedding planes and tensile fractures;

- High lateral shear stress can be generated along bedding planes by a combination of:
  - lateral stress relief towards the opening,
  - floor strata bending (flexing) adjacent to the goaf edge and
  - maximum compressive stress acting at low angles to bedding planes.

- Primary floor failure provides impetus for a secondary floor failure (section 4.3) where strata movement is reactivated along pre-existing (primary) fracture surfaces, causing large floor displacements and disruption to mining operations.

8.1.7 Stress Field at the longwall Face

Fracture formation underground occur in response to high stress concentrations surrounding an excavation, therefore it is important to measure and understand stresses at the point of interest. The origin of underground stresses and how they concentrate has been detailed in Chapter 4. One
important aspect that has been reported in this study is the increase of lateral stress in proportion to the depth of cover (Section 4.2.2). Most researchers believe the tectonic component of lateral stress is fixed and does not increase with depth, but as explained in Chapter 4, the lateral component of tectonic stress is directly proportional to stress confinement \( \sigma_3 \) (usually vertical stress) that increases with depth. This finding agrees with the underground stress measurements conducted in Australia by the writer (Nemcik, 1998) and other locations around the world (Fairhurst, 1986).

Based on current study it can be further concluded that:

- The weight of overburden strata generates vertical stress that increases with depth;
- Stress measurements indicate that the large portion of underground horizontal stress is tectonic in origin;
- Stress measurements and theoretical work presented in this thesis indicate that maximum horizontal compressive stress in the pre-mining state increases with depth of cover, and is usually greater in magnitude than vertical stress (Figure 4.9);
- It is not possible to estimate pre-mining horizontal stress. The direction of maximum compressive horizontal stress is often unique and can be verified by stress measurements;
- Magnitudes of vertical and lateral stress concentrations can either be measured or numerically determined depending on excavation geometry;
• Calculation of approximate vertical load increase ahead of the longwall face can be approximated using Equations 4.1 and 4.2 derived in this thesis.

8.1.8 Aspects of Numerical Modelling

A numerical model using FLAC was constructed to simulate and monitor primary floor failure mechanisms and displacements ahead of the longwall face (Chapter 4). Reasonable agreement between field data and numerical predictions verified the numerical model, which was used to model primary fracture zones of progressive failure that occur periodically.

On the basis of numerical modelling the following conclusions can be made:

• The extent of floor failure is related to the overall strength of floor strata, the magnitude of shear and compressive stress concentrations, and the presence of weak bedding planes.
• Significant increase of maximum compressive stress ahead of the coal face was predicted in the modelled floor (Figure 4.20). Such large stress concentrations indicate the likelihood of floor failure,
• Periodic nature of floor failure shows variation of displacements over a short distance, but despite this, a reasonable correlation between monitored and computed results was obtained.
• Shear stress magnitude in the floor is at its maximum just ahead of the coal face, where the likelihood of floor failure is high,
• Bedding plane failure occurs due to strata flexing, when located adjacent to a progressively widened goaf. This failure can propagate far ahead of the longwall face.

• Near-vertical shear fractures can develop in response to maximum vertical stress concentrated ahead of the longwall face (Section 3.4.4).

• The model could correctly simulate periodic development of near vertical shear zones that were steeply dipping towards the goaf (Figure 4.22) as seen underground;

Future developments in numerical techniques (eg. Discrete element and coupled FEM-DEM techniques) may improve the current FLAC (Fast Lagrangian) model’s accuracy and enable the detailed 3-dimensional longwall excavation studies which are not feasible today. Despite that, the writer believes the current fast lagrangian finite difference modelling methods are sufficiently effective, as long as joint patterns and excavation geometries are not too complex.

8.1.9 Aspects of Buckling Mechanisms

This type of floor failure typically occurs in deeper mines where weak bedded floor strata, weak coal seam, and a strong roof, predominate. High vertical stress concentrations close to the longwall face yield the coal seam and
generate movement of fractured coal towards the goaf. This process shears the floor bedding planes and buckles the upper floor portion against the longwall supports which pin the bedding planes together. This common floor failure is a typical example of how otherwise strong, high bearing capacity strata can fail. Operational problems usually occur when severe floor buckling tilts the face conveyor, or when severely fractured floor is too weak to support the tips of the powered supports that dig into it.

Based on this current study several conclusions on floor buckling failure were drawn:

- A strong coal-floor interface must be present for shear stress to transfer between fractured coal face and floor (Figures 5.2 and 5.3);
- Buckling floor failure can only occur if weak bedding planes are present in the upper floor strata. The likelihood of this failure increases with the number of weak bedding planes within the floor strata;
- Since the powered supports reacts against the buckling upper floor, the angle of friction along the bedding can be used to determine the maximum reaction force that the powered supports can supply (Equation 5.1). An increase in the powered support loads increases the reaction forces that oppose upper floor movement,
- A small angle of friction and low cohesion along the bedding planes can increase the likelihood of buckling (Figure 5.8).
This study offers a new approach to determining whether buckling floor failure will develop or not. Even though the current analysis describes this failure mechanism in some detail, further work is needed to improve analytical and numerical approaches. These would require better accuracy of parameters such as bedding plane properties and anisotropic rock mass characteristics.

8.1.10 Aspects of Multiple Sliding Block Model

This floor failure mechanism (Chapter 6) was developed as part of this research study because it can occur if a weak bedding plane exists 1-3m below the floor. It is based on the lateral shearing of weak bedding planes due to floor strata flexing that typically occurs at the edge of the goaf and the formation of near-vertical shear fractures occurring at regular intervals ahead of the longwall face. Once the block matrix forms, interaction between them during floor movement can induce a large compressive stress state at floor level. Two types of block motion were studied, block rotation, and slip with no rotation along failed and inclined bedding surface. Analytical solutions representing this failure were developed (Chapter 6) enabling the stress range at floor level to be calculated. A UDEC numerical model was also formulated to simulate sliding block behaviour and compare the predictions with the analytical model.

The multiple sliding block model indicates the following:
• This mode of failure can generate large compressive stresses at floor level. The analytical calculations quantify compressive stresses that can develop at the longwall face (Figures 6.10 and 6.11).

• Floor geometry, rock properties and powered support loads influence floor failure.

• Closed form analytical equations enable stresses within the floor to be calculated and indicate the likelihood of floor failure (Eqn 6.53).

• UDEC numerical predictions closely agree with the analytical model (Figures 6.5 to 6.7).

• Results from both analytical and numerical methods indicate that stress magnitudes between blocks increase with the distance towards the longwall face, thus, maximum failure would develop close to the coal face as soon as the floor is exposed.

• The study also indicates that maximum stress at floor level also increases with longwall support loads, angle of friction along the bedding plane on which the blocks slide, and an increase in floor inclination that occurs at the edge of the goaf (Figures 6.10 and 6.11).

8.1.11 Remedial and Preventive Measures

If an existing floor failure is being investigated, the engineer needs to identify the type of failure and why it occurred. If a prediction of floor failure is conducted in a new panel or on a new lease, the engineer needs to conduct a risk analysis to predict the likelihood of failure.
In general it can be concluded that:

- Most floor failure mechanisms cause secondary failure of rock at floor level.

- The information needed to successfully predict floor failure consists of:
  - Floor geology,
  - Rock strength criteria (as discussed previously in Chapter 7),
  - The in-situ stress magnitudes and directions (section 7.3.4),
  - Mine design plans, and
  - Relevant information on factors influencing floor failure (Chapter 7).

Once the floor failure mechanism is identified, remedial actions need to be put in place to minimise the impact of floor failure on mining (explained in Chapter 7).

Appropriate remedial actions may include:

- Optimise the mining direction to minimise the influence of high stress environment,
- Minimise floor saturation especially in claystone strata by reducing dust suppression water sprays, mining uphill where possible, dewatering gate roadways and using a gas drainage program to drain water from the floor strata,
- Minimise longwall stoppages in risk areas,
• Conduct geological mapping and floor strength tests to identify problem areas, and
• Use hydraulic pull out rams mounted at the base of powered supports.

8.2 RECOMMENDATIONS FOR FURTHER RESEARCH

The new methods of determining floor failure mechanisms in underground coal mines described in this study can significantly improve current design and stability analysis of the mine floor. Even though this comprehensive study on how to identify, predict and minimise floor failure has been advanced to a new level, more research is still needed to further refine these methods.

8.2.1 Rock Strength Criteria

Floor strength criteria in sedimentary strata are still not well understood. Variation in rock type, bedding properties, existing faults and structures intercepted with mining induced fractures create a complex matrix of floor strata where strength versus direction plays an important role, and still no simple solutions exist. Further sophistication of current rock strength criteria may not be welcome by practitioners, mainly due to the added complexity with which one needs to deal. It is certainly desirable to provide engineers with comprehensive concepts and methods to understand design and analysis
procedures, but the writer believes it may be more desirable to keep rock strength criteria to manageable (simplified) levels for the practising engineer. Future predictions for floor failure mechanisms may be developed by exploiting directional strength of rock mass (anisotropy effect) into a step by step procedure, rather than always employing the ultimate strength criterion. When dealing with a weak bedding plane, one considers only its simple properties to calculate parameters such as stress, or displacements due to bedding plane failure. If the failed bedding plane causes stress concentrations within the rock mass itself, rock strength at a particular direction can then be considered separately, and the safety factor determined accordingly.

One of the most common problems is the low bearing capacity of severely fractured floor strata that typically occurs below the hydraulic supports at the longwall face, where strata movement is reactivated along pre-existing (primary) fracture surfaces. Formation of fractures in the strata is stress and mining dependent, and even though the floor is severely fractured it is possible the failed rock may be strong in one direction but weak in the other. Therefore the writer recommends a better understanding of how post-failure behaviour affects floor bearing capacity, for example, further research is needed to analytically quantify permissible floor bearing capacity after it has yielded.
8.2.2 Underground Monitoring of Floor Deformation.

Underground measurements of strata failure mechanisms provide a powerful base for new research. There has only been a limited number of measurements taken to quantify floor failure in the roadways and along the longwall face. New methods of monitoring are needed to provide a database for the various conditions that often occur underground, particularly stress measurements in the floor. It is imperative we understand how shear stress is transferred between the coal seam and floor strata, because stress concentrations can cause floor failure ahead of the coal seam movement. While measuring these stresses may be difficult to conduct, they can be taken using the new ANZI 3-dimensional stress measuring instruments that can bond successfully to the rock while under water.

8.2.3 Buckling of Mine Floor in High Lateral Stress Field

A proper understanding of numerous floor failures that occur due to buckling under high lateral stress environment, particularly the influence of pillar failure on lateral stress needs to be researched and the results quantified. The most common floor failure in gate roadways is due to concentrations of existing lateral stress (virgin stress), while other mechanism that often generates high lateral floor stress is the expansion of a failed coal pillar. In some cases, floor failure can be caused by a combination of virgin lateral
stress and pillar expansion. Even though concentrations of virgin lateral stress are well researched, pillar failure associated with weak bedding planes in the floor is still not well understood, these aspects may be studied in the future through a subsequent doctoral thesis.

Coal expansion can generate large shear stresses in the floor and cause floor failure in mine roadways. Weak bedding planes in the roof and floor greatly reduce pillar strength, where movement of failed coal at the pillar edge can buckle the adjacent roadway floor. The ability to generate shear stress at the coal-floor boundary is not well understood and needs further research through analytical and numerical methods because the same mechanism that generates confining stress within the coal pillar itself, determines pillar strength. Researchers are still trying to perfect empirical pillar strength formulae without taking the lateral bedding contacts into consideration, therefore the writer recommends further computer based numerical modelling (eg. UDEC) to examine the role of lateral bedding contacts.
REFERENCES:


APPENDIX A

Programs developed by writer:

A1 FLAC PROGRAM TO SIMULATE LONGWALL EXCAVATION

A 2-dimensional FLAC program used in this thesis simulates progressive excavation of longwall as explained in Chapter 4, Section 4.4. Note that FLAC program print is not included here due to its extensive length. This program can be supplied on CD.

A2 BASIC PROGRAM

The programs presented here were used to calculate forces transferred between the multiple blocks as described in Chapter 6, Section 6.3.

Following program uses Equation (6.9) on page 168 to calculate Q forces:

CLS
REM PROGRAM TO EVALUATE Q-FORCE ON BLOCK SIDES (for planar inclination)

OPEN "q.dat" FOR OUTPUT AS #1

CONV = ATN(1) / 45

PHIV = 15 * CONV

PHIH = 35 * CONV

W = 5: REM tonnes

PRINT "TITLE: SIDE FORCES BETWEEN BLOCKS ON PLANAR SLOPE"

PRINT "Q FORCE (tonnes)"

PRINT "------------------"

PRINT "alpha 2deg 4deg 6deg 8deg 10deg"

PRINT #1, "TITLE: SIDE FORCES BETWEEN BLOCKS ON PLANAR SLOPE"

PRINT #1, "Q FORCE (tonnes)"

PRINT #1, "------------------"

PRINT #1, "alpha 2deg 4deg 6deg 8deg 10deg"

FOR I = 1 TO 6

FOR J = 1 TO 5

A = 2 * J * CONV: REM a=inclination angle

ABCDEF = SIN(A) + TAN(PHIH) * COS(A)

QHL = I * W * COS(A)

QH2 = 1 / ABCDEF - TAN(PHIV) * COS(A) - SIN(A)
\[ Q(j) = \frac{Q_{h1}}{Q_{h2}} \]

\text{NEXT} j

\text{PRINT} "Block"; i;

\text{PRINT USING} "######.#"; Q(1); Q(2); Q(3); Q(4); Q(5)

\text{PRINT} #1, "Block"; i;

\text{PRINT} #1, USING "######.#"; Q(1); Q(2); Q(3); Q(4); Q(5)

\text{NEXT} i

\text{END}

\textbf{Following program uses Equation (6.24) on page 172 to calculate Q forces:}

\text{CLS}

\text{REM PROGRAM TO EVALUATE Q FORCE ON BLOCK SIDES (Including non-linear elevation of bedding and load from powered supports)}

\text{OPEN} "qip.dat" \text{FOR OUTPUT AS} #1

\text{PRINT} "TITLE: SIDE FORCE BETWEEN BLOCKS ON SLOPE = X^2/50"

\text{PRINT} "INCLUDING P=200 tonnes - SUPPORT LOAD ON FIRST 3 BLOCKS"

\text{PRINT} "phiv=45deg phih=25deg Block 1m*2m high"

\text{PRINT} #1, "TITLE: SIDE FORCE BETWEEN BLOCKS ON SLOPE = X^2/100to400"
PRINT #1, " INCLUDING P=133.3 tonnes (600t)- SUPPORT LOAD ON FIRST 3 BLOCKS"

PRINT #1, " phiv=35-45deg  phih=10-45deg Block 1m*2m high"

conv = ATN(1) / 45

phiv = 25 * conv

phih = 35 * conv

W = 5: REM tonnes

P = 133.3: REM tonnes

PRINT " Q FORCE (tonnes)"

PRINT " -------------"

PRINT " X^2/400  X^2/300  X^2/200  X^2/100"

PRINT " Force Incline Force Incline Force Incline Force Incline"

PRINT " Qa deg  Qb deg  Qc deg  Qd deg"

PRINT #1, " Q FORCE (tonnes)"

PRINT #1, " -------------"

PRINT #1, " X^2/400  X^2/300  X^2/200  X^2/100"

PRINT #1, " Force Incline Force Incline Force Incline Force Incline"

PRINT #1, " Qa deg  Qb deg  Qc deg  Qd deg"

FOR i = 1 TO 6

IF i > 3 THEN

P = 0!

ELSE
P = 133.3

END IF

x1 = 6 - i  REM Where x1-x2=block width [same for all 6 blocks!!!]
x2 = 7 - i

ya1 = x1 ^ 2 / 400: REM Equation for the angle increment (y=x^2/400)
ya2 = x2 ^ 2 / 400

yb1 = x1 ^ 2 / 300: REM Equation for the angle increment (y=x^2/300)
yb2 = x2 ^ 2 / 300

yc1 = x1 ^ 2 / 200: REM Equation for the angle increment (y=x^2/200)
yc2 = x2 ^ 2 / 200

yd1 = x1 ^ 2 / 100: REM Equation for the angle increment (y=x^2/100)
yd2 = x2 ^ 2 / 100

aa = ATN((ya2 - ya1) / (x2 - x1)): REM aa=Angle of Floor Inclination(in Radians)

ABCDEFa = SIN(aa) + TAN(phih) * COS(aa)

Qha1 = (W + P) * COS(aa) - Qa(i - 1) * (TAN(phiv) * COS(aa) + SIN(aa) - 1 / ABCDEFa)

Qha2 = 1 / ABCDEFa - TAN(phiv) * COS(aa) - SIN(aa)

Qa(i) = Qha1 / Qha2

ab = ATN((yb2 - yb1) / (x2 - x1)): REM ab=Angle of Floor Inclination(in Radians)

ABCDEFb = SIN(ab) + TAN(phih) * COS(ab)
\[ Q_{h1} = (W + P) \cdot \cos(ab) - Q_{b(i - 1)} \cdot (\tan(phiv) \cdot \cos(ab) + \sin(ab) - \frac{1}{ABCDEF_b}) \]

\[ Q_{h2} = \frac{1}{ABCDEF_b} - \tan(phiv) \cdot \cos(ab) - \sin(ab) \]

\[ Q_{b(i)} = \frac{Q_{h1}}{Q_{h2}} \]

\[ ac = \arctan((y_c2 - ycl) / (x_2 - x1)) \] REM ac=Angle of Floor Inclination(in Radians)

\[ ABCDEF_c = \sin(ac) + \tan(phih) \cdot \cos(ac) \]

\[ Q_{cl} = (W + P) \cdot \cos(ac) - Q_{c(i - 1)} \cdot (\tan(phiv) \cdot \cos(ac) + \sin(ac) - \frac{1}{ABCDEF_c}) \]

\[ Q_{c2} = \frac{1}{ABCDEF_c} - \tan(phiv) \cdot \cos(ac) - \sin(ac) \]

\[ Q_{c(i)} = \frac{Q_{cl}}{Q_{c2}} \]

\[ ad = \arctan((y_d2 - yd1) / (x_2 - x1)) \] REM ad=Angle of Floor Inclination(in Radians)

\[ ABCDEF_d = \sin(ad) + \tan(phih) \cdot \cos(ad) \]

\[ Q_{dl} = (W + P) \cdot \cos(ad) - Q_{d(i - 1)} \cdot (\tan(phiv) \cdot \cos(ad) + \sin(ad) - \frac{1}{ABCDEF_d}) \]

\[ Q_{d2} = \frac{1}{ABCDEF_d} - \tan(phiv) \cdot \cos(ad) - \sin(ad) \]

\[ Q_{d(i)} = \frac{Q_{dl}}{Q_{d2}} \]

PRINT "Block"; i;

PRINT USING "#######.#"; Qa(i); aa / conv; Qb(i); ab / conv; Qc(i); ac / conv; Qd(i); ad / conv

PRINT #1, "Block"; i;
PRINT #1, USING "######.#"; Qa(i); aa / conv; Qb(i); ab / conv; Qc(i); ac / conv; Qd(i); ad / conv

NEXT i

END

Following program uses Equation (6.27) and (6.28) on page 174 to calculate Q forces:

CLS

REM PROGRAM TO EVALUATE Q-FORCE ON BLOCK SIDES (for inclination= planar with no slip along vertical fractures)

OPEN "qp.dat" FOR OUTPUT AS #1

conv = ATN(1) / 45

phiv = 25 * conv

phih = 30 * conv

W = 5: REM tonnes

P = 133.3

PRINT "TITLE: SIDE FORCES BETWEEN BLOCKS ON PLANAR SLOPE no vert slip"

PRINT " Q FORCE (tonnes) P SUPPORT FORCE = 133 tonnes on each of 3 blocks"

PRINT " alpha .2deg .4deg .6deg .8deg 1deg"
PRINT #1, "TITLE: SIDE FORCES BETWEEN BLOCKS ON PLANAR SLOPE"

PRINT #1, " Q FORCE (tonnes)P SUPPORT FORCE = 133 tonnes on each of 3 blocks"

PRINT #1, "-------------------"

PRINT #1, "alpha   1deg   2deg   3deg   4deg   5deg"

FOR i = 1 TO 6: REM Block No.
FOR j = 1 TO 5: REM Slope increase (deg)
    a = .2 * j * conv: REM a=inclination angle
    IF i < 3 THEN
        Q(j) = i * (W + P) * (SIN(a) + TAN(phih) * COS(a))
    ELSE
        Q(j) = (i * W + 3 * P) * (SIN(a) + TAN(phih) * COS(a))
    END IF
    PRINT "Block"; i;
    PRINT USING "#######.#"; Q(1); Q(2); Q(3); Q(4); Q(5)
    PRINT #1, "Block"; i;
    PRINT #1, USING "#######.#"; Q(1); Q(2); Q(3); Q(4); Q(5)
NEXT i
END
Following program uses Equation (6.38) on page 176 to calculate Q forces:

CLS

REM PROGRAM TO EVALUATE Q FORCE ON BLOCK SIDES (Including non-linear inclination of bedding with no slip along vertical fractures)

OPEN "qip.dat" FOR OUTPUT AS #1

PRINT "TITLE: SIDE FORCE BETWEEN BLOCKS ON SLOPE = X^2/400"
PRINT "INCLUDING P=600 tonnes - SUPPORT LOAD ON FIRST 3 BLOCKS"

PRINT "phiv=35-45deg  phih=10-45deg Block 1m*2m high"

PRINT #1, "TITLE: SIDE FORCE BETWEEN BLOCKS ON SLOPE = X^2/100to400"
PRINT #1, "INCLUDING P=133.3 tonnes (600t)- SUPPORT LOAD ON FIRST 3 BLOCKS"

PRINT #1, "phiv=35-45deg  phih=10-45deg Block 1m*2m high"

conv = ATN(1) / 45

phiv = 25 * conv

phih = 10 * conv
W = 5: REM tonnes

P = 133.33: REM tonnes

PRINT " Q FORCE (tonnes)"

PRINT " ------------------"

PRINT " X^2/100  X^2/50  X^2/25  X^2/10"

PRINT " Force Incline Force Incline Force Incline Force Incline"

PRINT " Qa deg  Qb deg  Qc deg  Qd deg"

PRINT #1, " Q FORCE (tonnes)"

PRINT #1, " ------------------"

PRINT #1, " X^2/400  X^2/300  X^2/200  X^2/100"

PRINT #1, " Force Incline Force Incline Force Incline Force Incline"

PRINT #1, " Qa deg  Qb deg  Qc deg  Qd deg"

FOR i = 1 TO 6

IF i > 3 THEN

    P = 0!

ELSE

    P = 133.333

END IF

x1 = 6 - i: REM Where x1-x2=block width [same for all 6 blocks!!!]

x2 = 7 - i

ya1 = x1 ^ 2 / 100: REM Equation for the angle increment (y=x^2/400)

ya2 = x2 ^ 2 / 100
yb1 = x1 ^ 2 / 50: REM Equation for the angle increment (y=x^2/300)

yb2 = x2 ^ 2 / 50

yc1 = x1 ^ 2 / 25: REM Equation for the angle increment (y=x^2/200)

yc2 = x2 ^ 2 / 25

yd1 = x1 ^ 2 / 10: REM Equation for the angle increment (y=x^2/100)

yd2 = x2 ^ 2 / 10

aaold = aa

aa = ATN((ya2 - ya1) / (x2 - x1)): REM aa=Angle of Floor Inclination(in Radians)

Qa(i) = Qa(i - 1) * (COS(aaold - aa) + SIN(aaold - aa) * TAN(phih)) + (W + P) * (TAN(phih) * COS(aa) + SIN(aa))

abold = ab

ab = ATN((yb2 - yb1) / (x2 - x1)): REM ab=Angle of Floor Inclination(in Radians)

Qb(i) = Qb(i - 1) * (COS(abold - ab) + SIN(abold - ab) * TAN(phih)) + (W + P) * (TAN(phih) * COS(ab) + SIN(ab))

acold = ac

ac = ATN((yc2 - yc1) / (x2 - x1)): REM ac=Angle of Floor Inclination(in Radians)

Qc(i) = Qc(i - 1) * (COS(acold - ac) + SIN(acold - ac) * TAN(phih)) + (W + P) * (TAN(phih) * COS(ac) + SIN(ac))

adold = ad

ad = ATN((yd2 - yd1) / (x2 - x1)): REM ad=Angle of Floor Inclination(in Radians)
Qd(i) = Qd(i - 1) * (COS(adold - ad) + SIN(adold - ad) * TAN(phih)) + (W + P) * (TAN(phih) * COS(ad) + SIN(ad))

PRINT "Block"; i;
PRINT USING "#######.#"; Qa(i); aa / conv; Qb(i); ab / conv; Qc(i); ac / conv; Qd(i); ad / conv
PRINT #1, "Block", i;
PRINT #1, USING "#####.#"; Qa(i); aa / conv; Qb(i); ab / conv; Qc(i); ac / conv; Qd(i); ad / conv
NEXT i
END

A3 UDEC ROGRAM

This 2-dimensional program simulates a multiple block movement during floor uplift at the longwall face as explained in Chapter 6, Section 6.5. Various dimensions and floor velocities can be assigned to the blocks to study their movement. The print is included below:

;;TITLE STONE BLOCKS
;set log on
round 0.01
block (-.5,-.5) (-.5,2.0) (7,2.0) (7,-.5)

split (-.5,0) (7,0)

;; form blocks

split (0,0) (0,2) ;; forming 1x2m blocks
split (1,0) (1,2)
split (2,0) (2,2)
split (3,0) (3,2)
split (4,0) (4,2)
split (5,0) (5,2)
split (6,0) (6,2)

gen quad .3

;; base

change -.5 7 -.5 0 mat=1 ;; base

prop mat=1 dens=2500 b=5e9 s=2e9 coh=20e9 tens=20e9 fric=35

change -.5 7 0 2.5 mat=2 ;; blocks

prop mat=2 dens=2500 b=5e9 s=2e9 coh=20e9 tens=20e9 fric=35

change -.5 7 -.1 .1 jmat=3 jcons 2 ;; base interface joint

prop jmat=3 jkn=20e11 jks=20e11 jfric=10 jcoh=0

change -.5 7 .1 2.5 jmat=4 jcons 2 ;; blocks interface joints

prop jmat=4 jkn=3.2e9 jks=3.2e9 jfric=35 jcoh=0

;; FIX

boundary -.5,.1 0.1,2 yvel=0 ; fix
boundary -.5,-.1 .1,2 xvel=0 ; fix
boundary -.5,-.1 -.5,-.1 yvel=0 ; fix
boundary -.5,-.1 -.5,-.1 xvel=0 ; fix
;;; Assign velocity to bottom of the base in small increases
boundary 0.1 .2 -.6 -.3 yvel=1e-3 ;;; Note: linear displacement of floor
boundary 0.22 .3 -.6 -.3 yvel=2e-3
boundary 0.32 .4 -.6 -.3 yvel=3e-3
boundary 0.42 .5 -.6 -.3 yvel=4e-3
boundary 0.52 .6 -.6 -.3 yvel=5e-3
boundary 0.62 .7 -.6 -.3 yvel=6e-3
boundary 0.72 .8 -.6 -.3 yvel=7e-3
boundary 0.82 .9 -.6 -.3 yvel=8e-3
boundary 0.92 1.0 -.6 -.3 yvel=9e-3
boundary 1.02 1.1 -.6 -.3 yvel=10e-3
boundary 1.12 1.2 -.6 -.3 yvel=11e-3
boundary 1.22 1.3 -.6 -.3 yvel=12e-3
boundary 1.32 1.4 -.6 -.3 yvel=13e-3
boundary 1.42 1.5 -.6 -.3 yvel=14e-3
boundary 1.52 1.6 -.6 -.3 yvel=15e-3
boundary 1.62 1.7 -.6 -.3 yvel=16e-3
boundary 1.72 1.8 -.6 -.3 yvel=17e-3
boundary 1.82 1.9 -.6 -.3 yvel=18e-3
boundary 1.92 2.0 -.6 -.3 yvel=19e-3
boundary 2.02 2.1 -.6 -.3 yvel=20e-3
boundary 4.52 4.6 - .6 - .3 yvel=45e-3
boundary 4.62 4.7 - .6 - .3 yvel=46e-3
boundary 4.72 4.8 - .6 - .3 yvel=47e-3
boundary 4.82 4.9 - .6 - .3 yvel=48e-3
boundary 4.92 5.0 - .6 - .3 yvel=49e-3
boundary 5.02 5.1 - .6 - .3 yvel=50e-3
boundary 5.12 5.2 - .6 - .3 yvel=51e-3
boundary 5.22 5.3 - .6 - .3 yvel=52e-3
boundary 5.32 5.4 - .6 - .3 yvel=53e-3
boundary 5.42 5.5 - .6 - .3 yvel=54e-3
boundary 5.52 5.6 - .6 - .3 yvel=55e-3
boundary 5.62 5.7 - .6 - .3 yvel=56e-3
boundary 5.72 5.8 - .6 - .3 yvel=57e-3
boundary 5.82 5.9 - .6 - .3 yvel=58e-3
boundary 5.92 6.0 - .6 - .3 yvel=59e-3
boundary 6.02 6.1 - .6 - .3 yvel=60e-3
boundary 6.12 6.2 - .6 - .3 yvel=61e-3
boundary 6.22 6.3 - .6 - .3 yvel=62e-3
boundary 6.32 6.4 - .6 - .3 yvel=63e-3
boundary 6.42 6.5 - .6 - .3 yvel=64e-3
boundary 6.52 6.6 - .6 - .3 yvel=65e-3
boundary 6.62 6.7 - .6 - .3 yvel=66e-3
boundary 6.72 6.8 - .6 - .3 yvel=67e-3
boundary 6.82 6.9 - .6 - .3 yvel=68e-3
boundary 6.92 7.0 -.6 -.3 yvel=69e-3

;; P-Load

;;-------

;boundary 3. 4. 1.9 2.1 yload=-1.333e6
;boundary 4. 5. 1.9 2.1 yload=-1.333e6
;boundary 5. 6. 1.9 2.1 yload=-1.333e6
boundary 3. 6. 1.9 2.1 str 0 0 -1.333e6

gravity 0 -9.8

damp auto

round 0.01

delete 6.2 7.1 2

cycle 119000

save bl.sav

set log off