

**NUMERICAL ANALYSES OF THE EFFECTIVENESS OF SECONDARY
TAILGATE SUPPORT SYSTEMS : A STABILITY APPROACH**

by

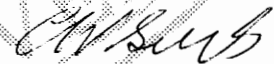
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Mining and Minerals Engineering

ABSTRACT

A numerical model was developed to analyze the effectiveness of active and passive secondary support systems on the stability of a retreating longwall tailgate opening. The range of loading conditions that any tailgate can be subjected to was identified to form the basis of numerical modeling. The tailgate entry was considered as a part of roof-pillar-floor system, whose behavior is controlled largely by the structural integrity of each member. Numerical modeling was then conducted on tailgate openings to determine roof, floor, and rib responses, including failure modes to a variety of loading conditions for wood cribs and roof trusses employed as active and passive means of secondary support. Trends were developed from numerical modeling to determine optimum load capacity and points of application for reducing the potential for entry failure.

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CHAPTER 1.

Introduction

Throughout the history of longwall mining, gateroad stability has always been a key factor in the continuous operation of any longwall mine. Any downtime caused by the closure of the gateroads will significantly effect the productivity of the whole panel. The tailgate area is most affected by abutment loads that are transferred from the sides and the front of the panel and the rather abrupt changes in the loading regime as the face passes.

Although secondary tailgate support can be a high cost activity in longwall mining, the benefits of such support are not always realized. Traditional methods typically involve installing large quantities of timber cribs while the opening is still a headgate. When the depth of cover is relatively shallow and the tailgate entries are competently designed in accordance with local conditions, the supports will experience little, if any, load and, therefore, represent an unwarranted expenditure. When pillar failure does occur due to depth or under-design, the cribs may be distorted or destroyed to the point where tailgate

closure is imminent, and continued operation of the longwall panel is in jeopardy. In order to minimize this possibility, support must be custom designed to meet projected loading conditions.

Loss of stability does not only result in interruption of mining operations but can also cause serious injury and loss of life. The Mine Safety and Health Administration indicate that during the five year period from 1984 to 1988 a total of 106 coal miners had lost their lives due to roof falls and 4,135 miners were injured during the same time span (Farley, 1989). Approximately half of the victims of these falls have been found to be in an area where the roof was not supported. Any improvement made in the area of ascertaining the stability of an underground opening has the potential to reduce the number of lives lost.

The primary objectives of this research are to devise a numerical model which would simulate, with a reasonable degree of accuracy, the loading conditions encountered in the tailgate entries. This model would then be used to analyze the effectiveness, in terms of stability, of the various support alternatives that may be used as secondary support for the entries.

CHAPTER 2.

Review of Literature

2.1 Introduction

The history of longwall mining in the United States dates back to 1875 when it was first introduced by emigrating miners from Wales (Laird , 1973). Currently , longwall mining accounts for nearly one third of all underground coal production and continues to set new productivity records. This is made possible by the high degree of mechanization and automation offered by the method itself. Even though longwall mining has evolved into the safest, most productive and most conservative of underground mining systems (Barczak, 1992) , it is not completely problem free. Maintaining stable openings in the head and the tailgates are crucial for the proper operation of the panel.

The two most widely used methods of operating a longwall panel are the (Trent and Harrison, 1982):

1. *Advance longwall mining method*

In the advance longwall mining method gateroads are driven immediately ahead of the longwall face simultaneously as the panel is producing coal. The major disadvantage of this method is that for some instances, the face can catch up with the headings, and in this respect the production is contingent on the development work. The major advantage is the time required for production is much sooner than retreating methods.

2. Retreat longwall mining method

Retreat longwall mining method is the most widely used longwall mining method for US coal mines. In this method headings are driven and completed before any production begins. Figure 2.1.1 illustrates a typical US retreat longwall panel layout.

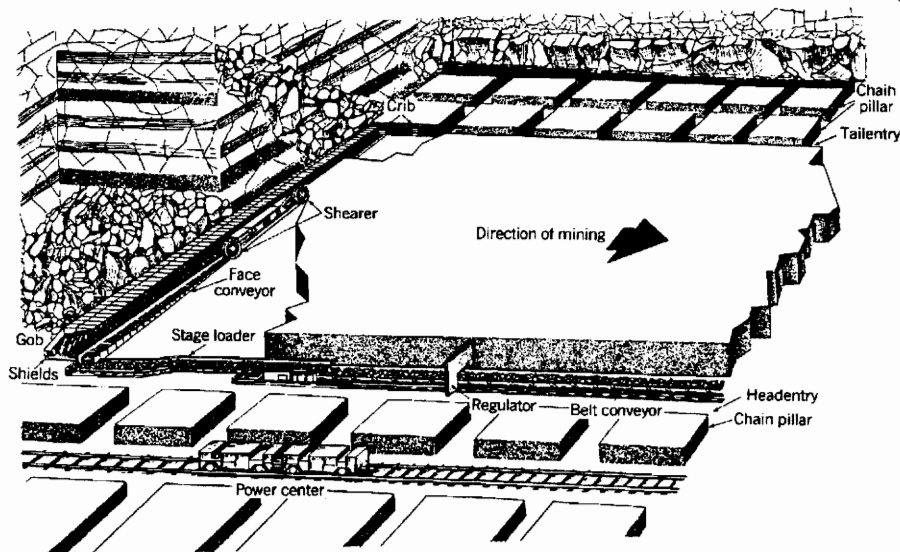


Figure 2.1.1 Typical longwall retreating panel layout (Peng, 1986)

In the retreating mining method the tailgate entry generally experiences higher loads than the head entry from the second panel onwards because it is

located on the side nearest the mined out panels. Figure 2.2.2 illustrates the typical critical pressure abutment zone for a tailgate entry. The tailgate entry is primarily used as a return air passage, therefore a complete clearance is not an absolute necessity (Peng, 1986).

Wood cribs are by far the most commonly used means of secondary support in this critical region. A recent survey conducted among 76 different longwall mines and 86 operational longwall panels revealed that wood cribs have been used in at least 86% of the mines (Peng and Biswas, 1994). At least 94% of the mines surveyed used 4 point cribs and 60% of all the mines surveyed used a two row in parallel arrangement of cribs within the tailgate.

Other material types, besides wood have also been used for supporting the tailgate (Didcoct, 1982). These are usually made of concrete or with some form of reinforced concrete to better their bending and tensile strength characteristics. Other alternative materials are being investigated which are claimed to offer better load handling characteristics and improved safety, especially against fire (Smart et. al., 1992).

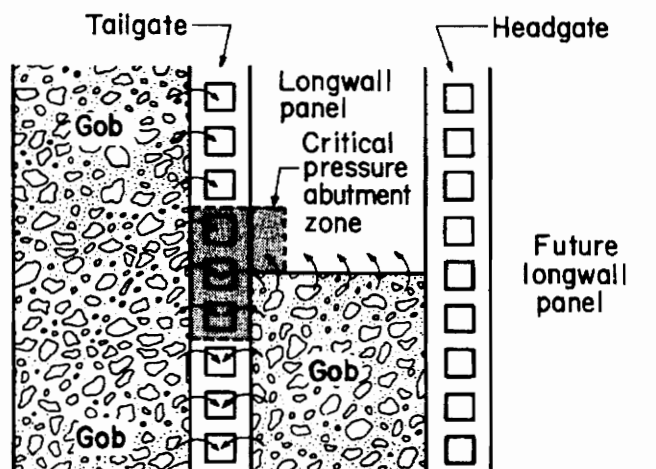


Figure 2.1.2 Critical abutment pressure zone (Kripakov et al., 1990)

2.2 Roof Loading

The tailgate experiences significant strata movements and stress redistributions throughout the life span of the longwall panel. The effect that this changing stress field has on the stability of the tailgate entry depends primarily on:

1. *The magnitude of longwall induced loading*
2. *Depth*
3. *Roof geology*
4. *Mining geometry and spatial dimensions.*

In reality, the problem is further complicated by the fact that roof loading and hence the behavior of the tailgate is also controlled by both pillar and floor response which act as an integrated system (Hosca and Haycocks, 1994). In order to have a thorough understanding of tailgate response the behavior of the pillar and the floor must also be taken into account.

The complete state of stress in any given underground mining situation can only be defined if both the vertical and horizontal stress components are known with an acceptable degree of accuracy.

2.2.1 Vertical Stresses

Vertical stresses encountered underground can be classified in two different ways in a chronological fashion as :

1. *Virgin stresses*
2. *Mining induced stresses*

Virgin stresses are the stresses present due to the weight of the overlying strata and can be estimated by the following formula :

$$\sigma_v = \gamma \cdot h \quad (2.1)$$

where:

σ_v = estimated vertical stress

γ = unit weight of the overlying material

h = depth

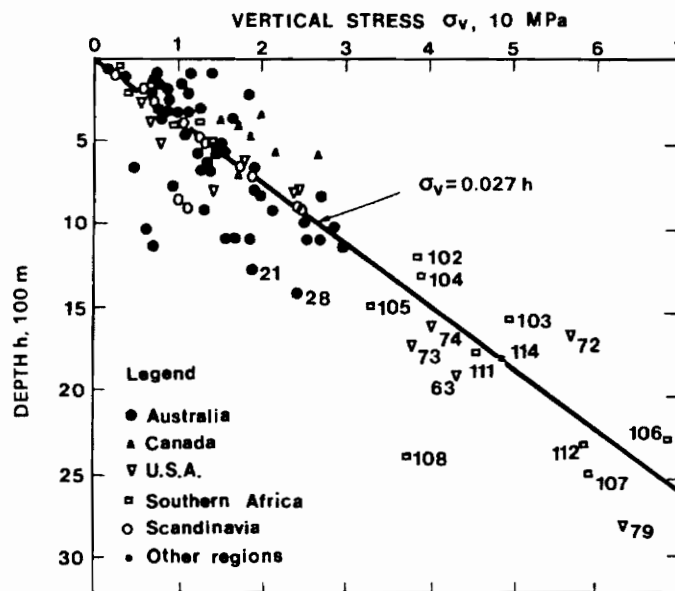


Figure 2.2.1 Measured vertical stresses vs. depth (Brown and Hoek, 1978)

With the exception of Gale et al. (1987) worldwide stress measurements have shown that in-situ vertical stress is approximately 0.027 Mpa per meter of depth (Brown and Hoek, 1978) as illustrated by Figure 2.2.1.

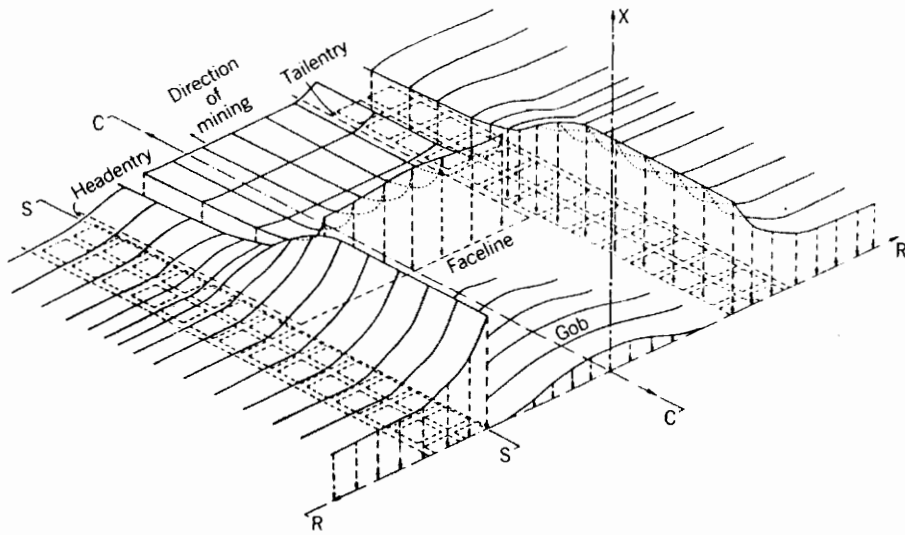


Figure 2.2.2 Idealized state of vertical stress around a longwall panel (Peng and Chiang, 1984)

Excavations necessary to conduct mining operations disturb this virgin state of stress and cause additional loads to act on the tailgate. Part of the load from the gob and the previously mined out panel, as mining proceeds are transferred to the gate entries. This load distribution is not uniform across the tailgate length and is in a continuous state of change as the longwall face is in operation. Figure 2.2.2 illustrates the idealized state of vertical stress distribution for a typical longwall panel.

The total load immediately adjacent to the tailgate can be estimated by :

$$L_T = L_D + L_S + L_F \quad (2.2)$$

where :

L_D = development load

L_S = side abutment load

L_F = front abutment load

Development load L_D can be estimated using the tributary area concept to be :

$$L_D = h \cdot \gamma \cdot [\sum (w) + (n-1) \cdot w_e] \quad (2.3)$$

where :

h = depth of cover

w = width of individual pillars

w_e = entry width

n = number of entries

The magnitude of the total abutment loads acting on the tailgate can be estimated by summing the front and side abutment loads. It has been found that the magnitude of the total abutment is load related to the location of the face and the position of the mined out panels (Mark, 1990).

Empirical methods have been proposed to estimate the side abutment loads. Based on the assumption that the vertical stress within the gob increases linearly with distance, from zero at the rib to the magnitude of the virgin stress at

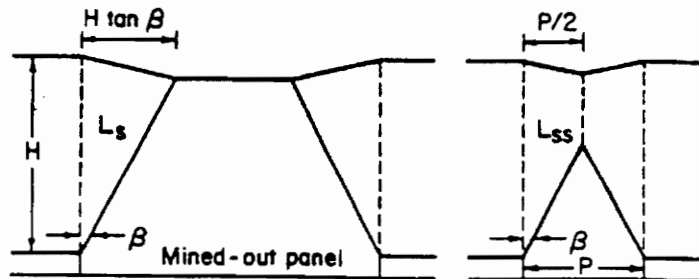


Figure 2.2.3 Conceptualization of shear angle (King and Whittaker, 1971)

some point within the gob, it has been estimated that the distance required for the stress in the gob to be reduced to the virgin stress value is typically one third of the depth of cover (Wilson, 1982). Another way of estimating side abutment loads is based on the concept of a shear angle that determines the loading on the pillars (King and Whittaker, 1971). Figure 2.2.3 illustrates this concept of forming a wedge of strata defined by the shear angle β . Two equations are defined to estimate the value of the side abutment load per foot of gate entry, one for critical and super-critical panels and the other for sub-critical panels.

$$L_s = h^2 \cdot \tan \beta \cdot \left(\frac{\gamma}{2} \right) \quad (2.4)$$

$$L_{ss} = \left[\frac{h \cdot p}{2} - \frac{h^2}{8 \cdot \tan \beta} \right] \gamma \quad (2.5)$$

where :

β = angle of shear

p = distance between mined out panels

Three dimensional numerical modeling has been used to determine the front abutment load (Hsuing and Peng, 1985; Kripakov et al., 1988). A much simpler method of estimating the front abutment load is given by (Mark, 1990) :

$$L_F = F \cdot L_s \quad (2.6)$$

where

F = front abutment factor

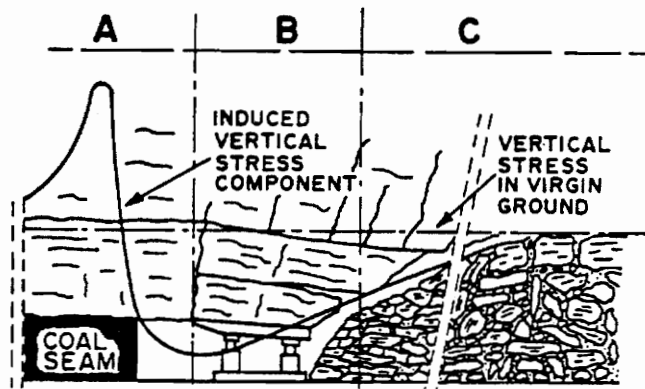


Figure 2.2.4 Variation of front abutment pressure (Whittaker, 1982)

The maximum value of the front abutment factor cannot exceed one, and is determined by field measurements. Figure 2.2.4 illustrates an idealized distribution of front abutment pressure.

The abutment zone extends into the surrounding strata and its magnitude gradually decreases to the original virgin state of stress value. This area which is encompassed by the abutment stresses is named as the zone of influence. The width of this zone D , defined by the distance from the edge of the panel has been determined from analysis of field measurements to be (Peng and Chiang, 1984) :

$$D = 9.3 \cdot \sqrt{h} \quad (2.7)$$

The value of the stress in the abutment zone has been found to decay according to the inverse square of the distance from the panel edge as illustrated in Figure 2.2.5 (Mark, 1990). However, it should be noted that the stress profile depicted in Figure 2.2.5 is only applicable once the longwall face has long past by, as in bleeder entries.

KEY	
σ_a	Abutment stress distribution function
x	Distance from the edge of the longwall panel
D	Extent of the side abutment influence zone
W_A, W_B	Pillar widths
L_S	Total side abutment load
L_A	Abutment load on pillar "A"
L_B	Abutment load on pillar "B"
L_{BP}	Abutment load on barrier pillar

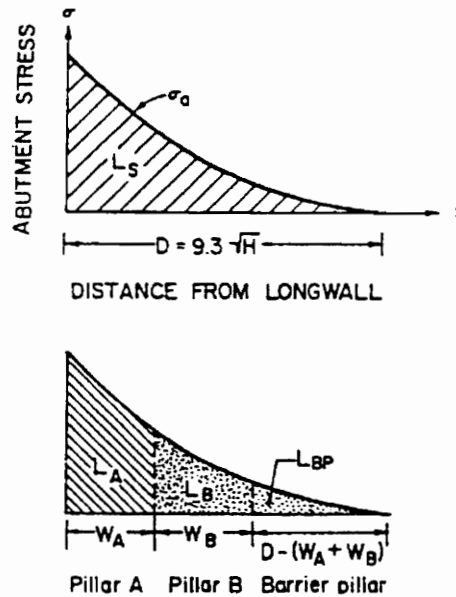


Figure 2.2.5 Side abutment stress decay function (Mark, 1990)

In-situ pressure measurements made in two western U. S. coal mines have indicated that the front abutment zone extends to approximately 0.25 times the depth ahead of mining and significant increases begin at 0.1 times the depth and peak stresses occur within 0.01 times the depth into the face, increasing in magnitude with depth (Haramy and Kneisley, 1989).

2.2.2 Horizontal Stresses

Horizontal stresses are developed at depth, due to the tendency of lateral expansion of strata caused by the weight of the overlying material and can significantly effect mining operations. One of the principal effects of excessive

horizontal stress on the stability of an underground opening is the development of cutter roofs (Su and Peng, 1987; Agapito et al., 1980). Assuming plane strain conditions and elastic behavior of rocks, the induced horizontal stress due to the vertically applied loads can be calculated from :

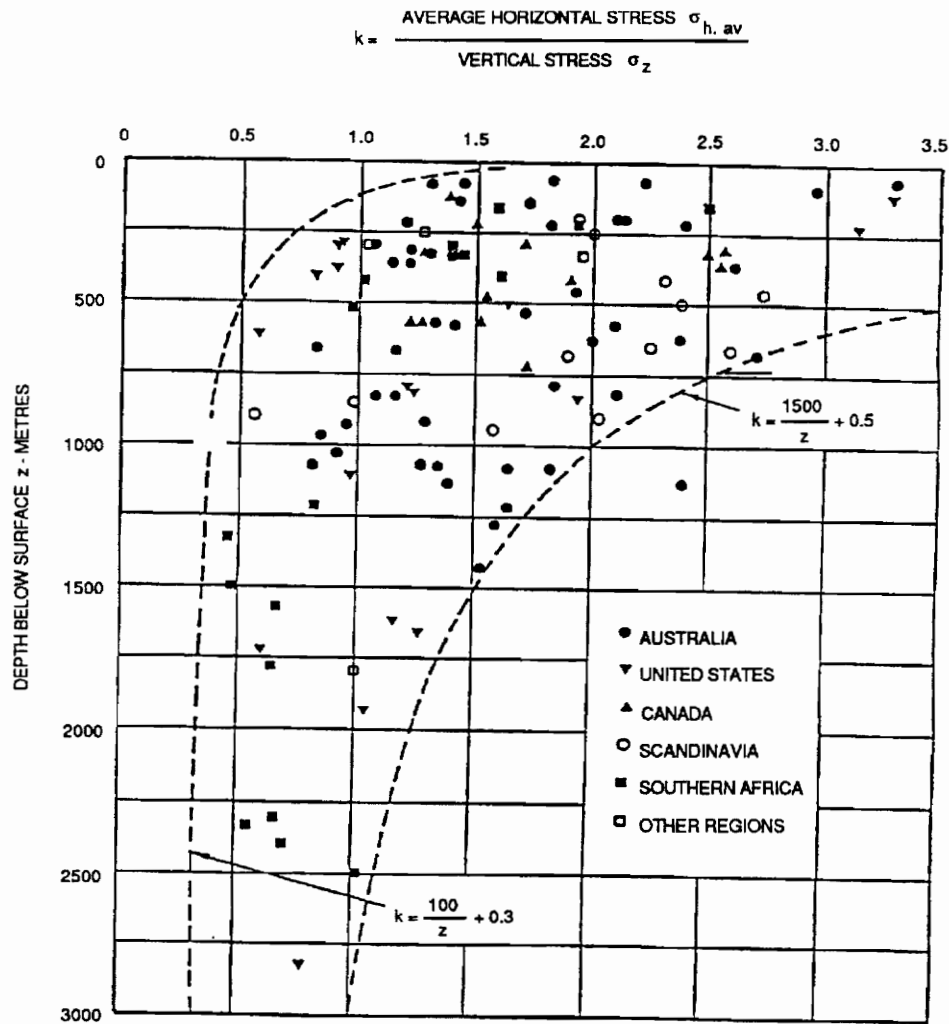


Figure 2.2.6 Measured horizontal stresses vs. depth (Brown and Hoek, 1978)

$$\sigma_h = \frac{\nu}{(1-\nu)} \cdot \sigma_v \quad (2.8)$$

where :

σ_h = horizontal stress

ν = Poisson's ratio

σ_v = vertical stress

However, field measurements usually do not comply with this formula. It has been determined that the ratio of the average of the two horizontal stresses to the vertical stress varies widely, from 0.5 to greater than 3.5 near the surface and from 0.3 to 1.0 at depths below 10,000 feet (Brown and Hoek, 1978). The measured horizontal stresses in many parts of the world are also much higher than predicted by Equation (2.8) (Hoek and Brown, 1982). Figure 2.2.6

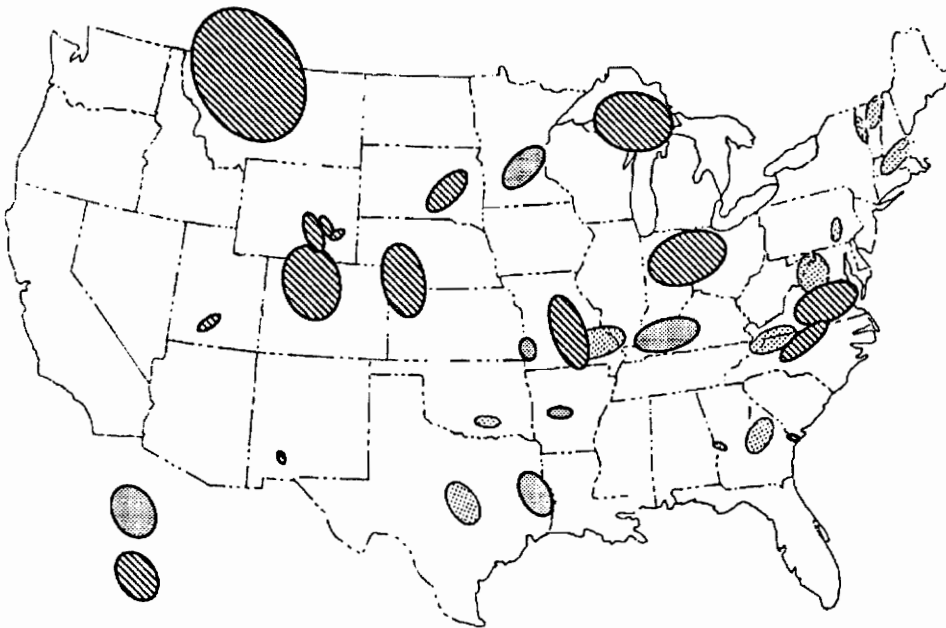


Figure 2.2.7 Orientation of horizontal stress for US (Aggson, 1978)

illustrates the ratio of horizontal stress to vertical stress for various regions of the world.

The orientation of the maximum horizontal stress for the United States is shown in Figure 2.2.7. The magnitude and orientation of the major and minor axes of each ellipse shown in the figure represent the excess maximum and minimum horizontal stresses at given locality (Aggson, 1978) Observation of Figure 2.2.7 reveals the following :

1. *The horizontal stress field is more or less parallel to the Appalachian Mountain chain in the eastern coast of the United States*
2. *Near the Great Lakes region the maximum compressive stress is*

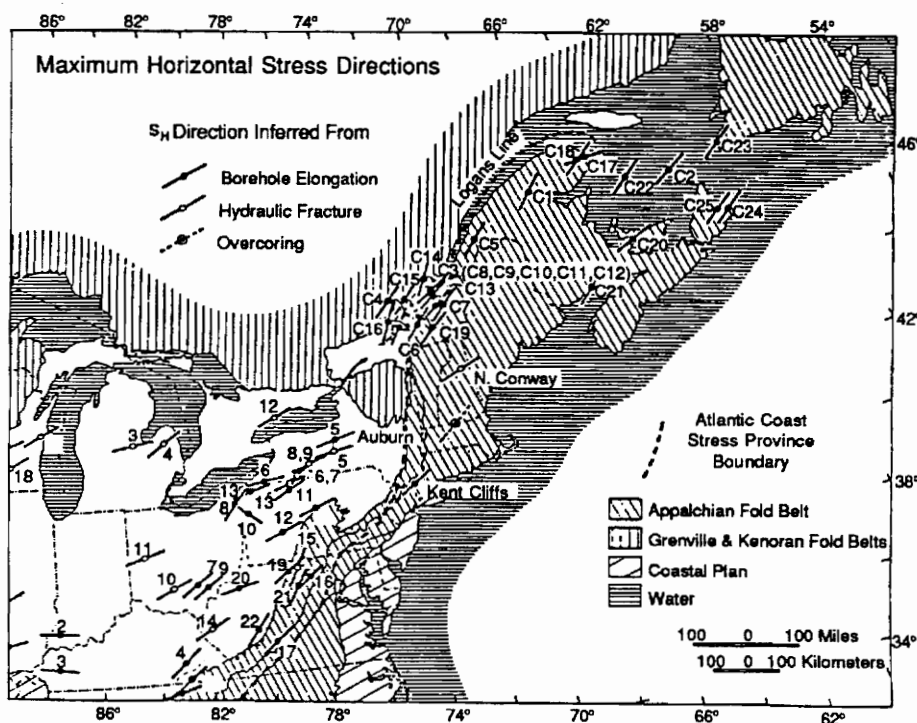


Figure 2.2.8 Direction of horizontal stress for northeast US (Plumb and Cox, 1987)

tangential to the Michigan basin

3. *In the western parts of the US the maximum stress is parallel to the Rock Mountains*

The orientation of the horizontal stress field in northeastern United States has been determined from various sources such as hydraulic fracturing, borehole breakouts or elongations and centerline fractures in oriented core (Plumb and Cox, 1987). The majority of the data were taken at a depth of 457 to 1200 meters. The analysis of the data revealed a persistent contemporary E- NE stress field, with some rotation towards E-W in the Illinois basin. The orientation map of maximum horizontal stress in deep boreholes is given in Figure 2.2.8.

The horizontal stress measurements compiled from more than 25 mines throughout the eastern United States confirms the presence of an E-NE stress field (Mark, 1991). Figure 2.2.9 shows that 67% of the measurements from mines in the Appalachian and Warrior coal basins found the orientation of

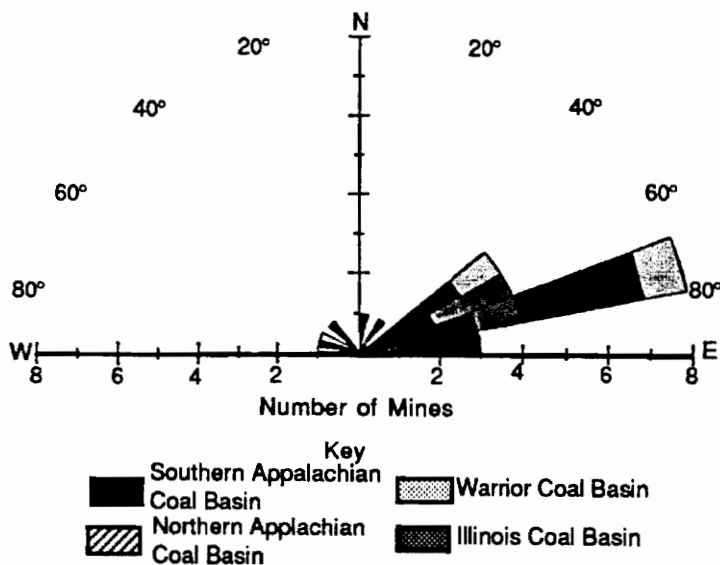


Figure 2.2.9 Distribution of horizontal stress directions (Mark, 1991)

maximum horizontal stress between N80E and N50E. In the Illinois basin, however, 75% of the measurements indicated that the stress field is rotated towards E-W by approximately 15 degrees. The magnitude of the horizontal stress exceeded the vertical stress in more than 90% of the observations, most often by a factor of two or more. It has also been noted that existing structural features did not appear to have a significant effect on the direction of the horizontal stress (Ingram and Molinda, 1988). Interestingly enough, surface topographic features such as stream valleys have been shown to reorient local stress fields (Molinda et al., 1992).

Horizontal stresses are known to adversely effect longwall operations all around the world. Since the late 1970s the lead on developing technology for detecting and controlling horizontal stresses have been taken by Australian and British ground control researchers (Matthews et al., 1992; Siddall, 1989; Gale et al., 1987). Many control strategies have been developed to control the adverse effects of horizontal stresses including stress mapping, entry orientation, stress-relief entries and high strength roof bolts. Incorporating yield pillars into the design of the longwall panel also have been reported to be quite successful in reducing the number of ground control problems caused by high horizontal stresses (DeMarco et al., 1993).

A good example of avoiding ground control problems caused by high horizontal stresses by the use of orienting the entries according to the stressfield can be seen in the Pittsburgh coal seam, which is the most extensively longwalled coalbed in the US (Mark, 1991). Another interesting characteristic of the Pittsburgh coalbed is that it appears to have the highest horizontal stress. During the past 20 years a total of 157 longwall panels have been extracted from the coalbed. Of these panels 58% were oriented between E-W and N70W, and another 29% had been oriented between N70W and N60W. No major ground

control problems were reported from these panels. However, only two of the 157 panels had been oriented N-S, both experiencing conditions so poor that both were abandoned even before they were completed.

Driving a stress relief roadway can also be used to control ground problems caused by high horizontal stresses. A good example is the Kitt Mine that has suffered from cutter roof throughout its life (Mark, 1991). The last longwall mined at Kitt was developed with an arched center entry driven 4.5 meters high. This entry was driven approximately 18 meters ahead of the outside headings and was supported by yieldable steel arches that would yield an additional 3 meters. A series of stress measurements have determined that the arched entry created a zone of stress relief that extended at least 24 meters (Aggson and Mouyard, 1988). Later attempts at reproducing the results through numerical modeling have concluded that slippage along horizontal bedding planes was necessary to explain the extent of the stress relief (Ahola and Kripakov, 1987).

While longwall panel design may be the most economical method of controlling or minimizing the effects of horizontal stress in the long run, few operating mines can afford to reorient panels (Krupa and Long, 1994). This being the case, remedial measures are leaning towards the use of secondary supports to alleviate the problems associated with high horizontal stresses.

2.3 Roof Response

The drivage of gateroads for a new panel in a longwall mining operation results in a redistribution of loads between the roof, pillar and floor. Each one of these elements behave differently in order to adjust to the new state of stress that results from the opening. Depending on various other factors as well, there are several distinct physical alterations that each member of the system will go through.

A stable roof is essential before any coal can be mined. Considerable research has been conducted on predicting roof conditions before mining starts. Taking into consideration the fact that 35% of direct and indirect mining costs are comprised of roof control expenses (Campbell et al, 1975), it is evident that successful prediction of roof conditions before the actual mining process takes place is well worthwhile. Geologic mapping , drill core analysis, lineament analysis and even aerial photography have been used to predict unstable roof conditions in advance of mining (Ledvina, 1986; Hylbert, 1978; Ealy et al., 1979; Chugh and Silverman, 1982). Integration of stress analysis before any excavation is made has great potential to reduce the likelihood of having to face an unstable roof when mining operations are carried out. In order to facilitate a seamless integration links between mine planning software and stress analysis codes have been developed (Coulthard et al. 1991).

Three distinct phases in which significant alterations in the loading of the tailgate can be identified chronologically as mining operations are carried out (Matsui et al., 1992). These are :

1. *The drivage phase*
2. *First face working*

3. Second face working

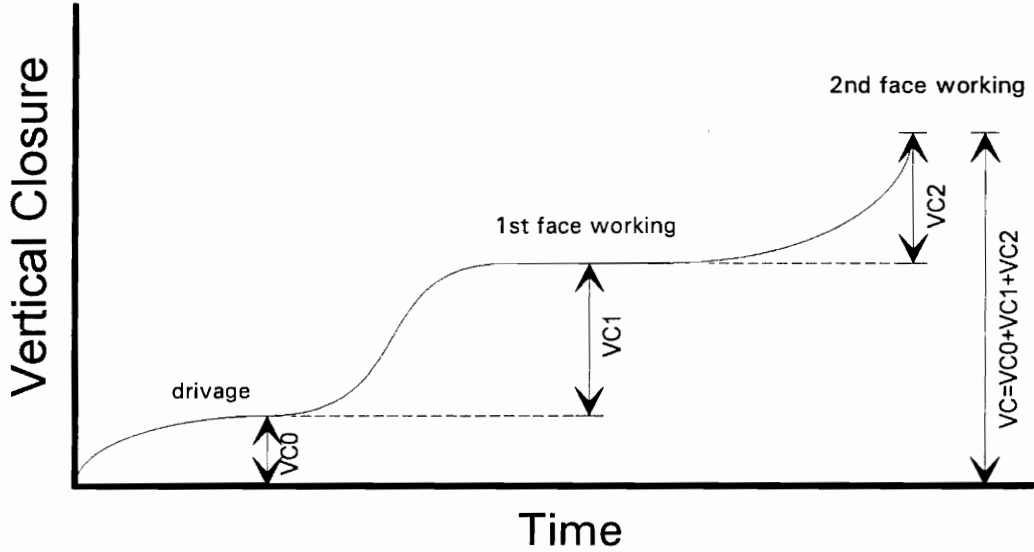


Figure 2.3.1 Vertical closure vs. time for a typical gateroad (Mitsui et al., 1992)

The effect of these three phases on the roof is predominantly reflected by the amount of vertical closure measured. Figure 2.3.1 illustrates the amount of vertical closure experienced in a typical tailgate during mining. The graph shows a rapid increase in the rate of vertical closure during the initial drivage stage of the roadway, reaching a total value of VC_0 , followed by a second increased rate of closure as the first retreat longwall face is in operation, amounting to VC_1 . A final increase in the rate of closure, VC_2 occurs when the mining of the second face is initiated. After the passage of the second longwall face the gateroad is no more functional. The total vertical closure can then be expressed as :

$$VC = VC_0 + VC_1 + VC_2 \quad (2.9)$$

The effect of floor heave, which can be a major contributor to the rate of closure under relatively weak floor strata and regions of high stress should also

be taken into consideration (Matsui and Shimada, 1994).

Continuous monitoring of the roof have indicated that the measured roof-floor convergence goes through three phases: an initial phase, a rapid increase phase and a stabilization phase whenever existing stress conditions are altered in one way or the other (Yu et al., 1993, Pasamehmetoglu et al., 1992).

Measured in-situ deformation values have been successfully reproduced with numerical analysis methods assuming elastic behavior of rockmass (Vervoort et al., 1992). Even though most widely used, extensometers are not the only measuring instruments for monitoring changes in the roof. Various other instruments and techniques are available for collection of data (Bauer, 1985). Maleki et al. (1993) has used elastic wave propagation in monitoring changes in roof stability. However, attempts made to reproduce the seismic characteristics of the rockmass using numerical methods have not been successful.

2.3.1 Roof Falls

Statistics published yearly by the US Department of Labor's Mine Safety and Health Administration (MSHA) (1989) indicate that between the years 1985 and 1989 roof falls in underground coal mines comprised 88% of all fatalities involving ground control problems and that these fatalities accounted for approximately 40% of all fatalities in underground coal mines.

Moebs and Stateham (1986) have introduced a scheme for categorizing roof falls in mines based on causative factors. These are:

1. *Stress related effects* which consist of failure due to in-situ stresses and induced stresses
2. *Geologic effects* ,which are divided into five subcategories as : low rock strength, moisture sensitivity, bedding plane spacing, minor structures and major

structures.

Various attempts have been made to characterize and classify roof falls

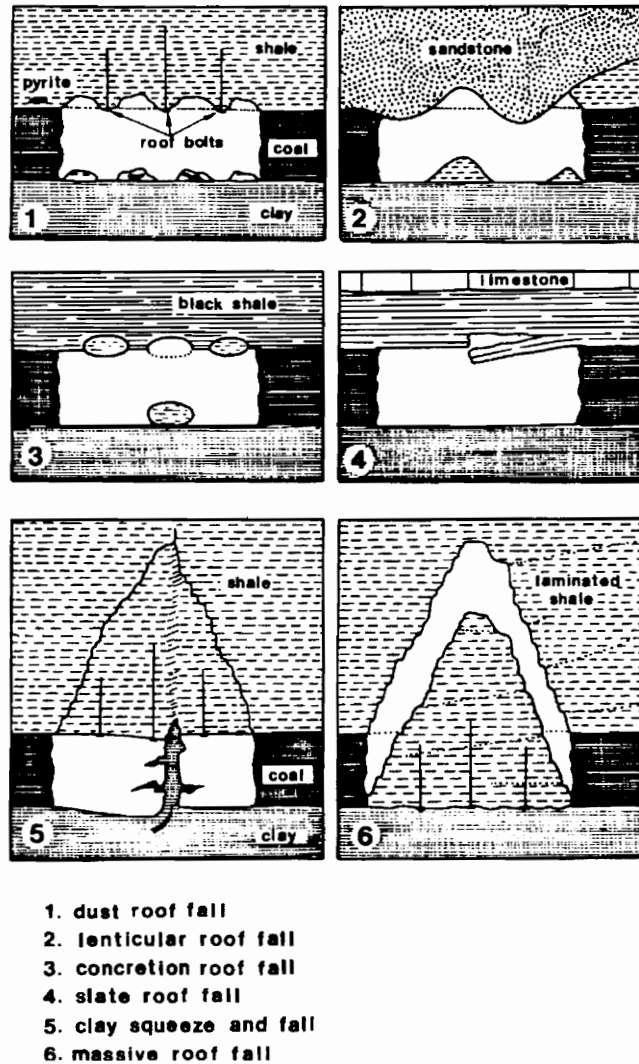


Figure 2.3.2 Different types of roof fall (Wier, 1970)

with hopes of prevention. Wier (1970) has proposed six different types of roof fall, controlled by different combinations of geological effects as illustrated in Figure 2.3.2. These are :

1. *Dust roof fall* , the thin (less than 1 ft.) and soft shales crumble into dust and fall out between roof bolts

2. *Lenticular roof fall* , the soft shale roof between two sandstone rolls come down after the removal of coal

3. *Concretion roof fall* , ironstone concretions fall by gravity

4. *Slate roof fall* , the fissionable black shale breaks in large slabs along a bedding plane or parting

5. *Clay squeeze and fall* , the clay floor intrudes into the coal and/or the roof which weakens the roof

6. *Massive roof fall* , extends up to 20-30 feet upwards

Patrick and Aughenbaugh, (1979) have proposed a roof fall classification scheme based on the size and geometry of the void created after the fall (Table 2.1). Two major types are identified as regular, which includes dome and arch type falls, and irregular, which encompass minor and sloughing or rashing. If l , h and w are used to denote the length, height and the width of the fall respectively, the terms circular ($l = w$), oval ($w < l < 2w$), linear ($l > 2w$), serpentine ($l > 2w$) with snake-like appearance in plan view and areal ($l < 2w$) are used to describe the fall in plan view.

1. *Dome type (1A)* , any fall that is circular or oval with $0.9m < h < w$.

2. *Arch type (1B)* , any fall that is linear or serpentine with $h < 0.9m$ or oval with $h > w$.

3. *Minor type (2A)* , any fall that is areal, linear or serpentine with $0.5m < h < 1.2m$.

4. *Sloughing or rashing* (2B) , any fall that is areal, linear or serpentine with $h < 0.5m$.

The development of localized stresses as a result of interruption of the rate of advance of the longwall panel has also been found to adversely effect the stability of the roof (Follington and Isaac, 1990).

Humidity has been identified as the single most important climactic phenomenon influencing roof falls under supported roof. Laboratory experiments on the effect of humidity on the physical properties of rocks have been previously evaluated (Haynes, 1975). Statistical links have been established between frequency of roof falls and humidity associated with certain seasons. Higher in-situ measurements of roof sag has been made in periods of high humidity (Moebs and Stateham, 1984). Compared to other factors that cause roof instability humidity is not a major concern unless unusually high amounts of

Table 2.3.1 Classification of roof falls by geometry (Patrick and Aughenbaugh, 1978)

General Description of Fall		Geometry of Fall					
		No discontinuity present at upper boundary of fall			Discontinuity present at upper boundary of fall		
1.Regular geometry	A. $0.9m < h < w$	Curved Contour Circular Dome 1		Curved Contour Oval Dome 2		Flat Topped Circular Dome3	Flat Topped Oval Dome 4
	B. $h > w$	Curved Contour Oval Arch 5	Curved Contour Linear Arch 6	Curved Contour Serpentine Arch 7	Flat topped Oval Arch 8	Flat topped Linear Arch 9	Flat topped Serpentine Arch 10
2.Irregular geometry	A. $0.5m < h < 1.2m$	Minor Areal 11		Minor Linear 12		Minor Serpentine 13	
	B. $h < 0.5m$	Areal Sloughing 14		Linear Sloughing 15		Serpentine Sloughing 16	

water are present.

In addition to in-situ monitoring, numerical methods have been widely used to analyze roof behavior (Wang et al., 1971; Park et al., 1992; Gale et al., 1992, Wang et al., 1974). Aggson (1979) has correlated the location of the initiation of failure of the roof to the ratio of horizontal stresses to vertical stresses. Three different loading conditions and their associated most probable failure modes have been found to be :

1. *Horizontal stress greater than vertical stress.* The most probable failure would be shear failure near the pillar or rib.

2. *Horizontal stress less than vertical stress.* The most probable failure would be the formation of a tensile fracture near the center of the roof.

3. *Horizontal stress equal to the vertical stress.* The most probable failure for this distribution would be a vertical shear failure near the pillar or the rib.

It has also been observed that the largest roof falls, forming an arch shaped void, were highly correlated with the apparent presence of low or tensile horizontal stresses (Patrick and Aughenbaugh, 1979).

2.3.1.1 Cutter Roof

Cutter roof is a special kind of roof failure that is commonly observed in the northern Appalachian coalfields. It causes massive roof failure with resultant delays in production, increased mining costs and loss of otherwise recoverable reserves. According to its definition, cutter roof "... initially begins as a fracture plane in the roof rock parallel to the rib, located at the roof-rib interface. The fracture propagates upward into the roof over the mine opening at an angle usually steeper than 60 degrees from the horizontal." (Bauer, 1989).

Figure 2.3.3 illustrates the various phases in the formation of a cutter roof. The cutter usually develops with a crack initiation phase in which a vertical crack is formed and propagates into the roof material. This is followed by an initial bed separation phase which provides the first clearly visible indications of instability. The crack formed propagates horizontally in the next phase followed by a massive collapse of the immediate roof afterwards. As can be seen this type of failure is particularly difficult to control because of the fact that although the mine

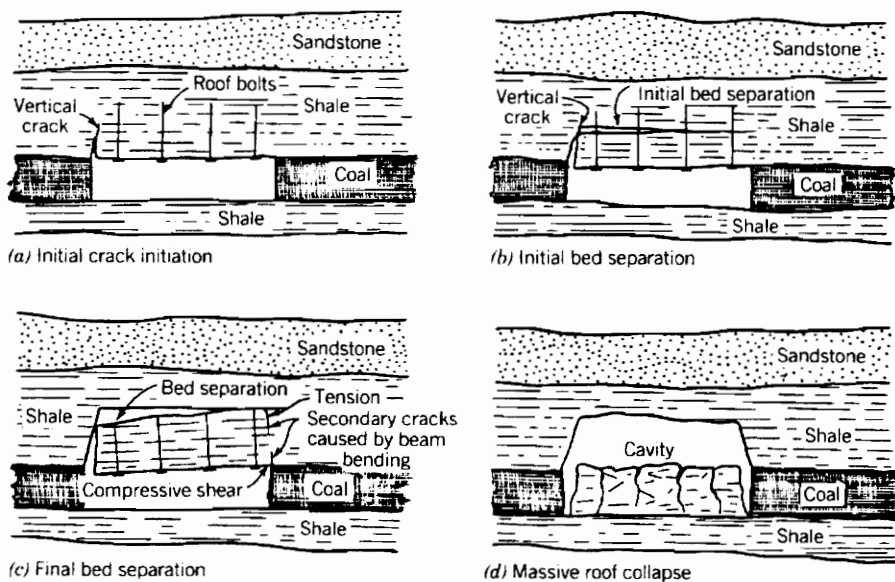


Figure 2.3.3 Phases of cutter roof (Kripakov, 1982)

roof is reinforced with conventional bolts, with time, massive roof collapse may still result. Therefore the most effective method of dealing with cutter is by prevention rather than trying to control the failure. In order to devise an effective ground control strategy against the formation of cutter roofs, the factors that cause the initiation of cutters need to be known.

Oddly enough, despite its widespread occurrence, the causes of cutter

roof are not thoroughly understood. Considerable research exists on the complex mechanisms involved in the formation of cutter roofs. However, there is continuing controversy regarding the exact causes of cutter roof failure.

Based solely on underground observations, Thomas (1950) proposed that two conditions were necessary to develop cutters in the roof line :

1. *a relatively strong immediate roof that may be thinly laminated, given that the cementation between the laminations must not break down easily.*

2. *a series of weaker strata that tend to sag and slowly load the immediate roof below it.*

Kripakov (1982) has summarized several factors that cause cutter roof failure to be :

1. *overburden stress*
2. *tectonic stress*
3. *lateral movement of the strata*
4. *variations in the strength of individual rock layers comprising the immediate roof*
5. *hydraulic pressure*
6. *gas pressure*
7. *variations in temperature and humidity*
8. *insufficient artificial support*

Again based on observation, frequent occurrence of clastic dykes have also been correlated to the number of cutter roof failures observed in a central Pennsylvania coal mine (Hill and Bauer, 1984). The findings were confirmed by

Innachione et al. (1984) who, through underground mapping showed that there indeed was a correlation.

Combining in-situ measurements and case studies with numerical analysis methods, six different categories of intrinsic failure mechanisms have been identified to cause cutter roof failure (Su and Peng, 1987). These are :

1. *Effect of high vertical stress*
2. *Effect of excess horizontal stress*
3. *Effect of relative stiffness between coal and its immediate roof*
4. *Effect of large topographic relief*
5. *Effect of bed separation and gas pressure*
6. *Effect of geological anomalies*

Ahola et al. (1991) has found that, the angle of the failure propagation observed in the field coincided with the plane of maximum shear stress, adding strength to the theory that excessive shear stresses are the principal causes of cutter roof failure.

Various methods that can be employed to control and/or prevent cutter roofs have been proposed based on the causes mentioned above. These are :

1. *Adoption of larger pillars while keeping the same opening width*

Larger pillars will ultimately reduce the absolute stress levels around the opening. Its quite easy and cheap to employ but the drawback is the loss of coal left as pillar unless some scheme of pillar recovery is also used.

2. *Adoption of smaller entries while keeping the same pillar size*

Using smaller entries essentially has the same effect as using larger

pillars. For deep mines experiencing high overburden stresses this method is most effective. The drawback is that ventilation and transportation are hindered.

3. Pillar softening

Pillar softening consists of drilling horizontal holes into the top of the coal pillar, thereby reducing its modulus of elasticity. Under hydrostatic stress conditions this method has the potential to significantly reduce the maximum shear stress levels around entry corners (Aggson, 1978).

4. Reorientation of the entries to a direction 45 degrees from the maximum principal horizontal stress plane

Reorientation of the entries to a direction parallel to the maximum horizontal stress direction would improve the roof conditions at entry corners, however the stability of the crosscuts will be jeopardized. Therefore orienting the entries at a 45 degree angle would be the best for both entries and crosscuts. Nevertheless, such an orientation may fail to be adequate in the presence of high overburden stresses.

5. Installation of angle bolts near the rib

Installing angle bolts at a direction approximately perpendicular to the fracture plane initiated at the corners will ultimately increase the resistance to shear failure. Mechanical bolts and combination bolts with pretension are best suited for this purpose.

6. Reorientation of the crosscuts while maintaining the same entry direction

In order to minimize the stress concentration around intersections the crosscuts maybe reoriented. This has a pronounced effect when the excess horizontal stress field is highly differential.

7. Installation of roof trusses and cribbing

When employed immediately after mining trusses and cribs are effective against development of cutter roofs. However, both means of support are quite expensive and in cribbing, although it is the most widely used means of secondary support, can hinder ventilation and transportation. Placement positions of the cribs are quite critical as well, as will be discussed in detail later on.

8. Application of yield pillars

Properly designed yield pillars will reduce the stress concentrations around entry corners. In essence, application of yield pillars employ the same mechanism as pillar softening. However, for high in-situ horizontal stresses and weak strata conditions with high laminations and poor bonding, the implementation of the yield pillar concept was found to be ineffective (Khair, 1992)

9. Application of caving entries

Driving a caving entry has been shown to create a stress relief zone around nearby openings given that they are close enough. This method is most effective for instances of high in-situ horizontal stress fields.

2.3.2 Geologic Structures

Another important factor that causes roof control problems are various geologic structures that are encountered during mining. Most of these structures are on small scale, usually in the order of few tens of feet in width. Premining investigations are the key to identify these structures, however, in the underground production environment they usually go unrecognized. Few number of boreholes are drilled as a part of the investigation stage in a typical coal mine

compared to an underground metal mine, due to the bedded and often predictable nature of coal deposits. This increases the likelihood of failing to identify certain geological structures which will then have to be dealt during the actual mining phase, usually resulting in increased operating costs.

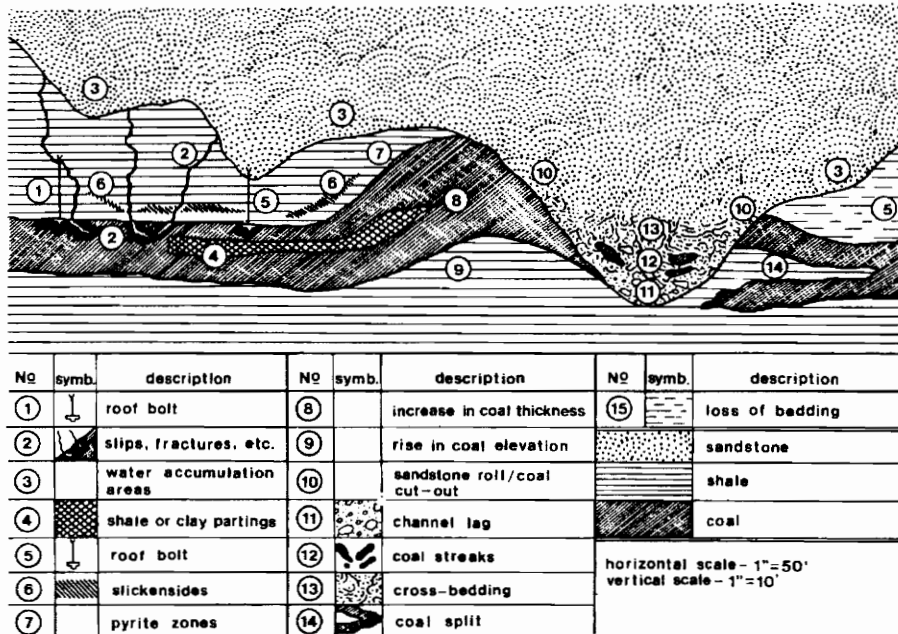


Figure 2.3.4 Recognizable characteristics of a sandstone channel (McCabe and Pascoe, 1978)

The most widely encountered geologic structures in the Appalachian coal mines are (Moebs and Ellenberger, 1980) :

1. *Paleochannels*

Paleochannels are trough shaped remnants of a stream channel that has been cut into older rock, such as roof shale. They are best recognized by their trough like shape, slickensided margins, horse belly appearance and generally hard crossbedded, poorly sorted sandstone fillings. Figure 2.3.4 illustrates the

recognizable characteristics of paleochannels. Paleochannels seldom exceed about 9.5 meters in width and range from a few meters to a few hundreds of meters in length. Because of their small dimensions and the wide spaced drillcore layout common for coal mines they can rarely be detected in advance of mining. However, paleochannels have been successfully located in advance of local faces by using Radio Imaging Method (Fry et al., 1985). In this method the coal immediately in front of the coal face is bombarded with electromagnetic waves which lose their energy intensity in relation to the type of strata they are penetrating through. Hence any anomalies immediately in front of the coal face are easily detected.

2. Scours

The term scour is used to designate an essentially highly curved, oblong, or elliptic shaped channel structure. They are caused by the erosional actions of ancient streams. Their presence in mine roof is more probable given that thick lenticular crossbedded sandstone is revealed close to the top of the coal seam through drilling.

3. Pinch out

Pinch-out is the abrupt termination of a roof stratum. It weakens the beam strength of the immediate roof. They are not easily detectable unless exposed by a roof fall however, distinct changes in the penetration rate of a rockbolting drill is may be an indicator of a pinch-out.

4. Slickenslides

Slickenslides are most common in highly argillaceous rocks such as shales and claystones. They can easily be identified by their smooth, polished and sometimes striated or grooved surfaces resulting from movement of rock on either side of the surface. Slickenslides are a common occurrence in

Appalachian coal mines and usually require additional means of ground support to prevent roof instability problems.

5. Clay Veins

Clay veins are wedge shaped masses of slickensided masses of claystone or mudstone filling in a crevice in a coalbed. They are particularly abundant where the immediate roof consists of a thick, clay rich rock although they have been observed under immediately beneath limestone, sandstone or shale strata.

6. Joints

A joint is a planar fracture in rock. In rock masses containing joints, the stability of the openings frequently depend on the stability of blocks formed by joint planes that intersect each other (Vaughan and Isenberg, 1991). Identifying the most critical joint planes that are most likely to slip is the most important part of analyzing the stability of the roof. The stability of the blocks formed by joint sets is governed by the internal angle of friction, cohesion and the direction and magnitude of the loads acting on the blocks.

There are significant differences in joint intensity and their effect on roof stability between central and eastern Appalachian coal fields. In the central Appalachian region for instance, the coal is nearly horizontal and jointing is not well developed resulting in almost no loss of roof stability. In the eastern Appalachian region however, abundance of folding and faulting has resulted in highly developed, close spaced joint sets which can significantly effect roof stability.

2.3.3 Roof support using cribs

Wood cribs are the most widely used means of secondary roof support in US longwall operations. Cribs are installed ahead of the critical stress abutment zone in the tailgate to stabilize the opening by providing resistance to deflection of immediate roof. Crib supports are simple in their design with a material cost of less than \$20 per feet of height, but their extensive use results in notable costs to coal mining (Barczak and Gearhart, 1993). Unlike steel arches, cribs cannot be used recursively, resulting in an increase in cost of support.

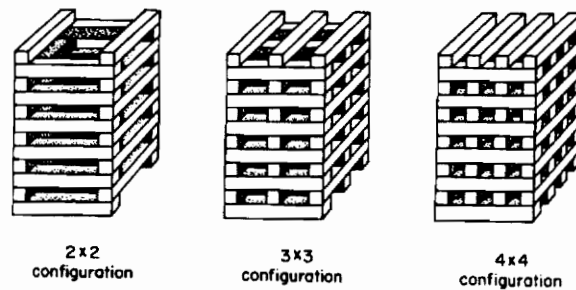


Figure 2.3.5 Wood crib configurations (Barczak and Gearhart, 1993)

Wood cribs are constructed by layering two or more parallel sets of timber. Subsequent layers are stacked on top of each other until the entry height is reached. The most common configuration used to erect cribs is the two timber blocks per layer (2 x 2) (Peng and Biswas, 1993). However, when ground conditions demand higher load carrying capacities three and four timber blocks per layer are also used. Figure 2.3.5 illustrates the most commonly used wood crib configurations. An interesting point to note is that the strength/cost ratio of a crib can be increased significantly simply by increasing the number of crib blocks in each layer. For example, the number of contact areas increase by 225% with a 50% increase in material usage when three piece crib is used instead of a two piece one (Jones, 1974).

There are various reasons for choosing wood as a support material. The most important characteristic of wood cribs, in terms of safety is that they rarely fail violently and provide visual and audible signs of loading. The load displacement characteristics of wood enable the cribs to deform as much as 30% strain as they take the load from the roof. Figure 2.3.6 illustrates the load vs. displacement characteristics of various crib arrangements.

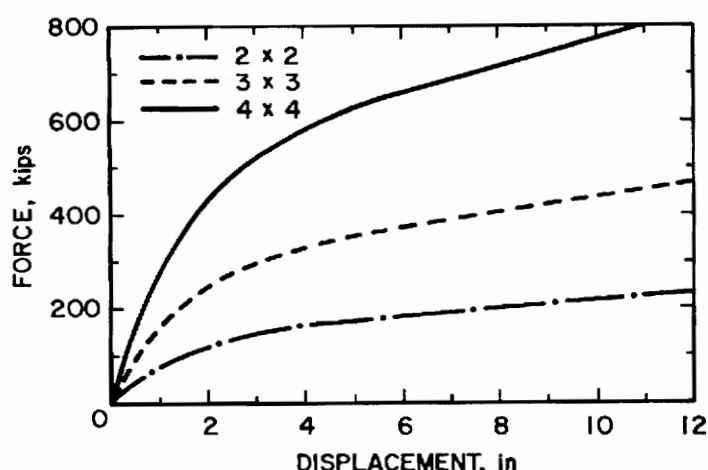


Figure 2.3.6 Comparison of crib performance (Barczak and Gearhart, 1993)

Timber is an orthotropic material. The deformation characteristics of timber is highly dependent on the loading direction. When timber is loaded parallel to the orientation of the grains it exhibits high initial stiffness and when the elastic strength is exceeded it fails suddenly. On the other hand, when timber is loaded perpendicular to the grain orientation it has elasto-plastic properties. The initial stiffness is low but when the elastic strength is exceeded it continues to yield in a somewhat controlled manner (Faure, 1993).

The key element in crib support selection is to design a structure that is compatible with the anticipated convergence (Barczak et al., 1989). Fiberglass

reinforced concrete cribs are normally stiffer than wood cribs but usually fail at a vertical convergence of approximately 3% of the height of the crib itself. Wood cribs, however, have far better deformability characteristics as much as 40% of their original height in response to applied load. The rate and the amount of convergence is not constant throughout the opening, therefore, the placement and the capacity of the crib should be tailored to the expected loading conditions. Attempts at modeling the performance of wood cribs have resulted in the following formula which estimates the crib resistance in kips (Barczak, 1993):

$$F_{crib} = A \times OHFCT \times PCTFCT \times \left(1 - e^{-(HTFCT) \times \delta}\right) + (PCTFCT \times ARFCT \times K_p \times \delta) \quad (2.19)$$

where :

A = compressive strength coefficient

K_p = plastic stiffness coefficient

δ = displacement

$OHFCT$ = overhanging timber factor

$HTFCT$ = height factor

$PCTFCT$ = percentage of contact area factor

$ARFCT$ = aspect ratio factor

There are several important factors that need to be considered before a stable secondary support system, utilizing cribs is installed. These are :

1. *Crib stiffness and capacity*

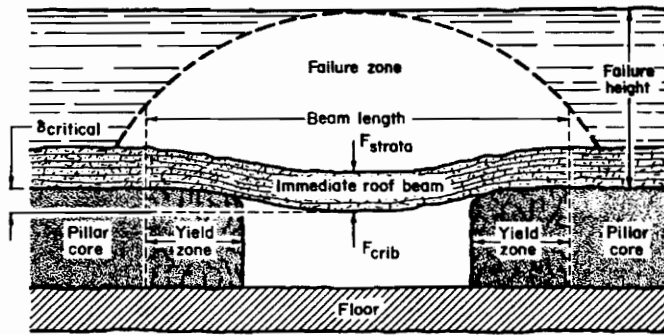


Figure 2.3.7 Crib strata interaction (Barczak, 1993)

Cribs should have adequate stiffness and the capacity to withstand the loads that are transferred from the roof. This requires the cribs to develop sufficient capacity within a displacement range that will offset the loading due to immediate roof. Figure 2.3.7 illustrates the reaction of cribs with the strata. There are various methods to estimate the loading that the crib is required to carry. The maximum required crib capacity is needed when the immediate roof has no strength to support itself and the full weight of the detached strata acts on the cribs. The critical loading of the roof beam in kips per feet is given by :

$$F_{critical} = \frac{4 \cdot t^2 \cdot \sigma}{3 \cdot L} \times 12 \times 10^{-3} \quad (2.20)$$

where :

t = beam thickness

σ = tensile strength of rock

L = length of roof beam

The critical beam deflection amount can be estimated in a similar manner by :

$$\delta_{critical} = \frac{5 \cdot \delta \cdot L^2}{24 \cdot E \cdot t} \quad (2.21)$$

where:

E = modulus of elasticity of roof rock

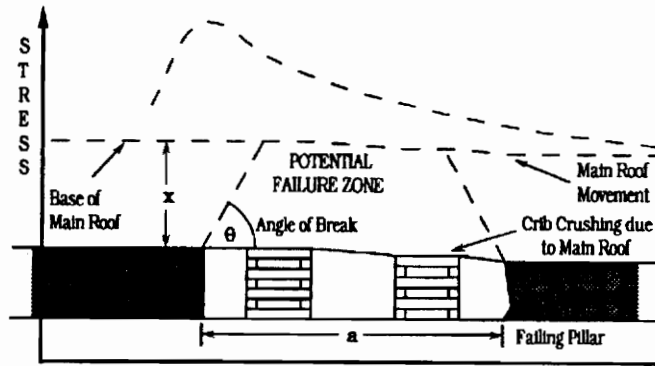


Figure 2.3.8 Tailgate geometry (Hosca and Haycocks, 1994)

A simpler method, utilizing the height and the angle of break, as illustrated in Figure 2.3.8 can also be used to estimate the magnitude of roof loading (Hosca and Haycocks, 1994) :

$$Load = \left(ax - \frac{x^2}{\tan \theta} \right) \gamma \quad (2.22)$$

where :

a = entry width

x = height of failure zone

θ = angle of break

γ = unit weight of the immediate roof

In practice failure heights of over 10 meters have been rarely observed (Zhou et al., 1991). Field observations have indicated that the angle of break is lies between 45 and 85 degrees. Typically supports should be designed to carry 10 meters of immediate roof which approximately amounts to 43 metric tons per meter of drift for a 6.5 meter wide opening.

In a similar computational manner, based on the geomechanics classification (RMR) value of the immediate roof, the support load can be estimated by (Unal, 1983) :

$$P = \left(\frac{100 - RMR}{100} \right) B \cdot \gamma \quad (2.23)$$

where :

RMR = the geomechanics classification value of roof rock

B = opening width

γ = the unit weight of immediate roof

2. Stability requirements

The main factor for ensuring crib stability over a range of loading conditions is the selection of proper timber length. Cribs should remain stable and provide support through a displacement function which includes vertical and horizontal movements of strata.

3. Roof and floor contact pressures

The roof and floor contact pressures are direct functions of the contact area and the loading. Care must be exercised to maintain an uniform contact area, which will avoid stress concentrations. A rough estimate of contact pressure can be made by dividing the crib load by twice the interlayer contact

area. Full timber length should not be used as an estimate of contact area as the load distribution is not uniform across the length of the timber block.

4. Costs

The cost of cribbing is directly related to the spacing between the cribs. Each crib must support a section of roof within a displacement that will control the integrity of the roof.

2.3.4 Roof Support Using Trusses

The roof truss concept was introduced as a means of supporting weak roofs too finely laminated and friable to provide adequate mechanical anchorage for vertical bolts used in beam building and too thick to be suspended by bolts from competent rock (White, 1970). Recently however, roof trusses are finding application as a means of secondary support in the tailgate area due to their advantages which include increased ventilation and manway access and reduced cost compared to cribs. (McCaffrey et al., 1994; Krupa and Long, 1994). Figure 2.3.9 illustrates a typical truss installation layout consisting of a two point anchoring system, a connecting bar, a turnbuckle to give the proper tension to the bar, bearing blocks and adjusting wedge box.

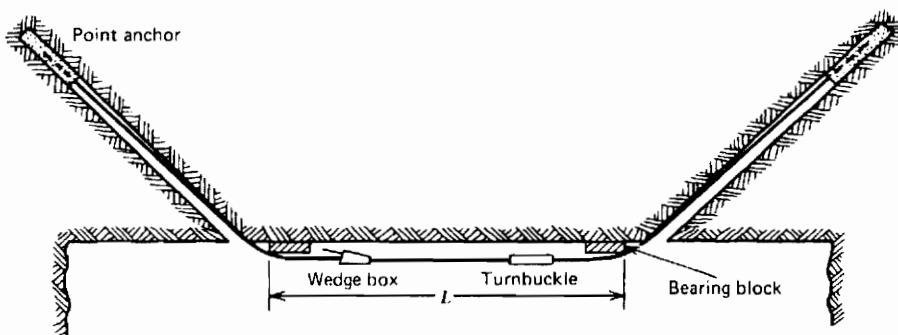


Figure 2.3.9 Typical layout of a truss installation (Biron and Arioglu, 1982)

In contrast to vertical bolting which either builds a beam or suspends for weaker, strata from stronger upper strata, the truss provides an uplift to the immediate roof at intermediate points between the ribs. This uplift is dependent on the anchorage capacity of the upper strata over the ribs which are relatively undisturbed by the removal of coal. Roof trusses are known to reduce dilation and therefore the height of the potential failure zone (Neall, 1975). Measurements made underground have shown roof lift during installation in finely laminated shale roofs (VPI&SU, 1981)

Figure 2.3.10 illustrates the forces involved in a typical truss installation. The tension P on the bar is established by the turnbuckle T . Through the bearing block $C-D(2a \times b)$ reaction R_2 is formed, and through the touching of the hole mouth reaction R_1 are formed. Resolving the forces along and perpendicular to the direction of T , and taking moments around point B gives :

$$T - \mu R_2 - R_1 \sin \alpha - P \cos \alpha = 0 \quad (2.24)$$

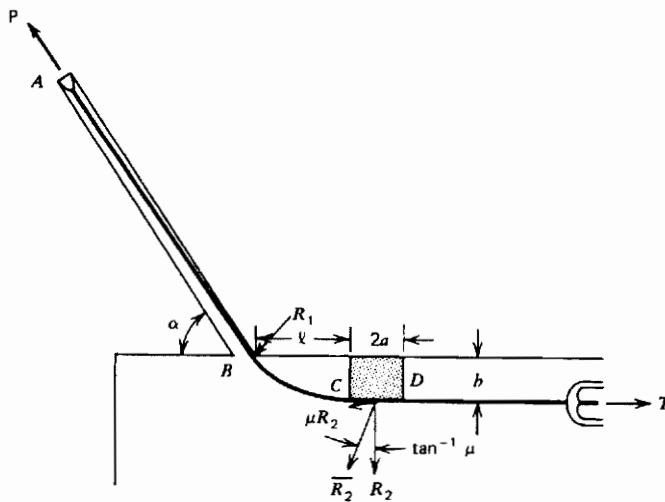


Figure 2.3.10 Statics of a roof truss (Biron and Arioglu, 1982)

$$R_2 + R_1 \cos \alpha - P \sin \alpha = 0 \quad (2.25)$$

$$R_2(a + l) + \mu R_2 b - Tb = 0 \quad (2.26)$$

Simultaneous solutions of these equations with respect to T gives :

$$P = \frac{T}{\mu b + a + l} [(a + l) \cos \alpha + b \sin \alpha] \quad (2.27)$$

$$R_1 = \frac{T}{\mu b + a + l} [(a + l) \sin \alpha - b \cos \alpha] \quad (2.28)$$

$$R_2 = \frac{Tb}{\mu b + a + l} \quad (2.29)$$

where :

P = anchorage load

T = tension in the truss

R_1 = reaction at the mouth of the hole

R_2 = reaction at the block

Field measurements have shown that its possible to install a truss system with a tension of 4 to 5 tons, however, subsequent monitoring has indicated that this load is quickly dissipated. Trusses are not effective where roof strata is prone to high horizontal closure as this dissipates the tension in the truss reducing support force (Grady and Fuller, 1992).

2.4 Pillar Response

Coal pillars are the portions of coal left in place to maintain the integrity of the openings. The behavior of the pillar under loading has a significant impact on the stability of the opening due to the interactions between the roof pillar and the floor. According to their use underground, there are three different types of pillars (Bieniawski, 1984). They are :

1. Support pillars

In situations where the main support to the roof strata is provided by a systematic arrangement of pillars. Room and pillar mining, and chain pillars left on the sides of the gateroads to protect the stability of the entries in longwall mining are best examples of support pillars.

2. Protective pillars

Protective pillars are used to protect either surface structures or underground excavations. In longwall mining barrier pillars that are left to protect adjacent panels are a good example for protective pillars.

3. Control pillars

Control pillars are used mainly to prevent rockbursts by controlling the energy release rate. They are most commonly employed in hard rock mines operating at great depths.

Pillars can also be classified as conventional pillars or yield pillars based upon their failure behavior. Conventional pillars are designed to support both the overburden weight and the mining induced loads that are imposed on the panel during the extraction of adjacent panels. The main goal in the design of conventional pillars is to increase the load bearing capacity of the longwall pillar

system to a point where the average anticipated pillar stress is below the failure stress. Generally, pillars yield progressively rather than sudden violent failure, however, as the mining depth increases the chances of having a violent pillar failure, or so called “bumps” also increase. This idea is employed in the design of yield pillars where the pillars are designed to fail under a controlled manner, transferring excessive stresses to more competent structures during the process (Kripakov and Kneisley, 1992). The use of yield pillars have also been shown to be effective in stabilizing roof conditions in regions of high horizontal stress (Haramy and Kneisley, 1989).

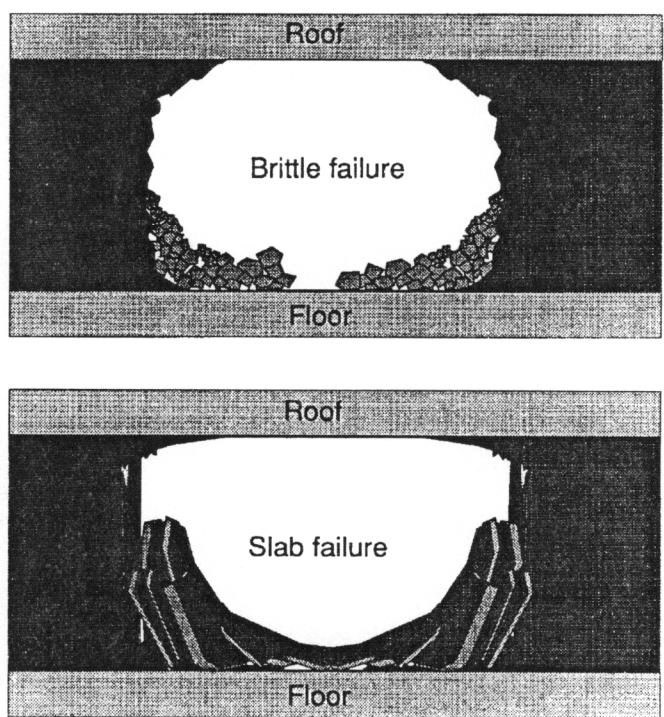


Figure 2.4.1 Pillar failure modes (Smith, 1992)

As mentioned earlier pillar failure is generally progressive. Although, massive pillar failures, in which the pillars in a complete section of the mine fail

(Khair and Peng, 1985) have been reported, rib falls are, by far the most common pillar failures observed in longwall mining. Rib falls can be classified into two main groups as brittle failures and slab failures as illustrated in Figure 2.4.1. Brittle failures of coal induces micro and macro fracturing resulting in granulated pieces of coal on the pillar rib. Slab failures, also called pillar spalling, result in blocky, plate like structures most commonly observed along a particular cleat or fracture surface. Owing to a somewhat lesser degree of fracturing and granularity, pillar slabbing usually indicates that the coal is relatively strong, compared to stress levels within the seam (Smith, 1992). The most common way to prevent rib failures is to bolt the ribs with headers or straps between the bolts, generating a wrapping surface around the rib line. Less commonly used are steel bands that are wrapped around the pillar and tightened, which rely on the same mechanism as bolting in order to establish stability (Peng, 1986).

The process of designing pillars involves the determination of size and placement positions that will be adequate to ensure stability of the opening. The expected load history including premining loads and mining induced abutment loads, stress distribution within the pillar, pillar strength and the interaction between the roof floor and the pillar need to be considered and quantized before pillars can be designed (Peng, 1986). There are two approaches for coal mine pillar design namely, the ultimate strength approach and the progressive failure approach (Bieniawski, 1984). The ultimate strength approach assumes that the pillar will fail as soon as the applied load on the pillars exceed the load bearing capacity. The load bearing capacity of the pillar is assumed to be zero once its strength is exceeded. The ultimate strength approach is used mainly in the design of conventional pillars by determining an overall or global safety factor, based on the average pillar strength and load, for the entire pillar. The progressive failure approach, on the other hand, signifies the existence of

defects or a non-uniform stress distribution within the pillar and assumes that the failure initiates at the most critical point(s) and propagates gradually to complete failure. However, the overall stability of the pillar can be maintained during this failure propagation phase. The progressive failure approach is inherently suitable in the design of yield pillars by assigning local safety factors according to pillar strength and loading at a given point. As there are variations in load and strength across the pillar itself, the local safety factors are a step closer to real world conditions.

Computers are becoming widely used in the pillar design process. The computer program ALPS (Analysis of Longwall Pillar Stability) has been developed by the USBM in order to aid mine operators in the design process (Mark, 1990;1989). ALPS is tailored to design conventional pillars and has three components :

1. *Estimating the load applied to the pillar*, by using the shear angle concept, which will be discussed under pillar loading.
2. *Estimating the load bearing capacity of the pillar system*, using Bieniawski's pillar strength formula.
3. *Calculation of a stability factor*, by taking the ratio of the two values.

However, the extensive use of computers in the pillar design process has been subject to criticism for being too quantitative and the acute stability assesment scheme (Parker, 1993).

2.4.1 Pillar Loading

In order to analyze the stability of a given pillar quantitative values of the loads that are acting on the pillar need to be known. The theories originally developed for the estimation of loading at depth are also used to estimate the

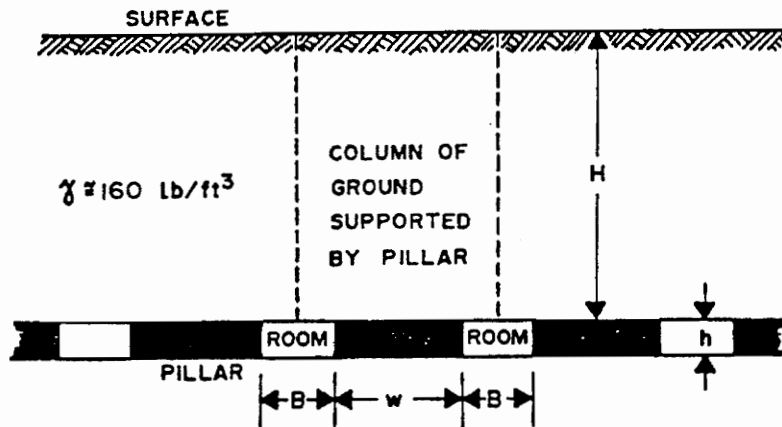


Figure 2.4.2 Tributary area concept for pillar loading (Bieniawski, 1984)

loads acting on the pillars. There are three approaches to estimate the loads acting on the pillars. These are :

1. Tributary area theory

Tributary area theory is based on the requirement that the pillar would at least be able to support the column of overburden that is overlying it as illustrated in Figure 2.16. Then the average stress acting on the pillar is given by :

$$S_p = \frac{S_v \cdot (w + B)(L + B)}{wL} \quad (2.10)$$

where :

S_v = virgin vertical stress

w = pillar width

L = pillar length

B = entry width

There are various shortcomings to this theory, most in the assumptions that are made in order to simplify the estimation process. The main assumptions made are :

1. The seam is only subject to vertical pressure which remains constant over the mined area. However, due to various stress transfer mechanisms the vertical pressure underground may be relieved to an extent.

2. The load is uniformly distributed over the cross-sectional area of the pillar. However, research have indicated that :

- a. *The stress distribution within a pillar is not uniform (Mark, 1990).*
- b. *The stress on pillars is proportional to extraction ratio*
- c. *The stress distribution is also dependent on width to height ratio*

It has been shown that due to the assumptions made by the theory the estimated stress magnitudes are approximately 40% higher than in-situ measurements (Hulstrulid and Swanson, 1981). However, due to its simplicity and conservative estimations the tributary area theory is widely used.

2. Shear angle concept

The effect of the abutment loads acting on the pillars are taken into consideration in this method by introducing the concept of a shear angle which is approximately equal to the angle of draw (King and Whittaker, 1971). This concept has already been described under the heading vertical stresses and will not be discussed here. It also suffers from assumptions made by the tributary area theory and failing to include stress transfer mechanisms.

3. Elastic-deflection theory

Based on the theory of elasticity Coates (1966) has derived an equation

which estimates ΔS_p , the incremental loading on a pillar due to mining (Eq. 2.11). The value estimated is then used inserted into Eq. 2.12 to obtain the total pillar stress.

$$\Delta S_p = \frac{2R - kh(1-w)(1-x^2+h) - wp(khn)}{hn + \pi(1-R)\left(\frac{1}{N}\right)\left(\frac{1+h}{1-x^2}\right)} \quad (2.11)$$

$$\frac{2 + 2Rb'(1-w)\pi}{}$$

$$S_p = \left(\frac{S_v}{S_v + 1} \right) \cdot \Delta S_p \quad (2.12)$$

where for plane strain conditions :

$$M = E / (1 - \nu^2) \quad b' = b / L$$

$$w = \nu / (1 - \nu) \quad x = x^* / l^*$$

$$k = \sigma_h / \sigma_v \quad h = h^* / l^*$$

$$n = M / M_p$$

where :

E = Young's modulus

L = breadth of mining zone

ν = Poisson's ratio

x^* = displacement in x direction

σ_h = horizontal stress on the seam

h^* = pillar height

σ_v = vertical stress on the seam

l^* = pillar length

b = width of the pillar

N = number of pillars

R = radial distance from center

S_v = virgin stress on the seam

The average pillar loads estimated by the above equations are approximately 40% lower than the values predicted by the tributary area theory and in close agreement with actual in-situ measurements. However, the complexity of the estimation equation hinders the practical applicability of the theory.

2.4.2 Pillar Strength

The next step after the determination of the loads acting on the pillar is to quantize the strength of the pillar so that the stability of the pillar can be assessed. There are numerous formulae that can be used to estimate the strength of a pillar.

1. *Obert Duvall formula*

Derived from laboratory tests on hardrock and elasticity theory the strength of a coal pillar can be calculated as (Obert and Duvall, 1967) :

$$\sigma_p = \sigma_1 \left(0.778 + 0.222 \frac{w}{h} \right) \quad (2.13)$$

where :

σ_1 = the uniaxial compressive strength of a cubical specimen

w, h = pillar dimensions

Assuming gravity loading this formula is claimed to be valid for w/h ratios of 0.25 to 4.0.

2. *Holland Gaddy formula*

Extending the previous work of Gaddy (1956), Holland (1964) proposed that pillar strength can be estimated via following :

$$\sigma_p = \frac{\sigma_c \sqrt{wD}}{h} \quad (2.14)$$

where :

σ_c = compressive strength of specimens tested

D = diameter or cube size dimension

w = pillar width

The expression $\sigma_c \sqrt{D}$ is also known as the Gaddy factor.

For w/h ratios of 9 to 10 the pillar strength will be underestimated because the formula does not consider effects of confinement. Later on, the formula was further improvised by Holland (1973) to :

$$\sigma_p = \sigma_1 \sqrt{\frac{w}{h}} \quad (2.15)$$

where :

σ_1 = strength of cubical pillars

3. *Salamon Munro formula*

Analyzing the results of a survey conducted in 125 mines in South Africa Salamon and Munro (1967) concluded that the strength of a pillar can be estimated in metric units by :

$$\sigma_p = 7.2 \frac{w^{0.46}}{h^{0.66}} \quad (2.16)$$

While deriving this formula it was assumed that the failed pillars in the database were too small and the intact pillars were properly designed.

4. *Barrier pillars formula*

The width of a barrier pillar which serves to separate unmined and mined out panels or to protect main entries can be estimated in feet by (Ashley, 1930) :

$$W_{bp} = 20 + 4h + 0.1H \quad (2.17)$$

where :

h = thickness of the coal seam

H = depth below surface

5. Panek's formula

Perhaps the most complex formula for estimating the strength of a coal pillar is given by Panek, (1980). Based on similitude modeling of the in-situ coal pillar Panek proposed the following formula for estimating pillar strength :

$$\begin{aligned} \sigma_p = E \cdot K_0 & \left[\frac{d_1}{w} \right]^{c_1} \left[\frac{w}{h} \right]^{c_2} \left[\frac{l}{w} \right]^{c_3} \left[\frac{E_r}{E_s} \right]^{c_4} \left[\frac{E_f}{E_s} \right]^{c_5} [\mu_{s/r}]^{c_6} [\mu_{s/f}]^{c_7} \\ & \times \left[\frac{1}{\nu_r} \right]^{c_8} \left[\frac{1}{\nu_f} \right]^{c_9} \left[\frac{1}{\nu_s} \right]^{c_{10}} \left[\frac{d_1}{d_2} \right]^{c_{11}} \left[\frac{d_1}{d_3} \right]^{c_{12}} \end{aligned} \quad (2.18)$$

where :

l, w, h = pillar dimensions

d_1, d_2, d_3 = discontinuity spacing in each direction

ν_s, ν_r, ν_f = poisson's ratios of seam, floor and roof

$\mu_{s/r}, \mu_{s/f}$ = coefficient of friction at roof and floor contacts

E_s, E_r, E_f = youngs moduli of seam, floor and roof

K_0, c_i = coefficients

Even though, this method incorporates the size effect, shape effect and mechanical properties of roof, seam and floor, it has severely limited applicability in practice due to the large number of parameters that have to be determined before the formula can be put into use.

In addition to numerous formulae postulated, rock mass classification (RMR) has also been applied to estimating the strength of coal pillars (Trueman et al., 1992). Following a similar approach, Vutukuri and Hossaini, (1992) have investigated the applicability of the strength criteria used for rock and rock mass to coal pillars, through extensive laboratory testing and statistical analysis.

2.5 Floor Response

Derived directly from soil mechanics theory, bearing capacity analysis of the floor has been used to quantize the stability of the floor. However, this method is applicable to room and pillar mining and its suitability for longwall mining is under question. The major factors that have been observed to effect the bearing capacity of the floor are (Bieniawski, 1987) :

1. *Applied load*
2. *Moisture content*
3. *Strata compressibility*
4. *Layering of strata*
5. *Material compressive and shear strengths*
6. *Horizontal stress in non-clay floor strata*
7. *Claystone thickness and continuity*

Stability problems in the coal mine floor commonly manifest themselves in two distinct observable modes of failure. These are floor heave and floor buckling (Peng, 1986). Floor heave is caused by the difference in the modulus of elasticity between the coal and the floor rock when the floor rock is relatively weaker than the coal. The most common occurrence of floor heave is due to pillars punching into the floor. Figure 2.5.1 illustrates the various stages of floor heave. White, (1956) has proposed that floor heave generally occurs in areas that contain large amounts of clay, montmorillonite in particular. The moisture content is a significant factor for the elastic-plastic behavior of clay. Pittsburgh fireclay for instance, has a compressive strength of 5000 psi when dry, but once saturated with water the compressive strength drops to 2000 psi (Peng, 1986).

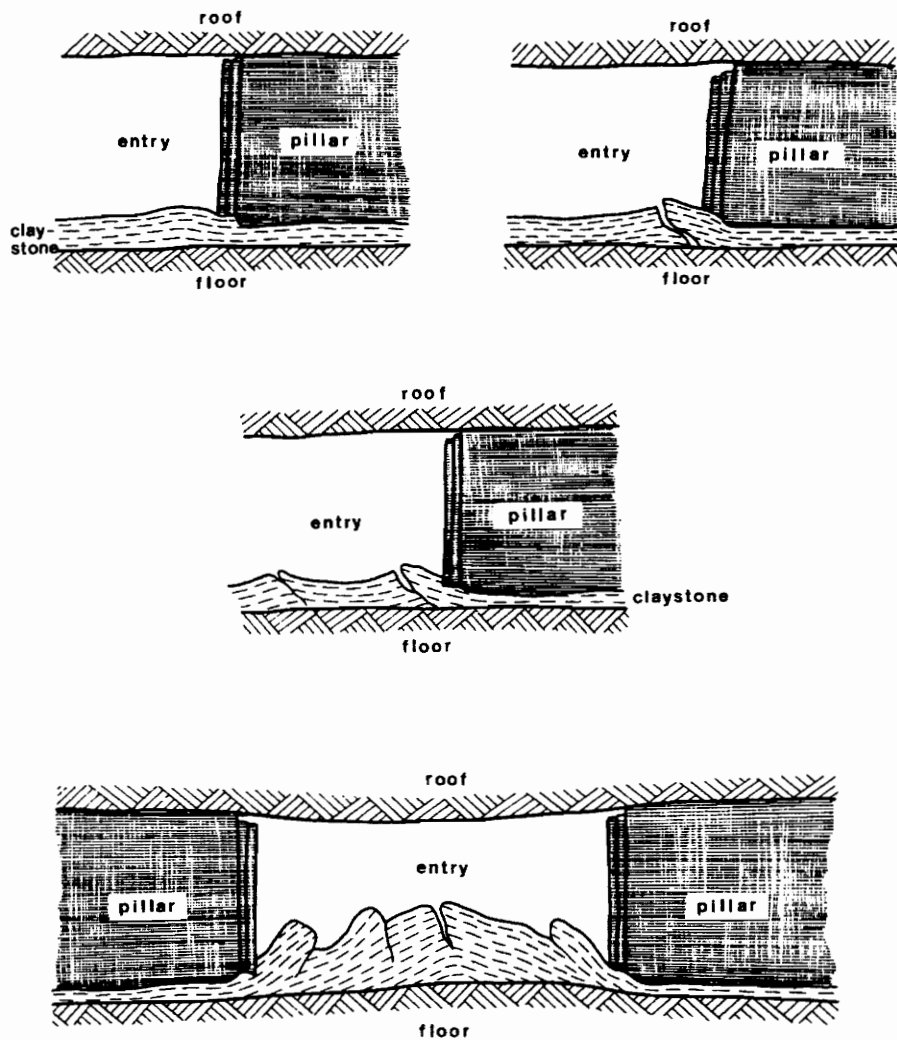


Figure 2.5.1 Stages of floor heave (Peng, 1986)

Floor bolting has been found to be effective in controlling floor heave (Cowan and Sharpe, 1953). Driving roadways along the strike and parallel to the cleat direction of the coal as a remedial measure to prevent heaving has been suggested by Afrouz (1975). Based on scale model studies, the removal of heaved material should not be considered as a means of controlling floor heave as such an operation has an adverse effect on floor behavior, causing it to heave more (Hobbs, 1969). Utilization of yield pillars has been successful in

not only controlling roof problems but also in providing better floor conditions as well (Carr et al., 1985).

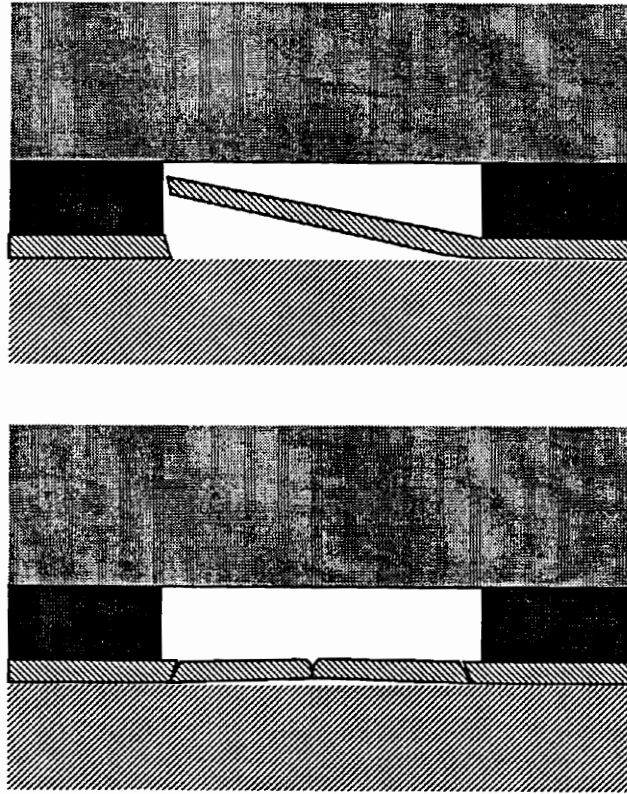


Figure 2.5.2 Types of floor buckling (Aggson, 1978)

Floor buckling, on the other hand, is observed when the floor rock is stiffer than the coal, causing the floor rock to break in sections. Normally the breakage is observed to initiate around the pillar-floor intersection. However, the failure initiates at the center when the sides are pinned down as illustrated in Figure 2.5.2. Floor buckling is highly correlated with relatively thin but stiff strata and high horizontal stress (Aggson, 1978).

CHAPTER 3.

Evaluation of Stability

For engineering purposes the stability of the underground workings need to be quantized as a part of the design phase, before any mining operations are carried out. Sound judgements on the anticipated working conditions, particularly in ground control have to be taken into consideration. For any design purpose there is a basic need to evaluate the field phenomenon quantitatively. The stability of the underground openings are especially important due to their impact on the mining operations. There are essentially three different methods that can be used to predict and hence evaluate the ground behaviour of underground openings.

3.1 Closed form solutions

In theory, virtually all physical phenomenon in nature can be expressed as a closed form solution. However, in reality closed form solutions to physical problems can be found only for relatively simple problems that exhibit well defined boundary conditions. The two most popular closed form solutions that

are applied to the analysis of stability of an underground opening are the beam and plate problems, originated in civil engineering work, where the immediate roof is treated as a uniform beam and the entry intersections are represented as plates. However, the bold assumptions that need to be made in order to reduce the problem of stability to a beam or a plate imposes severe limitations on representing the real world conditions that are encountered in-situ.

3.2 Empirical methods

Empirical methods are based on statistical analysis of observations regarding a particular set of conditions. Various parameters are noted and correlations are made among the parameters and stability. Such parameters can be discontinuity spacing, ground water conditions, physical dimensions etc. Numerical values are assigned to each parameter and some sort of function, that will factor in all the considered parameters numerically, is generated. The outcome of this function is used as an estimate of stability. There are a number of methods available which may be used to evaluate the stability of an opening, each with their own database of case studies and various parameters that are taken into consideration.

3.2.1 Terzaghi's rock load classification

Originated in 1946 and based on civil engineering work Terzaghi's rock load classification system is perhaps the earliest attempt in order to predict loading and hence the stability of an underground opening (Terzaghi, 1946). The method is proposed to estimate the loads that are acting on steel set tunnel supports as illustrated in Figure 3.1. However, although this classification system is appropriate for its purpose, it is not suitable for use in coal mines.

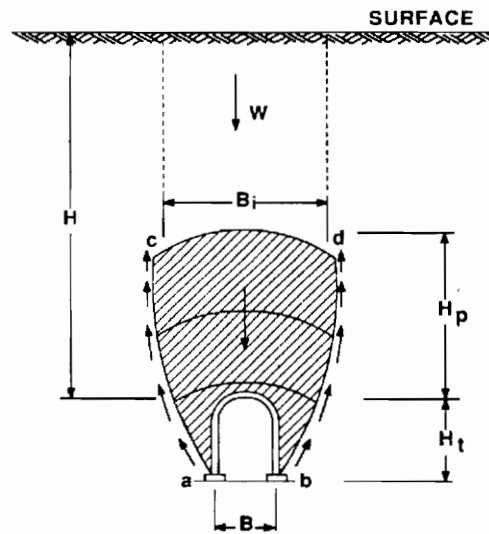


Figure 3.2.1 Loading on tunnel supports (Terzaghi, 1946)

Table 3.2.1 Terzaghi's rock load classification (Terzaghi, 1946)

Rock Condition	Rock Load H_p in ft	Remarks
1 Hard and intact	zero	Light lining required only if spalling occurs
2. Hard stratified schistose	0 to 0.5 B	Light support, mainly for protection against spalls
3. Massive, moderately jointed	0 to 0.25 B	Load may change erratically from point to point
4. Moderately blocky and seamy	$0.25B$ to $0.35(B+H_t)$	No side pressure
5. Very blocky and seamy	$(0.35 \text{ to } 1.1)(B+H_t)$	Little or no side pressure
6. Completely crushed but chemically intact	$1.1(B+H_t)$	Considerable side pressure
7. Squeezing rock, moderate depth	$(1.1 \text{ to } 2.1)(B+H_t)$	Heavy side pressure
8. Squeezing rock, great depth	$(2.1 \text{ to } 4.5)(B+H_t)$	Circular ribs recommended
9. Swelling rock	upto 250 ft	Yielding support

The main features of Terzaghi's classification system are given in Table 2.3.1. Revisions of this classification scheme has been proposed which suggest

that the rock loads for classes 4 to 6 can be reduced by as much as 50% given that the tunnel is driven above the water table (Rose, 1982). The major criticism of Terzaghi's classification system is that its too general and does not provide any quantitative on rock mass properties (Cecil, 1970).

3.2.2 Rock quality designation (RQD) index

The rock quality designation index is a modified core recovery percentage which incorporates only the intact pieces of core that are greater than 10 cm. in length (Deere et al., 1967). It is primarily used to identify low strength rock zones within the rock mass. The International Society of Rock Mechanics recommends the use of double tube core barrels and a core size of at least NX diameter for RQD determination. The following relationship between the RQD index and the engineering quality of rock has been proposed (Deere, 1968) :

<u>RQD (%)</u>	<u>Rock Quality</u>
< 25	Very Poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

The application of the RQD index in tunnel support requirement estimation is illustrated in Figure 3.2.2.

Presently the RQD index is used as a standard parameter in drill core logging and forms a basic element of the two major rock mass classification systems, namely the RMR system and the Q system.

The RQD index by itself is not sufficient to provide adequate description of rock mass because it does not take into account the rock joints that contain clay fillings and weathered materials (Merriit, 1972).

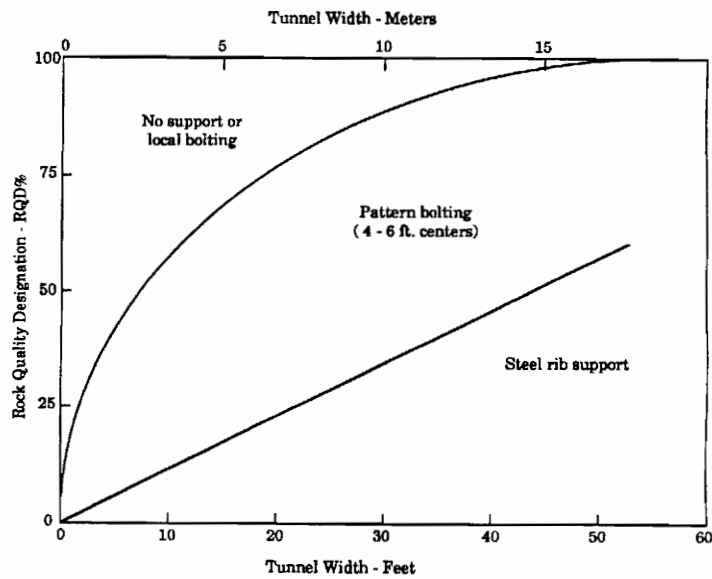


Figure 3.2.2 Rock support assesment from RQD values (Merrit, 1972)

3.2.3 ISRM classification

A general geotechnical description of rock masses aimed at characterizing and clasifiying the various regions that constitute a given rock mass has been developed by the International Society of Rock Mechanics (ISRM, 1981). However, the ISRM classification is not considered to be as exhaustive as the geomechanics classification or the Q-system, which will be dealt with in subsequent headings. The following characteristics are taken into account for this particular classification scheme :

1. *Rock name, with a simplified geologic description*
2. *Layer thickness and discontinuity spacing*
3. *Uniaxial compressive strength and the angle of friction of the fractures*

The suitable intervals of values and their associated descriptions are :

Discontinuity Spacing

<u>Intervals (cm)</u>	<u>Terms</u>
> 200	Very wide
60 - 200	Wide
20 - 60	Moderate
6 - 20	Close
< 6	Very close

Uniaxial Compressive Strength (MPa)

<u>Intervals (MPa)</u>	<u>Terms</u>
> 200	Very high
60 - 200	High
20 - 60	Moderate
6 - 20	Low
< 6	Very low

Angle of Friction of the Fractures

<u>Intervals (deg)</u>	<u>Terms</u>
> 45	Very high
35 - 45	High
25 - 45	Moderate
15 - 25	Low
< 15	Very low

3.2.4 Geomechanics classification (RMR)

The geomechanics classification system, otherwise known as the rock mass rating (RMR), was developed by to provide a comprehensive and quantitative description of various types of strata encountered in tunneling and other underground excavations (Bieniawski, 1973). The RMR system uses six different parameters to identify the engineering quality of rock mass. These are :

1. *Uniaxial compressive strength of rock material*

2. The rock quality designation (RQD) index

3. Spacing of discontinuities

4. Condition of discontinuities

5. Groundwater conditions

6. Orientation of discontinuities

To apply the geomechanics classification, rock mass is divided into a number of structural regions such that certain features are approximately uniform within each region. However, it must be pointed out that each of the six parameters used in this classification do not necessarily contribute in equal amounts to the classification scheme. A quantitative rating is assigned to each parameter from a range of values for that parameter. The rock mass rating is then obtained by merely summing the individual parameter ratings. The

Table 3.2.2 Geomechanics classification (Bieniawski, 1989)

Parameters		Range of Values				
Strength of intact rock material	Point-load strength index (MPa)	> 10	4-10	2-4	1-2	For this low range, uniaxial test is preferred
	Uniaxial compressive strength (Mpa)	> 250	100 - 250	50 - 100	25 - 50	5-25 1-5 < 1
	Rating	15	12	7	4	2 1 0
	Drill Core Quality RQD (%)	90 - 100	75 - 90	50 - 75	25 - 50	< 25
	Rating	20	17	13	8	3
	Spacing of discontinuities	> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm
	Rating	20	15	10	8	5
	Condition of discontinuities	Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge > 5 mm thick or Separation > 5 mm Continuous
	Rating	30	25	20	10	0
	Inflow per 10 m tunnel length (L/min)	None	< 10	10 - 25	25 - 125	> 125
Ground-water	Ratio ~~Joint water pressure~~ Major principal stress	0	< 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5
	General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating	15	10	7	4	0
	Adjustments for Strike and Dip Orientations of Discontinuities	Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings		0	-2	-5	-10	-12

geomechanics classification is given in Table 3.2.2.

This overall rating can then be used to estimate the typical standup time, that the opening will sustain its stability before any additional support is applied, for an underground opening with respect to the opening width, as illustrated in Figure 3.2.3.

The RMR value obtained can also be correlated with the Q-index, which

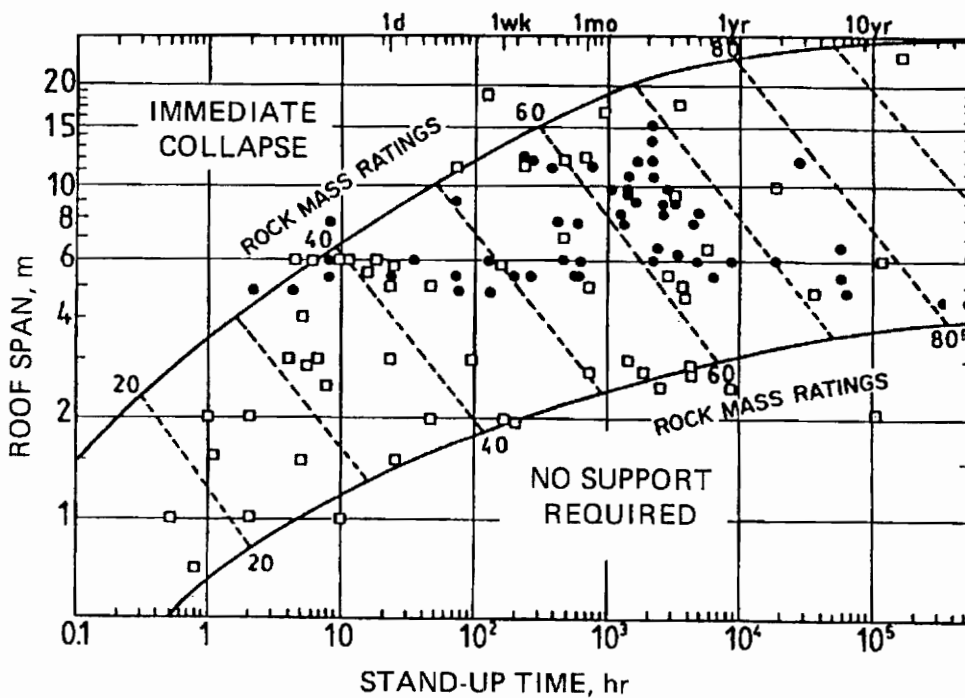


Figure 3.2.3 Standup time vs. RMR values (Bieniawski, 1989)

will be described in the next heading, by using the following formula (Bieniawski, 1989) :

$$RMR = 9 \cdot \ln Q + 44 \quad (3.1)$$

The RMR classification has found wide popularity in the industry, however, it should be used in conjunction with observational and analytical

methods to formulate an overall design methodology that is compatible with the design objectives and site geology.

3.2.5 Q-System of rock mass classification

The Q-system of rock mass classification was developed in Norway, based on an analysis of 212 tunnel case histories from Scandinavia (Barton, 1976). Similar to the geomechanics classification the Q-system also uses six different parameters of rock mass :

1. *Rock Quality Designation (RQD)*
2. *Number of joint sets*
3. *Roughness of the most unfavorable joint or discontinuity*
4. *Degree of alteration or filling along the weakest joint*
5. *Water inflow*
6. *Stress condition*

These six parameters are grouped into three quotients to give the numerical value of the overall rock mass quality as :

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF} \quad (3.2)$$

where :

J_n = joint set number

J_r = joint roughness number

J_a = joint alteration number

J_w = joint water reduction number

SRF = stress reduction factor

The numerical range of values for Q can range from 0.001 to 1000 on a

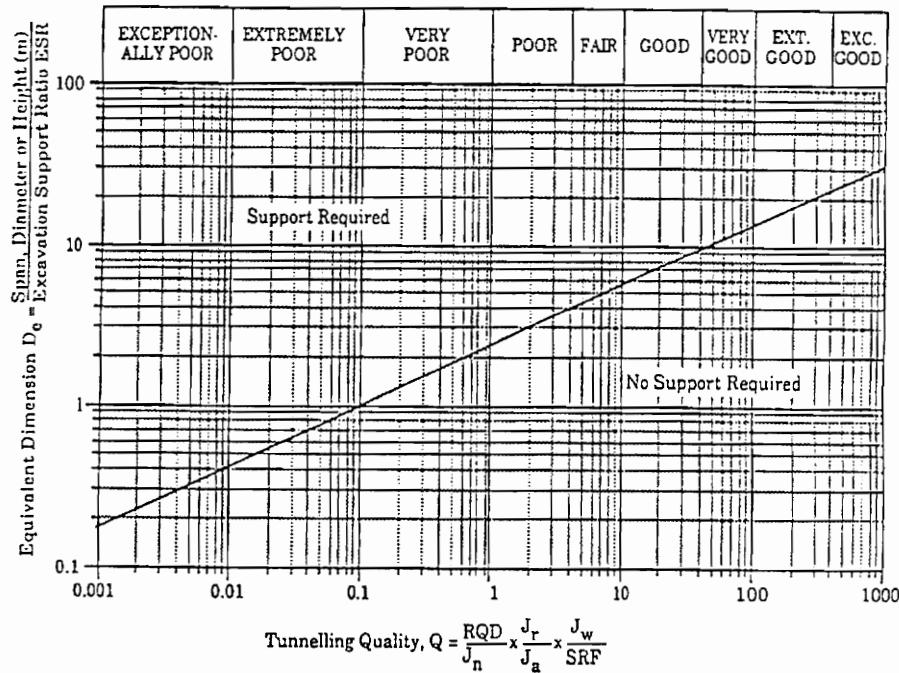


Figure 3.2.4 Relationship between D_e and the Q index (Barton et al., 1976)

logarithmic rock mass quality scale. Again similar to the RMR system these six parameters are assigned numerical values and finally summed to give the quality of rock mass. The first two parameters represent the overall structure of the rock mass, and their quotient is a relative measure of the size of blocks. The quotient of the third and fourth parameters is an approximation of the interblock shear strength of the joints. The fifth parameter is a yardstick for water pressure whereas the sixth and the final parameter is a measure of the combination of the amount of loosening load in the case of shear zones and clay bearing rock, the

relative magnitude of the rock stress in competent rock and the amount of squeezing and swelling loads in plastic incompetent rock. The Q value obtained is related to an equivalent dimension, D_e indicating support requirements for a given set of conditions as illustrated in Figure 3.2.4.

3.2.6 Coal Mine Roof Rating (CMRR)

The CMRR classification scheme was introduced as an attempt to classify the roof conditions exclusively for coal mines (Molinda and Mark, 1992). The CMRR combines the best aspects of available rock mass classification systems with extensive experience in coal mine ground control. Its unique features are :

1. It focuses on the characteristics of bedding planes and other discontinuities such as slickensides that determine the structural competence of the roof
2. It treats bolted mine roof as a single structure and considers the contributions of the different beds within the bolted interval, emphasizing on a strong and competent layer
3. It is applicable to all US coalfields, and allows for a meaningful comparison of structural competence even for cases when the stratigraphic sequences are quite different.

Similar to geomechanics classification the CMRR also employs a scale from 0 to 100. The components of CMRR are the following parameters for each layer in the roof :

1. *Compressive strength*
2. *Weatherability*
3. *Discontinuities*

- 4. Cohesion
- 5. Roughness
- 6. Spacing and persistence
- 7. Ground water
- 8. Other information

Using tables the data obtained in the above-given fields are converted into numeric values for each layer in the roof rock. Then as a second phase these values combined by a thickness weighted average and roof adjustments are made to arrive at the final CMRR value. Figure 3.2.5 illustrates the distribution of CMRR by various regions in the US.

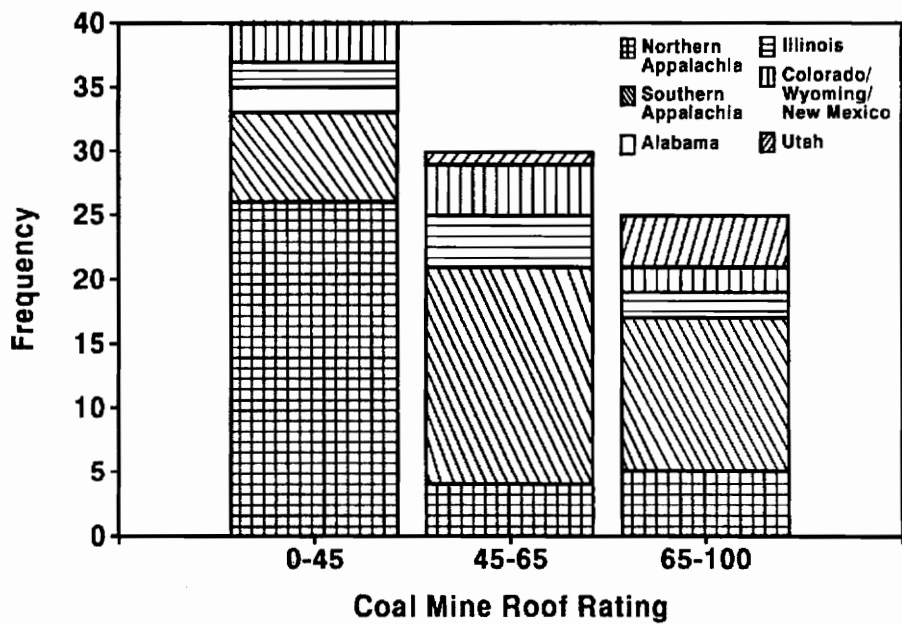


Figure 3.2.5 Distribution of CMRR values by region (Molinda and Mark, 1992)

3.3 Numerical Methods

The third approach to evaluate the stability of an underground opening is by the use of numerical techniques. As mentioned earlier virtually any physical phenomenon can be represented as a closed form solution. However, in reality the vast number of parameters that effect the system as a whole is quite difficult to define. The common approach used used by various mathematical procedures is to divide the problem into smaller physical and mathematical regions and then to sum the influence of each of these regions to approximate the behavior of the total system. The problem is generally formulated as a set of simultaneous equations which are usually assembled in matrix form. The solution can then be obtained by using a suitable matrix solving technique (Hoek et al., 1991).

The state of stress and the corresponding displacement field for an arbitrary geometric shape can be obtained by a closed form solution. However, this is only possible when the boundary conditions, strain compatibility equations, constitutive relationships for the materials and the differential equations of the state of equilibrium are known (Brady and Brown, 1985).

Numerical methods can be divided into two major groups depending on the formulation of the problem (Zienkiewicz, 1985) :

1. *Differential methods*

2. *Integral methods*

In differential methods such as, the finite element method and the finite difference methods, the region of interest is divided into simple geometric shapes whereas, in integral methods only the boundaries of the problem are broken into smaller sections, as in the boundary element method.

3.3.1 Integral Methods

In all of the integral methods, approximations are made on the boundary of the excavation only, treating the rock mass as an elastic continuum. Two different methods are available to accomplish these approximations :

1. *Indirect method*

2. *Direct method*

In the indirect method, sometimes referred to as fictitious stress method, an iterative procedure is used to adjust the magnitude of each “fictitious” force which is assumed to act in the opposite direction to the stresses generated by the excavation, until the sums of the two components (horizontal and vertical) applied to each segment, and the effect on that segment of the two components applied to every other segment, result in forces on each segment corresponding to the induced stress due to the excavation. Once this has been accomplished, the induced components of stress and displacement anywhere in the rock mass along the boundaries of the excavation can be found by summing the effect of forces on each segment. To find the total state of stress the induced and virgin stresses are summed (Jaeger and Cook, 1979). The boundary element method is a good example of integral methods using a fictitious stress field to approximate the stress and displacement characteristics on the problem boundaries.

In the direct method, Betti’s reciprocal theorem is used to prevent dealing with fictitious forces or displacements (Rizzo, 1967). The solutions for the unknown stresses and displacements are found directly from the boundary conditions.

Integral methods have the inherent ability to discretize the problem geometry with fewer elements which, in turn, translates into less computing time.

However, the ability of integral methods to handle different material types in a given problem geometry is quite limited.

3.3.2 Differential Methods

The differential methods available for analyzing the stability of an underground opening are the finite difference method and the finite element method. The finite element method involves an exact solution of a differential approximation to the problem, whereas the finite difference method involves an approximate differential solution to an exact problem. The methods are essentially similar and the principal differences arise out of the techniques by which the set of simultaneous equations are solved.

3.3.2.1 *The Finite Element Method*

The finite element method is a powerful numerical technique that employs variational methods and interpolation theory for solving differential equations of initial and boundary value problems (Desai and Abel, 1972). In this method the region of interest is divided into a collection of smaller sections called finite elements. Each of these elements are connected at joints named as nodes. The distribution of stress and displacement over an individual finite element is represented by simple functions. The forces acting on the body, both the surface and the regional forces are replaced by a statically equivalent system of forces acting on these nodal points. The fundamental assumption made is that the stress and strain are homogeneous in each element. Linear equations are generated for each element. The equilibrium condition is obtained by combining the equations of the individual elements in such a fashion that the displacements are conserved at the nodal points. Matrix algebra techniques are used to solve this system of simultaneous equations resulting from the application of boundary conditions.

There are eight basic steps in the formulation and application of the finite element method (Desai, 1979). These are :

1. Discretization and the selection of element configuration

In this step the region of interest is divided into a suitable number of smaller regions called finite elements. The intersection of the sides of the elements are referred to as nodes. The selection of elements depends on the specific problem dimensions. For a single dimensional problem the element type used is a line element, accordingly for a two dimensional problem triangular and/or quadrilateral elements can be used. For a three dimensional problem the element of choice is most likely to be a hexahedron.

2. Selection of approximation functions

This step involves the selection of a proper mathematical distribution for the magnitudes of stress and/or displacement. These distributions are evaluated at the aforementioned nodal points. Polynomial functions are generally preferred to be used as mathematical approximation functions due to the ease and simplification that they offer in formulating the finite element problem.

3. Definition of strain-displacement and stress-strain relationships

In this step mathematical links are formed between strains and displacements and stresses and strains using certain principles. For example : the magnitude of strain in a single dimension given that the magnitude of deformation occurring only in the same direction is given by :

$$\varepsilon_y = \frac{dv}{dy} \quad (3.1)$$

where :

v = deformation in the y direction

In addition to defining strain in terms of displacement, the stresses must also be defined in relation to displacement. In order to establish this link the relationship between stress and strain is used. The most simple case is illustrated by the use of Hooke's law, which relates stress to strain in a solid body as :

$$\sigma_y = E_y \cdot \varepsilon_y \quad (3.2)$$

where :

E_y = Young's modulus of elasticity in the y direction

By combining these two relationships an expression for stress in terms of displacements can be obtained as :

$$\sigma_y = E_y \cdot \frac{dv}{dy} \quad (3.3)$$

4. Derivation of element equations

The laws and principles are applied to to obtain equations that govern the behavior of element. The equations are derived in general terms and hence are applicable to all elements in the dicretized region. The two most commonly used methods for deriving element equations are the energy methods and the residual methods. Energy methods involve the use of variational calculus which use the principle of virtual work to identfy consistent states within the body. The residual methods use differential calculus to establish element equations.

In its most general form element equations for stress analysis can be expressed in the form :

$$[\mathbf{k}]\{\mathbf{q}\} = \{\mathbf{Q}\} \quad (3.4)$$

where :

$[\mathbf{k}]$ = stiffness matrix

$\{\mathbf{q}\}$ = vector of nodal displacements

$\{\mathbf{Q}\}$ = vector of nodal forces

5. Assembly of element equations and introduction of boundary conditions

Assembly of element equations is established simply by recursively using Eq. 3.4 for each element in the domain. This assembly process is based on the law of continuity, that is the adjacent points need to be in the neighborhood of each other after the load is applied. Recursive use of Eq. 3.4 for each element gives the assembly equations, which can be expressed in matrix notation as :

$$[\mathbf{K}]\{\mathbf{r}\} = \{\mathbf{R}\} \quad (3.5)$$

where :

$[\mathbf{K}]$ = assembly stiffness matrix

$\{\mathbf{r}\}$ = nodal displacement vector

$\{\mathbf{R}\}$ = nodal load vector

Eq. 3.5 is a measure of the capabilities of the entire region of interest to withstand externally applied forces. The next step is to apply the boundary conditions to this assembled region. Boundary conditions are in essence, constraints or supports that must exist so that the region of interest can stand in space uniquely. In order to reflect the boundary conditions in the finite element approximation of the region represented by Eq. 3.5 it is required to modify these equations by the geometric boundary conditions. Doing so reveals the final modified assembly matrix for the entire region of interest as :

$$[\mathbf{K}]\{\bar{\mathbf{r}}\} = \{\bar{\mathbf{R}}\} \quad (3.6)$$

6. *Solution for primary unknowns*

When written in standard form Eq. 3.6 becomes :

$$\begin{aligned} K_{11}r_1 + K_{12}r_2 + \cdots + K_{1n}r_n &= R_1 \\ K_{21}r_1 + K_{22}r_2 + \cdots + K_{2n}r_n &= R_2 \\ &\vdots \\ K_{n1}r_1 + K_{n2}r_2 + \cdots + K_{nn}r_n &= R_n \end{aligned} \quad (3.6a)$$

This set of simultaneous equations can be solved for the unknown displacements $(r_1, r_2, \dots, r_n)$ by iterative methods or Gaussian elimination. These values are called primary unknowns simply due to the fact that they are the first quantities sought in Eq. 3.6a. The designation of the word primary will depend on the way the problem is formulated.

7. *Solution for secondary quantities*

Once the primary unknowns are solved, secondary quantities can be obtained by using the previously defined mathematical relationships. For example, if the problem is formulated in terms of stress the primary unknowns will be the stress values at the given nodal points. The previously established relationship between stress and displacement can be used to obtain the values of displacement, as secondary quantities at the same nodal points or vice versa.

8. *Interpretation of results*

The final step in finite element analysis is to present the results obtained by the numerical procedure in a form that can be used for design purposes. This stage is commonly referred to as post-processing. The results of the analysis is tabulated and relevant plots, graphs etc. are generated.

3.3.2.1.1 Failure criteria

Perhaps the most important aspect of finite element analysis is the selection of a suitable failure criteria by which the stability of the rock mass is evaluated. The failure conditions of the rock mass must be accurately formulated by mathematical functions for a reliable analysis. The two most common failure criteria used in rock mechanics analyses are :

1. *Mohr-Coulomb failure criteria*
2. *Drucker-Prager failure criteria*

Mohr-Coulomb failure criteria predicts that the rock mass will fail on a given plane within when a critical combination of shear and normal stresses act on that plane. The mathematical expression of the linear form of Mohr-Coulomb

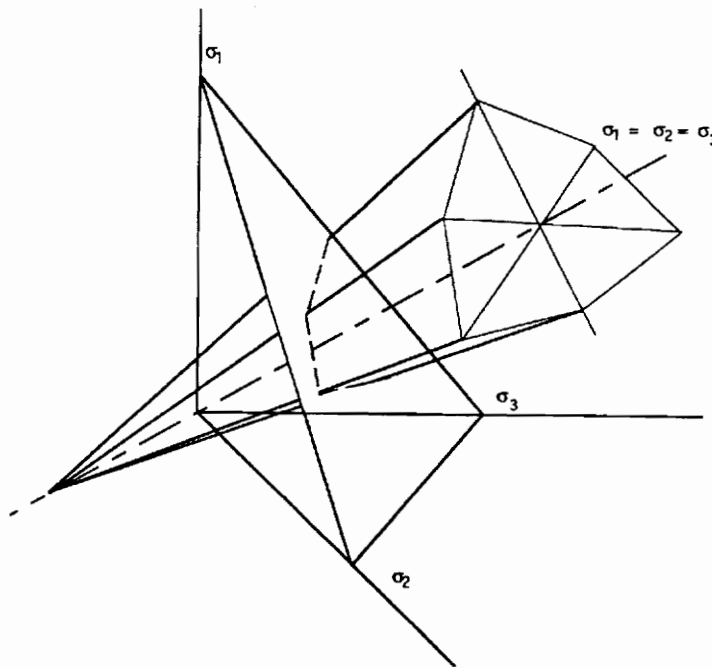


Figure 3.3.1 Mohr-Coulomb failure surface (Akram, 1993)

failure criteria is :

$$\tau = c + \sigma \tan(\phi) \quad (3.7)$$

where :

τ = magnitude of shear stress acting on the plane of failure

c = cohesion of rock mass

σ = magnitude of normal stress acting on the plane of failure

ϕ = internal angle of friction

An interesting point to note is that the Mohr-Coulomb failure criteria does not take the intermediate principal stress into consideration. The failure surface in the principal stress plane is therefore an irregular hexagon as illustrated in Figure 3.3.1. Due to these corners on the failure surface the Mohr-Coulomb

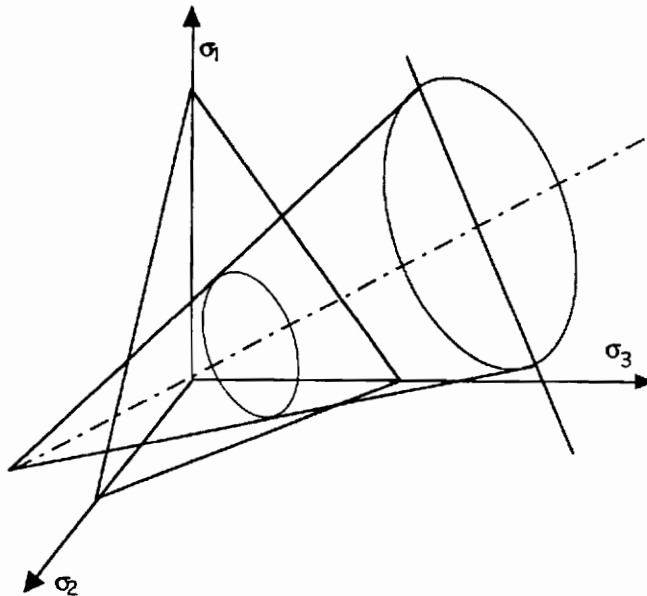


Figure 3.3.2 Drucker-Prager failure surface (Akram, 1993)

failure criteria is not particularly suited for use in numerical stress analyses of rock mass. Experimental evidence, on the other hand, suggests that the yielding behavior of rock under high confining stress is not linear (Tandanand and Thill, 1987).

The Drucker-Prager failure criteria, on the other hand, has a failure surface which is represented by a circular cone with its apex at the hydrostatic stress axis in the tension octant as illustrated in Figure 3.3.2. Drucker-Prager criteria uses the first invariant of the total stress tensor I_1 , and the second invariant of the deviatoric stress J_2 . Mathematically expressed these are :

$$I_1 = \sigma_1 + \sigma_2 + \sigma_3 \quad (3.8)$$

$$J_2 = \frac{1}{6} \left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right] \quad (3.9)$$

Then the mathematical formulation of the Drucker-Prager failure criteria becomes :

$$\sqrt{J_2} = A \cdot I_1 + B \quad (3.10)$$

where :

$A, B =$ constants derived from uniaxial compressive and tensile
strengths of rock mass

Methods of calculating the numerical values of the constants A and B for common yield conditions are summarized in Table 3.3.1. It is interesting to note that the Drucker-Prager criteria can be reduced to Mohr-Coulomb criteria by choosing suitable values for these constants.

Various applications of the Drucker-Prager failure criteria for numeric analysis of stability in mining environments are available in literature (Jones and Pariseau, 1990; McMahon and Pariseau, 1989; Pariseau, 1978).

Table 3.3.1 Methods of calculation of A & B for common yield conditions (Pariseau, 1968)

Yield condition	Tresca (constant)	Coulomb (linear)	Torre (parabola)	General (n-type)
n	1	1	2	n
A	0	$(C - T) / (C + T)$	$1/2(C - T)$	$\frac{(C/2)^n - (T/2)^n}{C/2 + T/2}$
B	$1/2 C$ or $1/2 T$	$CT / (C + T)$	$1/4CT$	$\frac{T(C/2)^n - C(T/2)^n}{C + T}$
$\sin \phi$	0	A	$A/2 \tau_m $	$A/n \tau_m ^{n-1}$

CHAPTER 4.

Model Description

4.1 UTAH2PC Finite Element Analysis Code

The analyses of the tailgate area and the various support alternatives were carried out using US Bureau of Mines finite element analysis code UTAH2PC (Pariseau et al., 1991). This is a two dimensional program originally developed in FORTRAN language and adapted for use on a personal computer, that can perform plane stress and plane strain and axially symmetric analyses. The program uses an elastic-plastic material model and has the capability to simulate sequential cuts and fills. The range of the purely elastic response is limited by material strength. An elastic-brittle analysis, referred to as caving analysis, is also available as an option. The maximum limit on the number of nodes that the program can handle is 2500. The condensed flowchart of UTAH2PC for solving problems in the elastic and elastic-plastic domains are illustrated in Figure 4.1.1 and Figure 4.1.2.

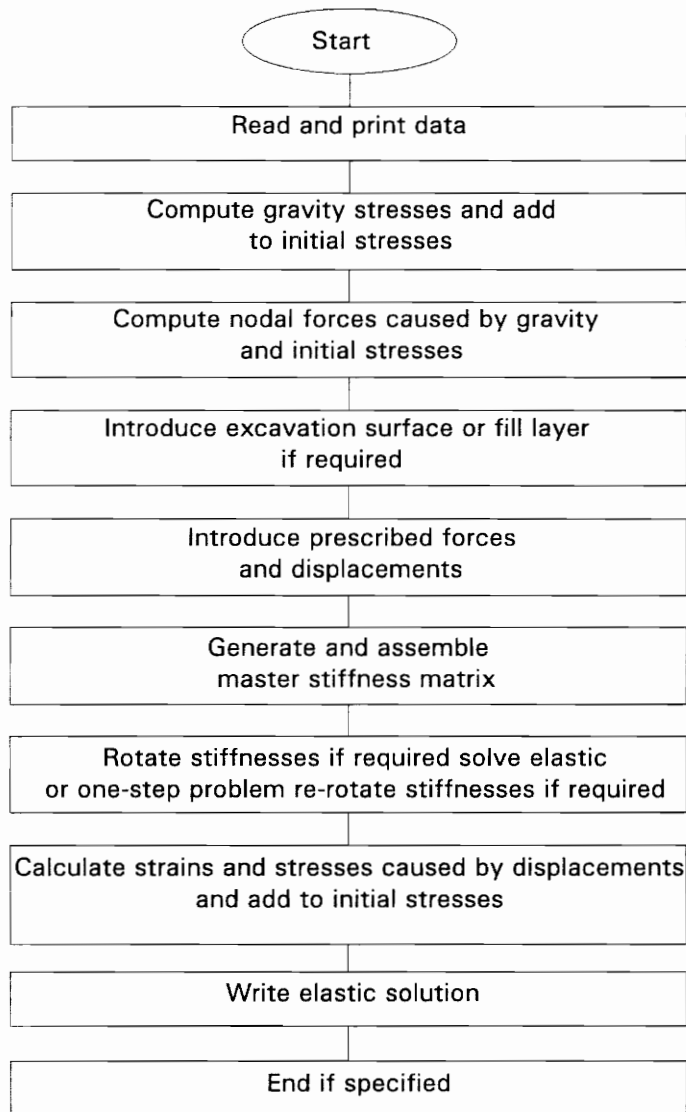


Figure 4.1.1 Elastic analysis program flow (Pariseau, 1978)

4.1.1 Input files for UTAH2PC

UTAH2PC is a file oriented code which requires nine different files to be prepared beforehand. These are :

1. *Master runstream file*

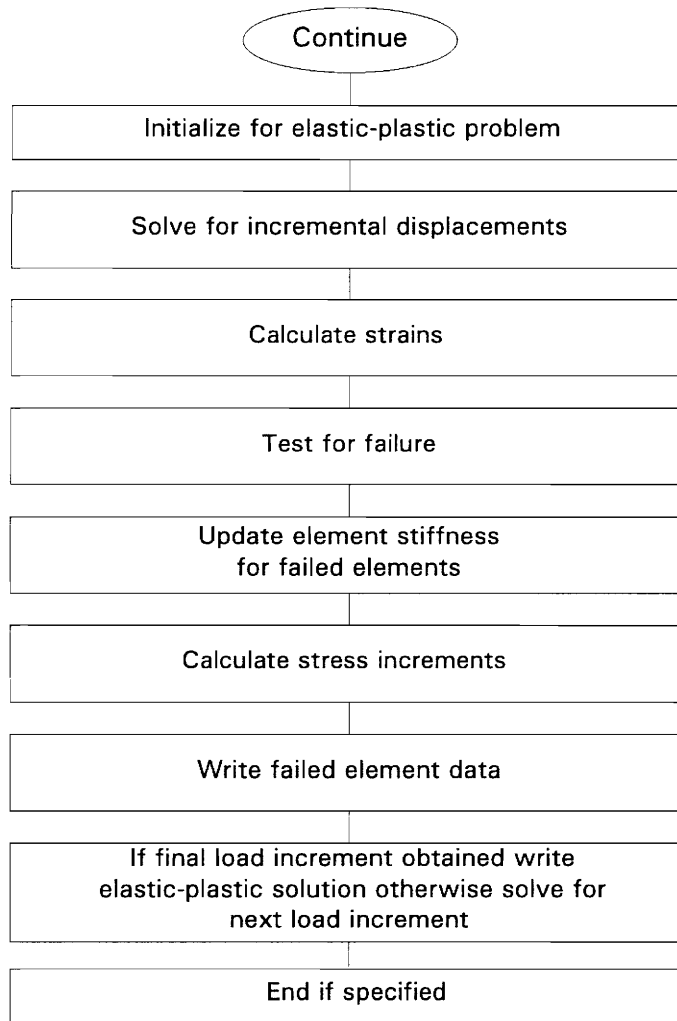


Figure 4.1.2 Elastic-plastic analysis program flow (Pariseau, 1978)

The master runstream file contains the file names and parameter specifications necessary for the analysis.

2. Material properties file

The material properties file contains records on the rock type name compressive strength, tensile strength, shear strength, Young's modulus, Shear modulus, specific gravity for each axes. The sequence is repeated for each

material type that is in the model. The program is capable of handling any number of materials.

3. Element connection file

The element connection file contains the element number and then the node numbers that make up that element along with element material type number. The nodes are numbered counter-clockwise and provision is made for triangular and/or quadrilateral elements.

4. Node coordinate file

The node coordinate file contains the node numbers and their respective coordinates in the x and y directions. Coordinates can be in any unit and provision is made for arbitrary scaling of units.

5. Boundary conditions file

Boundary conditions file contain forces and/or displacements that are specified around the mesh boundaries. Usually, the sides and bottom of a rectangular mesh are parallel to the coordinate axes, and are fixed in the normal direction but remain free to move parallel to the sides or the bottom. Forces may also be applied at the nodes.

6. Screen output file

This is simply a file where the results of the analyses are stored. It is restricted to 4 characters or less.

7. Cut or fill element file

This file contains a list of all the elements that are to be removed from or added to the mesh at runtime. The cut elements are automatically assigned air properties during program execution.

8. Initial stress file

UTAH2PC has the capability to analyse pre-stressed meshes. If this is the case the stress levels are introduced to the program via an initial stress file. Usually, the initial stresses for a new program pass are the output stresses from a previous run. An example is the gravity loading of a given problem for the first pass and the introduction of excavation induced stresses in the second pass.

9. Cut node file

This is a file that is generated by the program during program execution which simply contains a list of all the nodes that are removed from the mesh if a cut operation is specified.

4.1.2 Output files generated by UTAH2PC

Every program run of UTAH2PC generates four output files. These are :

1. Screen output file

The screen output file contains a list of the various input parameters that are supplied to the program. Basically it serves as a means of cross checking and fault finding if the run is not successful. The residuals at each iteration are also listed along with the element numbers that have changed into the plastic state.

2. Cartesian stresses and element safety factor file

This file contains the element number, element material type, coordinates of the element, the cartesian components of the stress and a numerical value flag for each element that has yielded.

3. Element principal stresses and strains file

This file contains the principal stresses and the calculated strains for each element. Along with this information the angle that the principal stress and the angle of the maximum strain with respect to the horizontal axis are also provided.

4. Node forces and displacements

Node numbers, coordinates, forces, calculated displacements in the x and y direction and the magnitude of the resultant displacement vector are listed in this file.

UTAH2PC also comes with a graphics post-processor program, UTAH2CAD written in AutoLISP programming language. The purpose of UTAH2CAD is to provide an interface between UTAH2PC and the popular computer graphics program AutoCAD, enabling the output files generated by UTAH2PC to be visually affirmed. Visualizations of the finite element mesh, displacements, principal stresses and safety factors for each element is made possible. However, UTAH2CAD suffers from not being able to read complex problems that involve more than 2500 nodes.

4.1.3 Failure criteria used by UTAH2PC

UTAH2PC has two options for governing the yielding behavior of materials used in the model. These are linear and quadratic. When the linear option is selected the program uses the Drucker-Prager failure criteria, which was described earlier. However, when the quadratic option is selected, which was claimed to be more suitable for modeling rock (Pariseau et al., 1991) the program modifies the Drucker-Prager criteria to simulate yielding behavior. This modification is carried out by introducing constants that are derived from the material properties. The safety factors for each element is calculated from :

Table 4.1.1 Description of procedure to calculate constants from material properties (Pariseau, 1994)

$$F = 1/2(p(6) - p(4) + p(5))$$

T = tensile strength

$$G = 1/2(p(4) - p(5) + p(6))$$

C = compressive strength

$$H = 1/2(p(5) - p(6) + p(4))$$

R = shear strength

$$L = 1/R_1(j)^2$$

$p_n = 1$ for linear , 2 for quadratic

$$M = 1/R_2(j)^2$$

$$N = 1/R_3(j)^2$$

$$U = [(C_1(j))^{p_n} - (T_1(j))^{p_n}] / p(1)$$

$$V = [(C_2(j))^{p_n} - (T_2(j))^{p_n}] / p(2)$$

$$W = [(C_3(j))^{p_n} - (T_3(j))^{p_n}] / p(3)$$

$$p(1) = T_1(j)(C_1(j))^{p_n} + C_1(j)T_1^{p_n}$$

$$p(2) = T_2(j)(C_2(j))^{p_n} + C_2(j)T_2^{p_n}$$

$$p(3) = T_3(j)(C_3(j))^{p_n} + C_3(j)T_3^{p_n}$$

$$p(4) = [C_1(j)T_1(j)] / p(1)$$

$$p(5) = [C_2(j)T_2(j)] / p(2)$$

$$p(6) = [C_3(j)T_3(j)] / p(3)$$

$$p_n = 2 / p_n$$

$$p(4) = p(4)^{p_n}$$

$$p(5) = p(5)^{p_n}$$

$$p(6) = p(6)^{p_n}$$

$$S.F. = \frac{\sqrt{J_{2_{maximum}}}}{\sqrt{J_{2_{actual}}}} \quad (4.1)$$

where :

$$\sqrt{J_{2_{maximum}}} = \left[1 - (U\sigma_{xx} + V\sigma_{yy} + W\sigma_{zz}) \right]^{1/n} \quad (4.2)$$

$$J_{2_{actual}} = F(\sigma_{yy} - \sigma_{zz})^2 + G(\sigma_{zz} - \sigma_{xx})^2 + H(\sigma_{xx} - \sigma_{yy})^2 \\ + L\tau_{yz}^2 + M\tau_{zx}^2 + N\tau_{xy}^2 \quad (4.3)$$

The material constants F, G, H, L, M, N, U, V and W are derived from the compressive , tensile and shear strength values of the materials used in the analysis. Equation 4.2 describes the anisotropic form of the first invariant of total stress, whereas, Equation 4.3 is the anisotropic extension of the second invariant of deviatoric stress. Table 4.1.1 summarizes the procedures used to calculate these constants from material property values.

4.2 Finite Element Model Description

It is a known fact that the depth of cover for most coal operations in the U.S. is relatively shallow, usually ranging from 200 ft. to 1000 ft. (Trent and Harrison, 1982). Under such shallow cover, geology induced failures are more likely to occur than stress related failures. However, as the existing seams are depleted, mining will continue in deeper deposits, which will in turn increase the likelihood of stress related failures. This research will only consider failures occurring due to the effects of high vertical and horizontal stresses, with a special emphasis on high horizontal stressfields.

4.2.1 Finite element mesh

The finite element mesh used in the analyses is given in Figure 4.2.1. This mesh is made up of 2468 elements and 2378 nodes. The size and type of the elements play a crucial role in the accuracy of the analysis. In general the smaller the element dimensions the higher the accuracy (Pariseau et al., 1991). However, there is a trade-off point as smaller dimensioned elements will result in a large number of elements to simulate the same region which in turn will require higher computational power and resources. The type of element used also has significance on the accuracy of the results. It is a well known fact that quadrilateral elements are inherently suited for modeling problems where bending is of prime importance.

UTAH2PC unfortunately does not have a preprocessor that will aid in mesh generation. Therefore, a computer program PREUTAH2, was written to generate the input files required for UTAH2PC such as, element connectivity file,

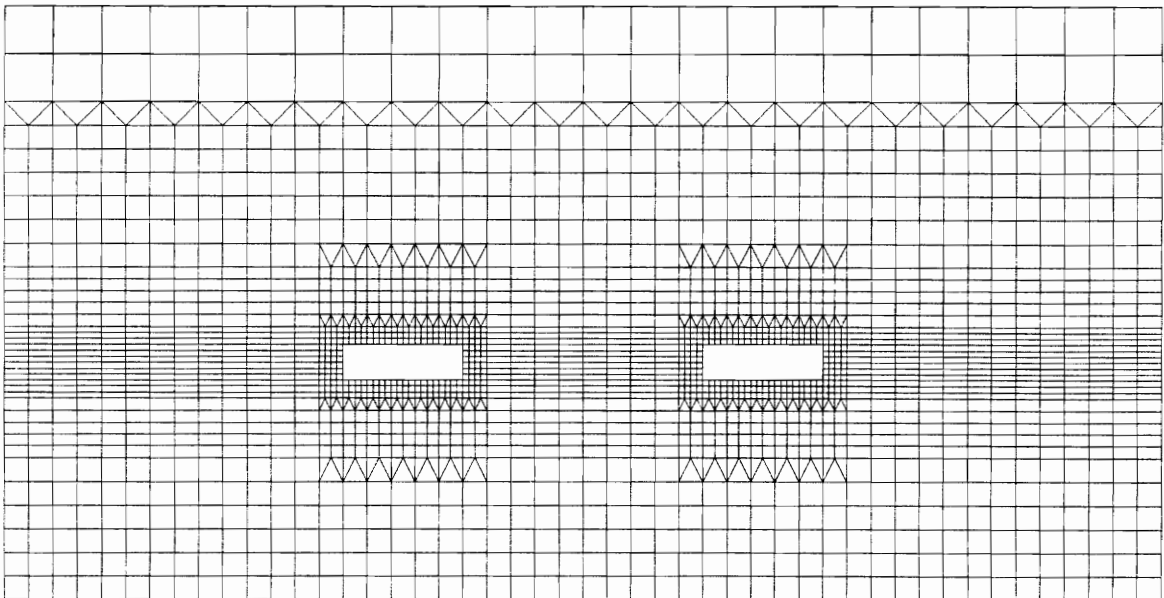


Figure 4.2.1 Finite element mesh used in the analyses

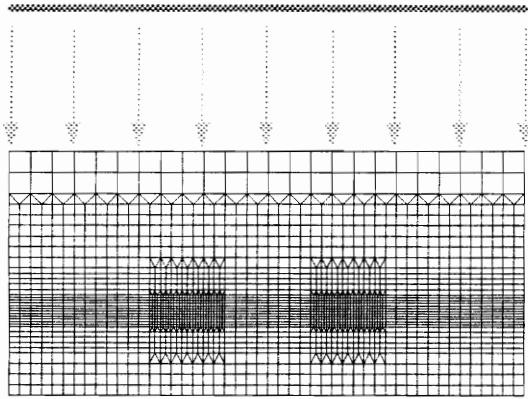
node coordinate file, boundary conditions file, cut/fill element file. This program was written in Microsoft QuickBasic and a complete listing is given in Appendix. As the primary region of interest is the immediate vicinity of the tailgate some means of decreasing the element dimensions around this critical region was necessary. This was accomplished using triangular elements which are inserted by the mesh generator program written. Using simple geometric relationships the element dimensions were halved each time a row of triangular elements were inserted in the mesh. This resulted in an element size of 1ft $\times$ 1ft in the immediate vicinity of the tailgate, gradually increasing to 8ft $\times$ 8ft at the top layers of the mesh. The overall dimensions of the simulated area was 92ft $\times$ 192ft. The entries are dimensioned to be 20ft in width and 6ft in height, which is the most common layout used in US longwall mines. The two panels were separated with a coal pillar 40ft in width. The ratios of the mesh size to excavation size and excavation size to smallest element size are in compliance with the mesh design guidelines recommended by Pariseau et al., (1991).

4.2.2 Boundary conditions

PREUTAH2 also generates the boundary conditions file required for the analysis. Almost any type of loading, including uniform to mathematically defined can be generated on the boundary nodes by modifying the program. The boundary conditions were set to simulate a depth of cover of approximately 1000 ft.

The effect of two loading conditions on the magnitude of the vertical stressfield at the roofline was investigated as a part of the model calibration process. First, forces of uniform magnitude were applied on the top nodes and second forces of exponentially decreasing magnitude over the mesh boundary as illustrated in Figure 4.2.2. The resultant values of vertical stress at the roof line level is given in Figure 4.2.3.

Uniform



Non-uniform

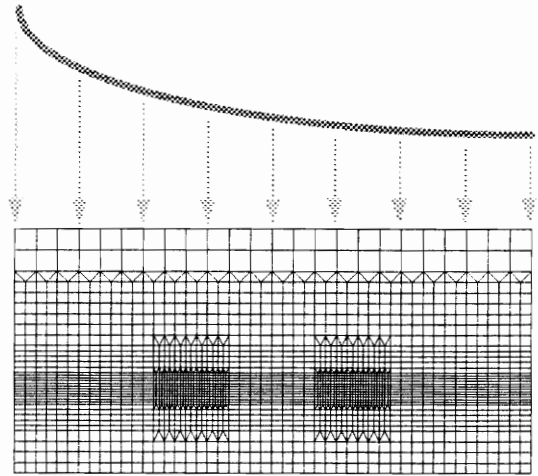


Figure 4.2.2 Loading types considered

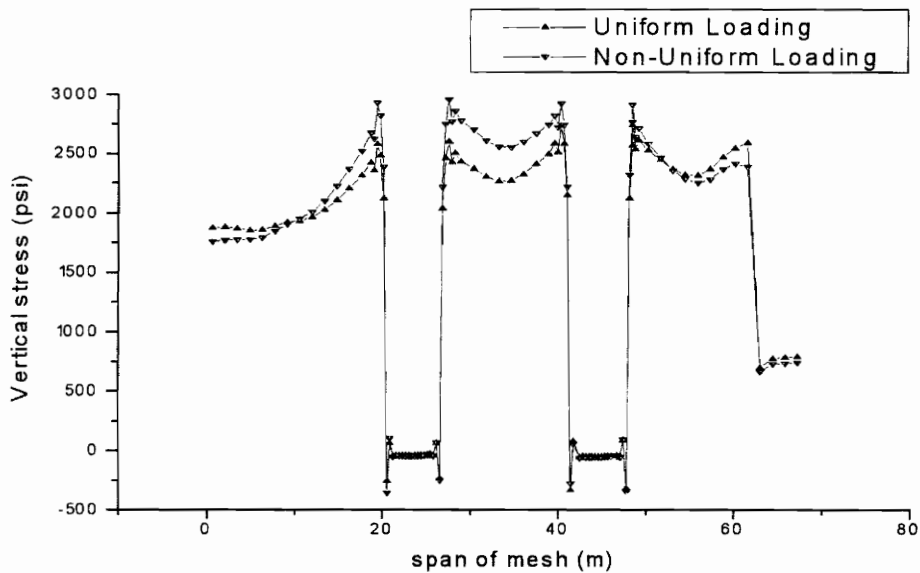


Figure 4.2.3 Change in vertical stress with respect to type of loading

Analysis of Figure 4.2.3 reveals that neither of the two loading conditions significantly effected the magnitude of the vertical stresses. Therefore in all

subsequent models loading of the mesh was accomplished by application of forces identical in magnitude and uniformly distributed on the top nodes. These were equivalent to a depth of 1000 ft.

The two bottom corner nodes were fixed in both dimensions, displacements on a single axis were permitted along the respective axes for the bottom and side nodes. The effects of high horizontal stresses were modeled by using high Poisson's ratios for the material types used in the mesh.

4.2.3 Material properties

A total of seven different materials have been incorporated into the model with an immediate roof of shale and a floor of sandstone. For modeling purposes the previous panel was assumed to be mined out, making the model unsymmetrical. A listing of these materials in their stratigraphic sequence, as used in the model along with their representative values are given in Table 4.2.1. A range of values are used for crib material to simulate cribs of different stiffness and load carrying capacity.

Table 4.2.1 The representative values of material properties

Material type	Thickness (m)	Compressive Strength (MPa)	Tensile Strength (MPa)	Shear Strength (MPa)	Young's Modulus (GPa)
Silstone	15.3	34.4	3.4	6.3	10.3
Shale	3.6	24.1	2.4	1.9	6.5
Coal	2.0	20.7	2.0	3.8	3.4
Gob	3.3	17.2	1.7	3.1	0.3
Sandstone	9.6	41.4	4.1	18.9	12.4
Crib	-	52.5-331	5.5-33.1	10-60.4	0.3-20.7
ASTM Steel	-				

4.2.4 Assumptions made

Finite element methods require certain assumptions to be made regarding the behavior of the material around the openings. These are :

1. *Elastic-plastic behavior of materials*
2. *No bed separation among the layers of strata*
3. *Use of conventional non-yielding pillars*
4. *Homogeneous non jointed roof and floor rock.*

The second assumption creates conditions that are approximately similar to a bolted roof. It was assumed that roof bolts were employed as primary means of support during the gateroad drivage and they prevented separation between layers of roof material.

The third assumption made is not a specific requirement for finite element methods.

The analyses was limited to stress related failures. Geological features that are observed underground, for example the discontinuities in the immediate vicinity of the opening, may also be the cause for roof failure. However, such physical features were not considered during the analyses.

CHAPTER 5.

Description of Experiments

5.1 Introduction

In order to analyze the effectiveness of various secondary tailgate support systems on the stability of the tailgate a model that could accurately simulate roof failure conditions as encountered in mines was developed. This model mimicked, with an acceptable degree of accuracy, the behavior of an actual roof failure. As indicated earlier, stability problems due to development of cutters in the immediate roof plays a significant role in the stability of the tailgate area for most northeast US coal mines. Therefore, the model was calibrated to simulate an actual tailgate opening suffering from cutter roof failure.

Sample finite element runs have indicated that the location of failure initiation is sensitive to the magnitude of horizontal stress. In low horizontal stress conditions failure was observed to initiate around the center of the opening due to exceeding the tensile strength of the shale roof. With increasing horizontal stress the location of failure initiation is observed to shift towards the

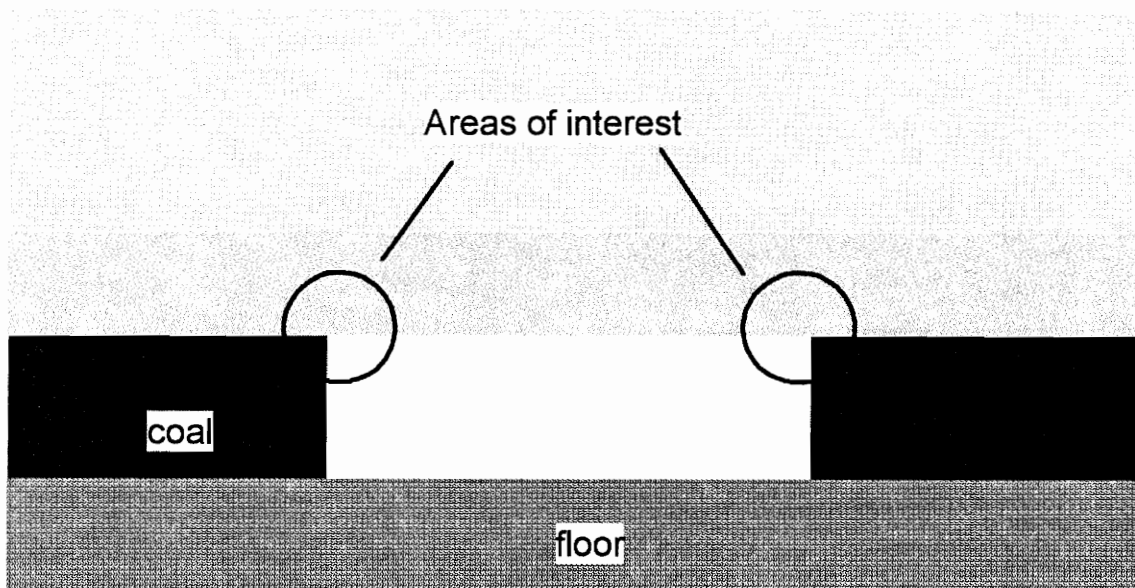


Figure 5.1.1 Areas of interest

corners of the opening along the rib line. The areas of interest in terms of opening stability is illustrated in Figure 5.1.1.

High horizontal stress conditions were simulated by using high Poisson's ratios. Another option that was considered for modeling high horizontal stresses was the application of horizontal forces on the side nodes of the mesh. However, care must be exercised in this method of simulating high horizontal stresses which is suitable only when the finite element mesh is made up of elements that have a symmetric layout within the mesh. The finite element mesh used in the analysis incorporates elements that simulate gob material only on one side of the mesh, thus altering symmetry. This layout of elements correspond to the last phase of mining operations when the adjacent panel has been previously mined and the tailgate is subjected to the highest stresses. Therefore, it is clear that applying horizontal forces to side nodes that contain gob elements would not constitute an accurate simulation of in-situ conditions. The initial state of the

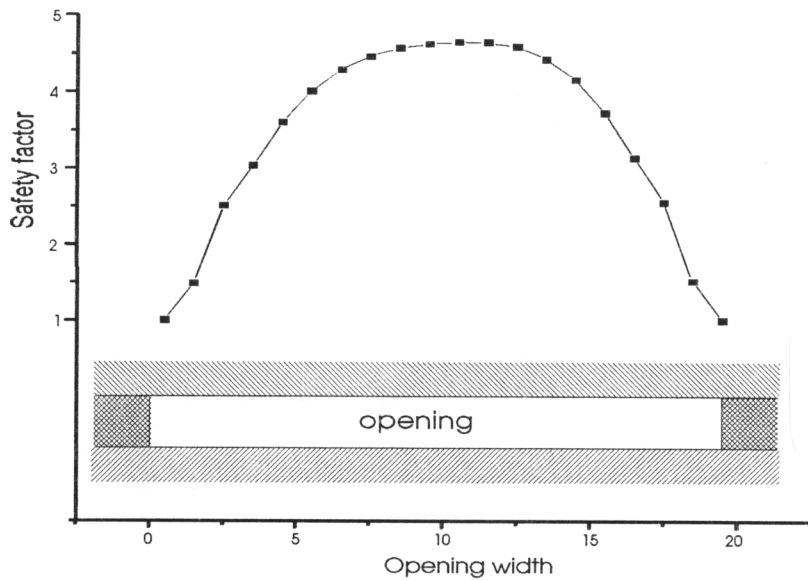


Figure 5.1.2 Initial state of the tailgate roof

calibrated model is illustrated in Figure 5.1.2 This state corresponds to the initiation of failure at the corner elements. Investigation of Figure 5.1.2 reveals that the corner elements have a factor of safety of unity which indicates that they are in the transition zone between elastic behavior and plastic behavior.

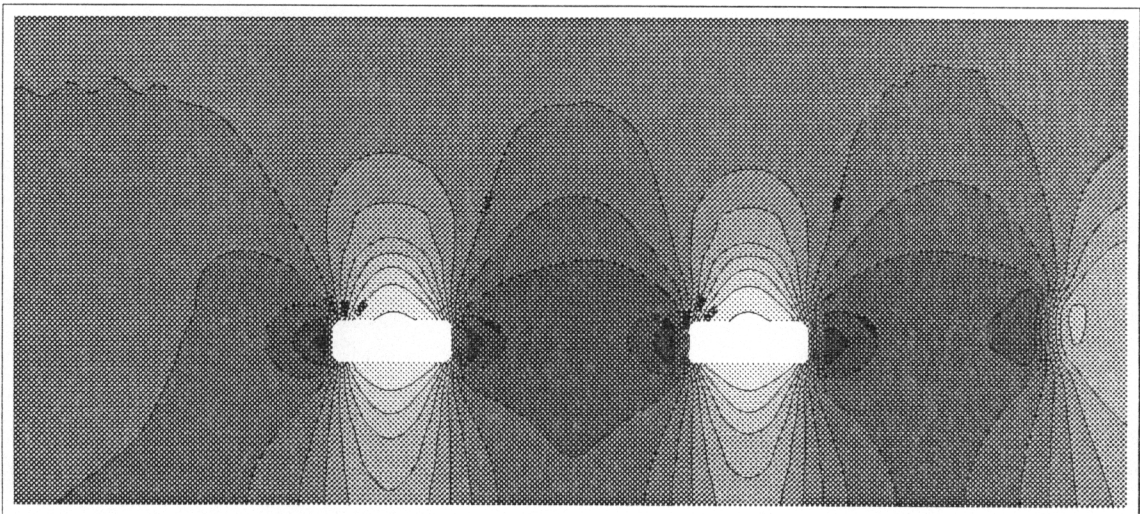


Figure 5.1.3 State of vertical stress before any support is applied

Figure 5.1.3 and Figure 5.1.4 illustrate the states of vertical and horizontal stresses throughout the modeled area after the tailgate entries are excavated but before any support is applied.

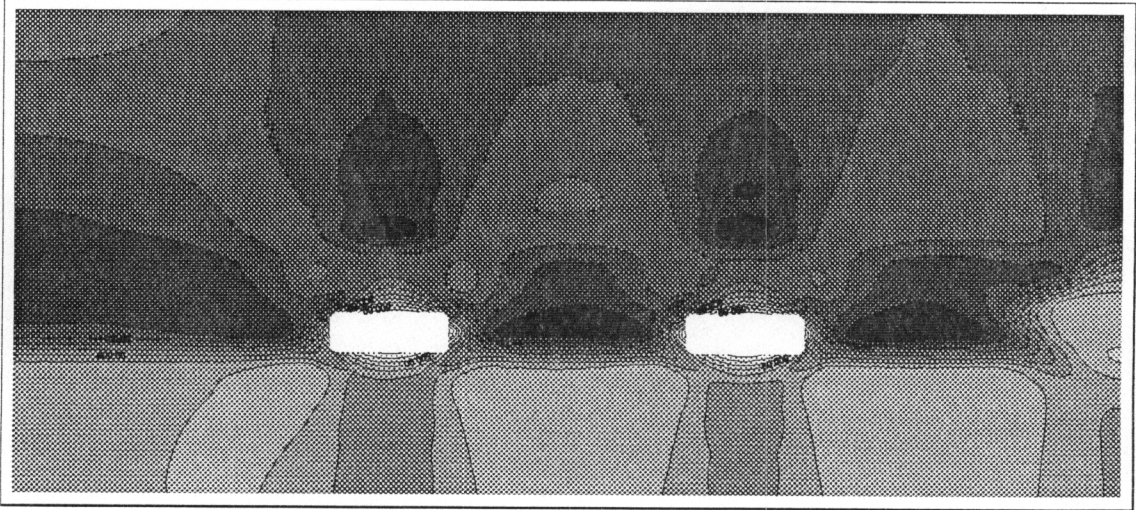


Figure 5.1.4 State of horizontal stress before any support is applied

The exact causes and the mechanism of cutter failure is a question of debate. However, there exists a high degree of correlation between the frequency of occurrence of cutter roof failures and the presence of horizontal stresses that exist in the vicinity. Examination of output files of the finite element run for a failing tailgate reveals that the magnitude and the orientation of the maximum shear stress plane closely matches the plane of initiation of cutter roof failure observed in real life conditions. This suggests that the primary mode of failure for a cutter roof is failure due to shear, as the magnitude of the maximum shear stress exceeds the shear strength value that is used for the rock material. Analysis of the state of stress for the corner element reveals that the orientation of the maximum shear stress plane is 82 degrees measured from the horizontal. The state of stress for the corner element is illustrated in Figure 5.1.5.

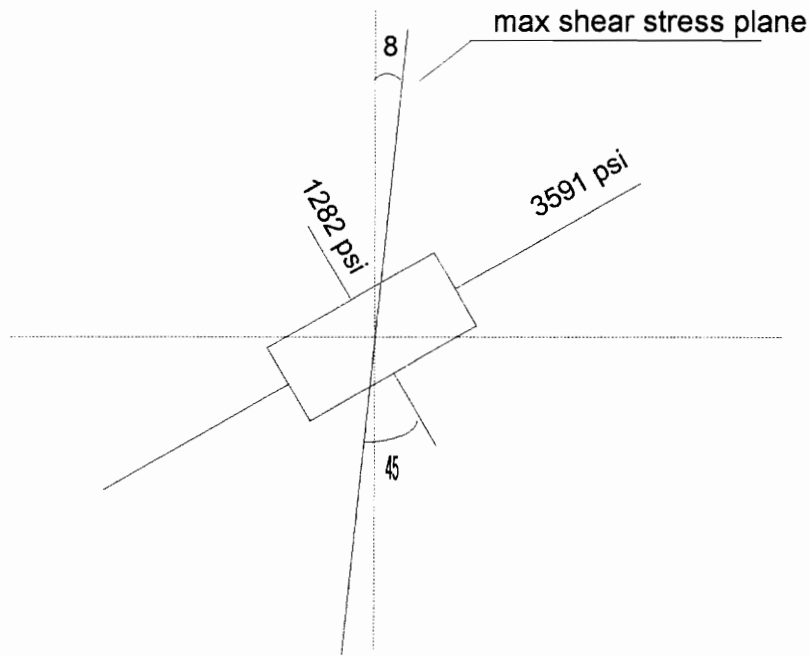


Figure 5.1.5 Orientation of maximum shear stress plane for failing corner model element

Tailgate support systems can be categorized into two major groups depending on the mechanics of the applied support to be passive and active supports. Each group have been analyzed using numerical modeling.

5.2 Passive Support

Passive support consists of installing support which does not forcefully apply pressure on the roof in order to prevent failure. As discussed earlier, the most common type of passive supports are wooden cribs. Wood cribs are installed and the roof is allowed to deform on to the cribs causing the stresses to be transferred into the floor through the cribs.

The most common arrangement of cribs in underground coal mines is by installing two rows of cribs centered within the opening (Peng and Biswas,

1994). In order to investigate the effects of crib positioning within the tailgate and crib stiffness on the stability of the opening a 6 by 8 factorial experiment was designed. Elements within the mesh were modified to simulate two rows of cribs each 3ft. wide and 6 ft. high placed within the tailgate. A total of eight different crib placement positions were investigated. For each position six different material properties, modified to reflect varying degrees of crib stiffness were analyzed.

Figure 5.2.1 illustrates the physical layout of the experiments. The distance between the cribs, shown as x in the figure was varied between 0 to 7ft, representing cribs adjacent to each other and cribs applied at corners respectively.

Installation of cribs within the tailgate causes a stress redistribution in the immediate vicinity of the opening. It is possible to determine the magnitude of this redistribution at any point within the modeled area merely by using the following formula for each node in the mesh:

$$\Delta\sigma_v = \sigma_{after} - \sigma_{before} \tag{5.1}$$

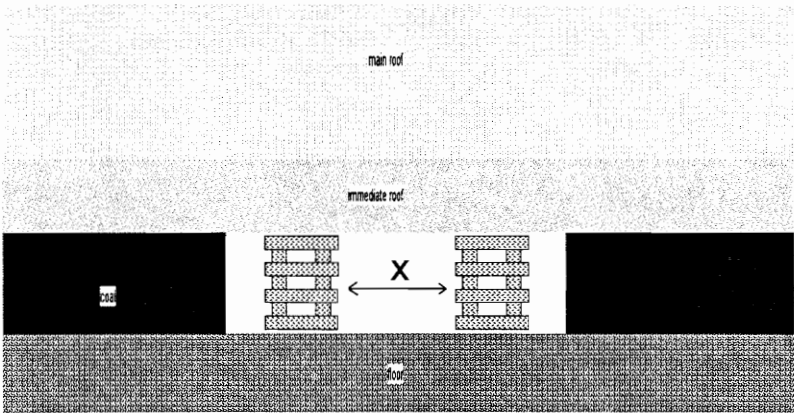


Figure 5.2.1 Physical layout of passive experiments

where :

$\Delta\sigma_v$ = change in the vertical stress

$\sigma_{after}, \sigma_{before}$ = values of vertical stress before and after the application of support

The location and magnitude of stress transfer can easily be obtained by using Eq. 5.1. Figure 5.2.2 illustrates the change in vertical stress values in the immediate vicinity of the opening before and after the cribs are installed at the corners of the tailgate.

Analysis of the results of the experiment reveals that the positioning of the

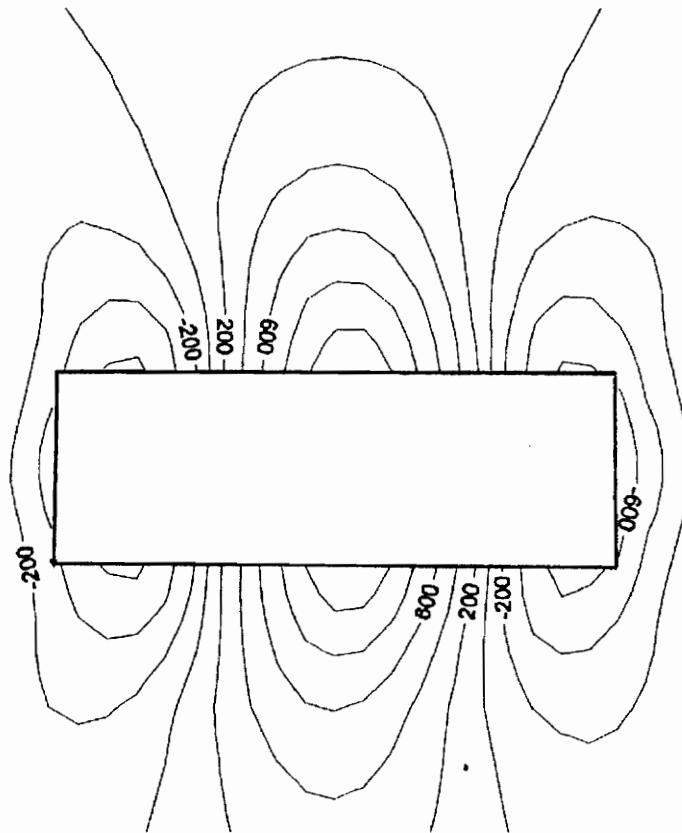


Figure 5.2.2 Change in vertical stress before and after crib placement at corners

cribs within the tailgate does have a significant impact on the stability of the opening. Figure 5.2.3 illustrates the sensitivity of the corner element to the placement position of the cribs. Analysis of the figure reveals that the maximum value for the safety factor of the corner element is obtained when the cribs are placed 2.5 ft. from the rib line, measured from the crib center. As the cribs are more closely spaced there is a decline in the safety factor of the corner element and ultimately when the cribs are placed adjacent to each other at the center of the opening, failure initiates at the corners.

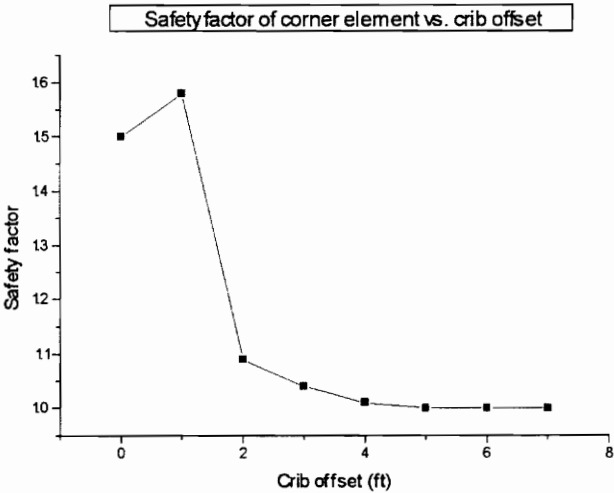


Figure 5.2.3 Safety factor of corner element vs. crib offset

The sensitivity of the corner element to crib material stiffness is illustrated in Figure 5.2.4. Analysis of the figure reveals that there is an approximate linear relationship between crib stiffness and corner element safety factor value. This is quite logical as the stiffness of the cribs increase the load carrying capacity also increases. The cribs are not forced into the roof and the roof is allowed to deform onto the cribs.

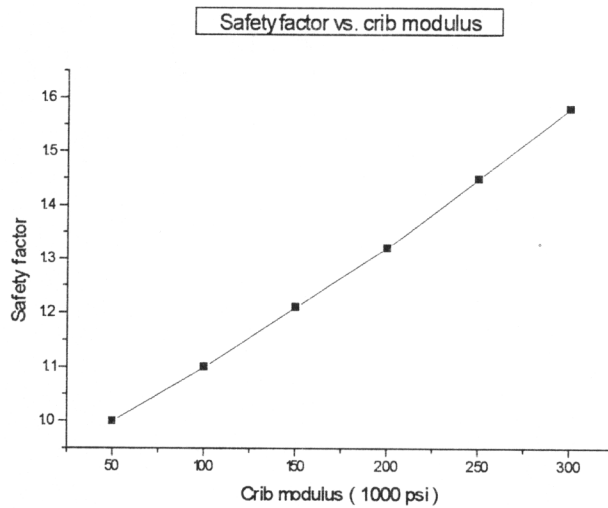


Figure 5.2.4 Safety factor of corner element vs. crib stiffness

As the placement position of the cribs are varied from the corners of the opening to the center there is a change in the stress field in the vicinity of the tailgate. This change is illustrated by comparing Figures 5.2.5 and 5.2.6. Figure 5.2.5 illustrates the vertical stress distribution contours around the entire area

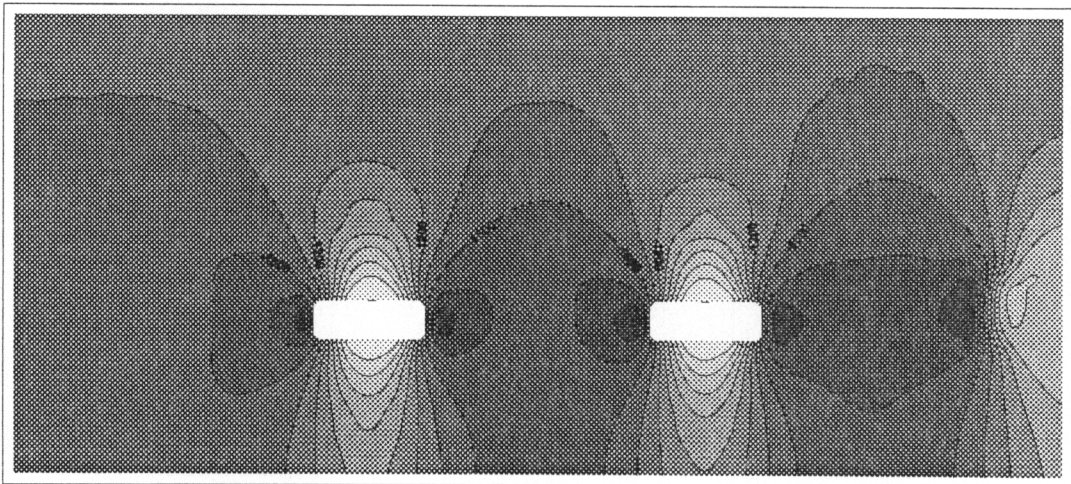


Figure 5.2.5 Vertical stress contours for cribs placed at corners

that is simulated for cribs that are placed at the corners of the opening, whereas, Figure 5.2.6 illustrates the same data but for cribs that are placed immediately next to each other located at the center of the opening.

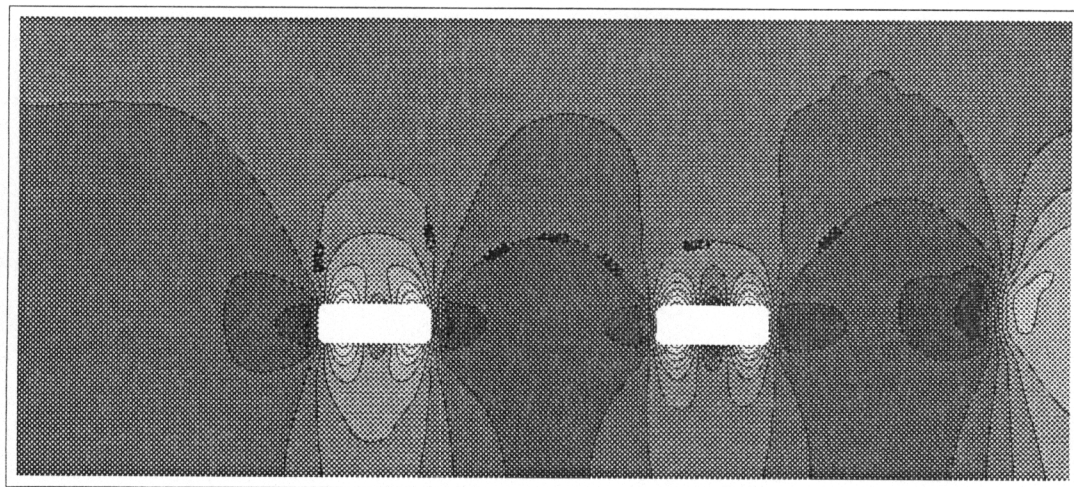


Figure 5.2.6 Vertical stress contours for cribs at the center

5.3 Active Support

5.3.1 Active support using cribs

An alternative means of supporting the roof is by actively exerting force against it. In practice this is accomplished by hand operated hydraulic jacks that are placed between crib chocks (Haycocks, 1993). Pressure is applied to the jacks usually by means of a hand pump. When the desired height or pressure is reached wedges are driven into the crib to maintain their positions.

A similar experimental procedure used for passive supports was designed to investigate the effectiveness of active supports. UTAH2PC allows external forces to be applied at selected nodes within the mesh. The pressure generated

by jacking the cribs was simulated by applying equivalent forces to the nodes that correspond to the crib-roof boundary. The sensitivity to the placement of cribs and various setting pressures were investigated by designing a 6 by 8 factorial experiment. The support was applied after the allowing the roof to deform and reach an equilibrium.

Similar to the passive support experiment 8 different crib placement positions were analyzed. For each one of these placement options 6 different setting pressures, ranging from 50 psi until the strength of the roof elements are exceeded were applied. The results for the two extreme cases of crib placements are illustrated in Figure 5.3.1 and Figure 5.3.2.

Analysis of the results of the experiment reveals that the safety factor of the corner element was not effected by the crib position as long as pressure was applied to the roof.

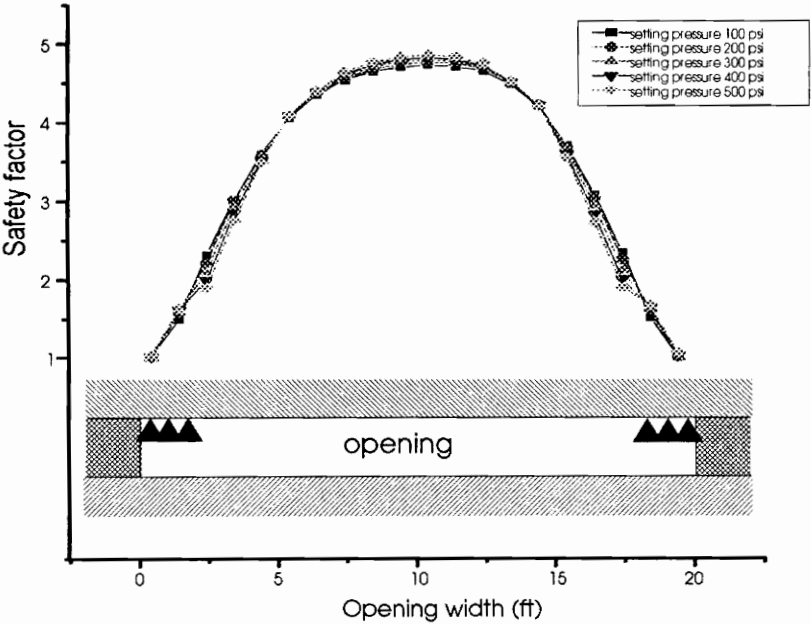


Figure 5.3.1 Active support applied at corners

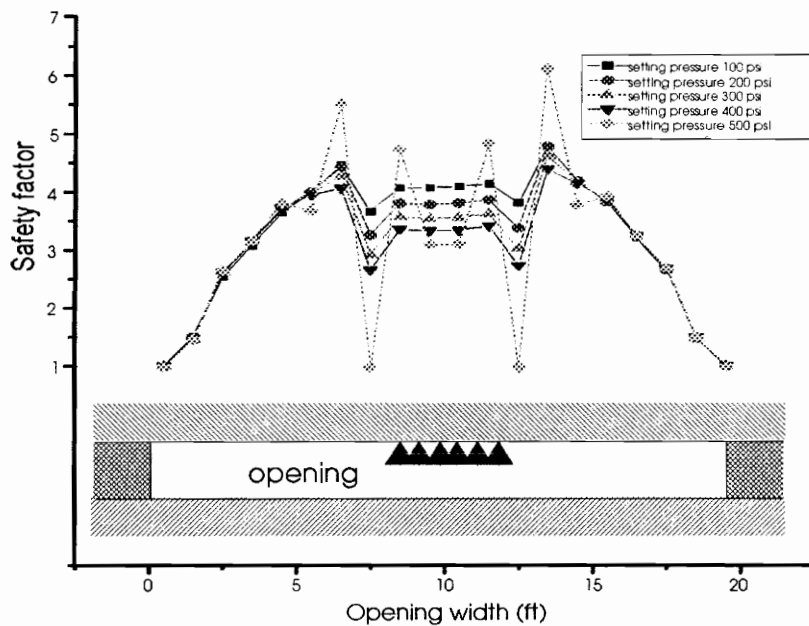


Figure 5.3.2 Active support applied at center

5.3.2 Active support using roof trusses

Another method of supporting the tailgate area is by the use of roof trusses. The use of roof trusses for tailgate support is gaining popularity due to the advantages it offers over the use of cribs (McCaffrey et al., 1994; Krupa and Long, 1994). Adequate stability can be achieved by use of such a support system without hindering ventilation and manway access at a lower cost than installing cribs.

In this method of roof support, rock bolts are installed 1 to 2 feet from the the ribline making an angle of approximately 45 degrees. Resin bolts or friction bolts are the most commonly used types. The ends of the bolts are tied together by means of a tensioning rod. The tension in the rod can be varied using a simple turnbuckle arrangement. A typical roof truss installation is illustrated in Figure 5.3.3.

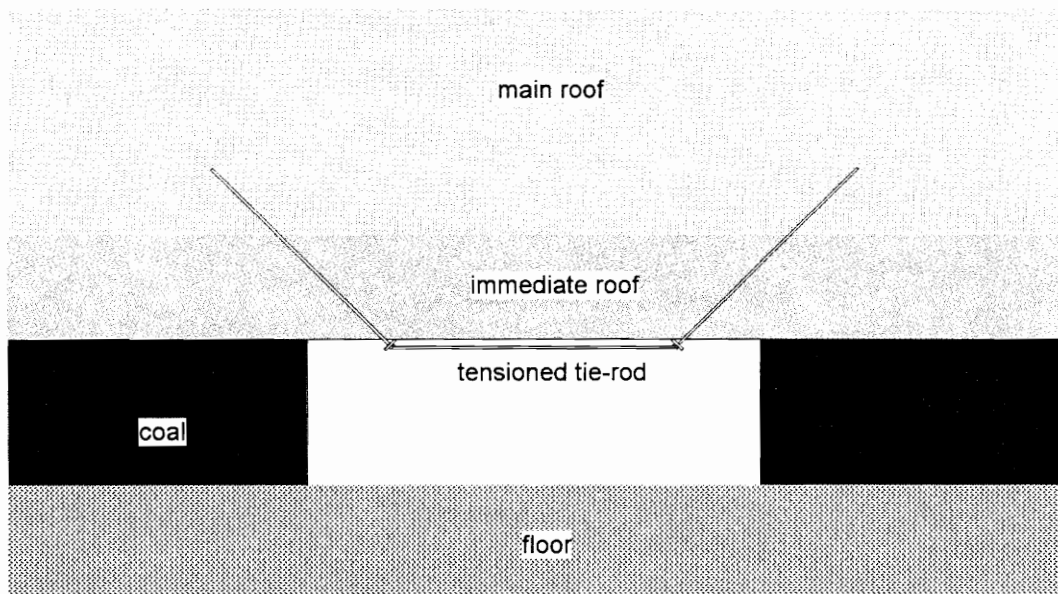


Figure 5.3.3 A typical truss installation

In order to numerically model a typical truss installation the finite element mesh was modified to include truss elements that were 2 inches in thickness and 6 ft. in length were drilled 1.5 ft. from the ribline at an angle of 45 degrees. Figure 5.3.3 illustrates a close up view of the modified mesh area.

The tension in the rod was simulated by applying forces to the relevant nodes within the mesh up to the yielding strength of steel. The safety factor of the corner element was found to be correlated with the magnitude of tension in the tension rod. The results of the experiment are summarized in Figure 5.3.5. Analysis of the figure reveals that the values of the safety factor for the corner elements are not the same, neither the curves. This is due to the fact that the model is unsymmetrical. The inclusion of the gob material left from the previously mined out panel causes is the cause of this unsymmetry.

The best results, in terms of the stability of the opening are obtained when the tension in the tension rod is set to 20,000 psi. Any further increase in

tension results in yielding of the truss material.

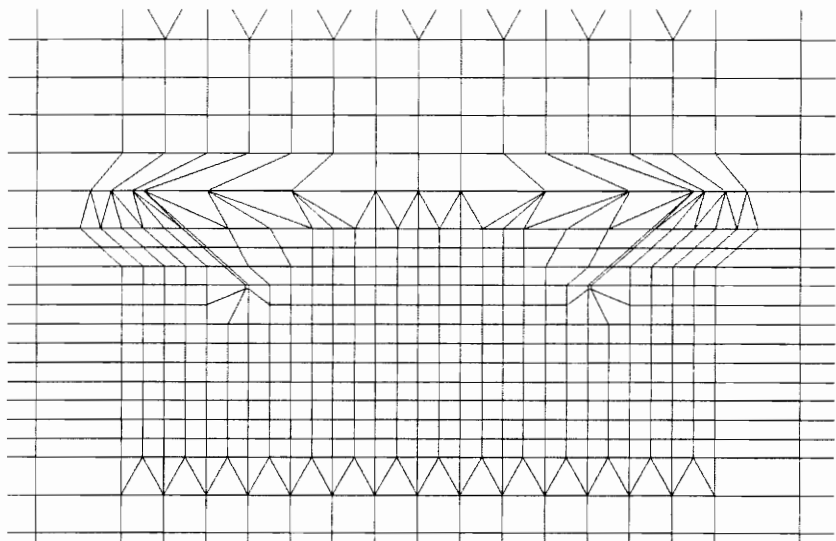


Figure 5.3.4 Close up view of the modified mesh

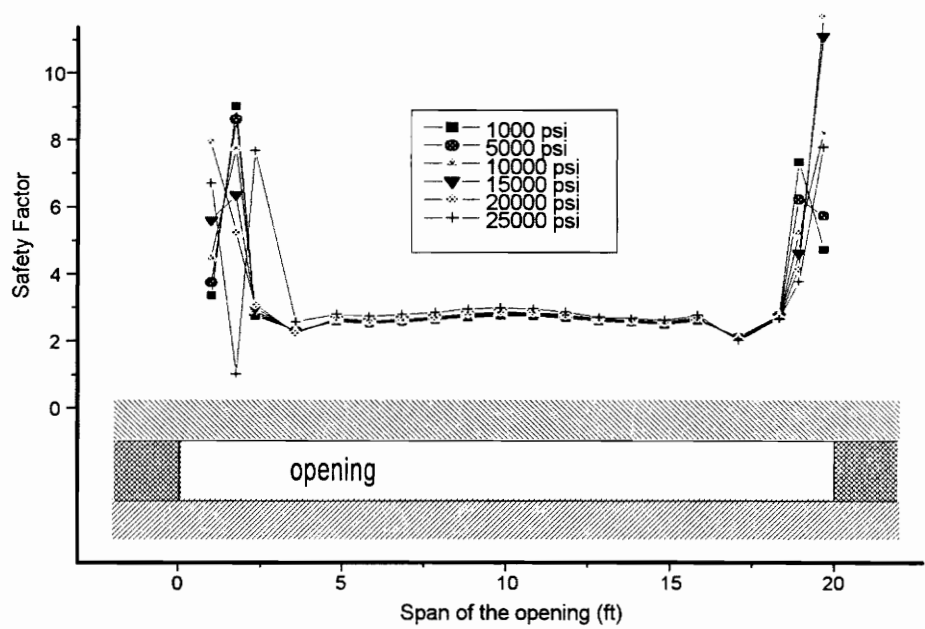


Figure 5.3.5 Safety factor vs. tension

CHAPTER 6.

Conclusions

A numerical model for a tailgate opening failing due to excessive stress was developed. The effectiveness of various forms of support that may be used to inhibit failure in this critical region were analyzed.

Based on the numerical modeling experiments performed for various secondary tailgate support systems the following conclusions are reached :

1. Optimal stability in a tailgate entry can only be achieved by matching the support to the loading conditions on a site specific basis.
2. The load capacity of the support should be based on the size of the potentially unstable zone above the roof.
3. The placement position of the cribs has a pronounced effect on the formation of cutter roof failure. Particularly in regions of high horizontal stress the cribs should be placed closer to the rib lines rather than the center of the opening. For the particular case investigated by numerical modeling optimum stability is achieved when the cribs are placed 1 ft. from the rib line, measured from the edge of the crib.

4. There is an approximately linear relationship between the stiffness of the passive cribs and the safety factor of the otherwise failing corner elements.

5. Excessive tailgate support capacity can force the immediate roof against the main roof with no significant benefits to opening stability.

6. Roof trusses can successfully be employed to prevent the formation of cutter roofs. The tensioning of the truss members has a pronounced effect on the safety factors of the otherwise failing elements.

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APPENDIX

Following is a listing of the code PREUTAH2 used to generate the mesh and the associated input files for UTAH2PC finite element analysis program.

```
DECLARE SUB cutelement ()
DECLARE SUB topnode (node, xc, yc)
DECLARE SUB sidenode (node, xc, yc)
DECLARE SUB cornernode (node, xc, yc)
DECLARE SUB bottomnode (node, xc, yc)
DECLARE SUB boundary ()
DECLARE SUB writecor ()
DECLARE SUB writecon ()
DECLARE SUB printcor (elm, ul, ll, lr, ur, m)
DECLARE SUB printnode (node, xc, yc)
DIM node(50, 100)
CLS
PRINT " finite element mesh generator ": PRINT : PRINT : PRINT
PRINT " generating mesh .....": PRINT : PRINT : PRINT
OPEN "c:\aaa\xxx. $$$" FOR OUTPUT AS #1
OPEN "c:\aaa\xx1. $$$" FOR OUTPUT AS #3
xc = 0: yc = 0
FOR xc = 0 TO 16 STEP 8
xi = xi + 1: yi = 0
    FOR yc = 0 TO 192 STEP 8
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
NEXT xc
FOR xc = 20 TO 40 STEP 4
xi = xi + 1: yi = 0
    FOR yc = 0 TO 192 STEP 4
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
NEXT xc
```

```

FOR xc = 44 TO 52 STEP 2
xi = xi + 1: yi = 0
  FOR yc = 0 TO 52 STEP 4
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
  NEXT yc
  FOR yc = 54 TO 78 STEP 2
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
  NEXT yc
  FOR yc = 80 TO 112 STEP 4
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
  NEXT yc
  FOR yc = 114 TO 138 STEP 2
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
  NEXT yc
  FOR yc = 140 TO 192 STEP 4
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
  NEXT yc
NEXT xc
FOR xc = 54 TO 66
xi = xi + 1: yi = 0
  FOR yc = 0 TO 52 STEP 4
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
  NEXT yc
  FOR yc = 53 TO 79

```

```

        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
    FOR yc = 80 TO 112 STEP 4
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
    FOR yc = 113 TO 139
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
    FOR yc = 140 TO 192 STEP 4
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
NEXT xc
FOR xc = 68 TO 76 STEP 2
xi = xi + 1: yi = 0
    FOR yc = 0 TO 52 STEP 4
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
    FOR yc = 54 TO 78 STEP 2
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
    FOR yc = 80 TO 112 STEP 4
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node

```

```

NEXT yc
FOR yc = 114 TO 138 STEP 2
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
NEXT yc
FOR yc = 140 TO 192 STEP 4
    yi = yi + 1
    node = node + 1
    CALL printnode(node, xc, yc)
    node(xi, yi) = node
NEXT yc
NEXT xc
FOR xc = 80 TO 92 STEP 4
xi = xi + 1: yi = 0
    FOR yc = 0 TO 192 STEP 4
        yi = yi + 1
        node = node + 1
        CALL printnode(node, xc, yc)
        node(xi, yi) = node
    NEXT yc
NEXT xc
CLOSE #1
CLOSE #3

OPEN "c:\aaa\yyy.***" FOR OUTPUT AS #2
elm = 0
m = 1
FOR xi = 1 TO 2
    FOR yi = 1 TO 24
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
NEXT xi
xi = 3
FOR yi = 1 TO 24
    elm = elm + 1
    ul = node(xi, yi)

```

```

    ll = node(xi + 1, 2 * yi - 1)
    lr = node(xi + 1, 2 * yi)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, 2 * yi)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi + 1, 2 * yi)
    ll = node(xi + 1, 2 * yi + 1)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR xi = 4 TO 8
    FOR yi = 1 TO 48
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
NEXT xi
xi = 9
    FOR yi = 1 TO 13
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
    FOR yi = 14 TO 20
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, 2 * yi - 14)
        lr = node(xi + 1, 2 * yi - 13)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)

```

```

    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, 2 * yi - 13)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi + 1, 2 * yi - 13)
    ll = node(xi + 1, 2 * yi - 12)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 21 TO 28
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, yi + 7)
    lr = node(xi + 1, yi + 8)
    ur = node(xi, yi + 1)
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 29 TO 35
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, 2 * yi - 22)
    lr = node(xi + 1, 2 * yi - 21)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, 2 * yi - 21)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi + 1, 2 * yi - 21)
    ll = node(xi + 1, 2 * yi - 20)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 36 TO 48
    elm = elm + 1
    ul = node(xi, yi)

```

```

        ll = node(xi + 1, yi + 14)
        lr = node(xi + 1, yi + 15)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
FOR xi = 10 TO 13
    FOR yi = 1 TO 62
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
NEXT xi
xi = 14
    FOR yi = 1 TO 13
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
    FOR yi = 14 TO 27
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, 2 * yi - 14)
        lr = node(xi + 1, 2 * yi - 13)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, 2 * yi - 13)
        lr = node(xi, yi + 1)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)
        elm = elm + 1
        ul = node(xi + 1, 2 * yi - 13)
        ll = node(xi + 1, 2 * yi - 12)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi

```

```

FOR yi = 28 TO 35
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, yi + 14)
    lr = node(xi + 1, yi + 15)
    ur = node(xi, yi + 1)
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 36 TO 49
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, 2 * yi - 22)
    lr = node(xi + 1, 2 * yi - 21)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, 2 * yi - 21)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi + 1, 2 * yi - 21)
    ll = node(xi + 1, 2 * yi - 20)
    lr = node(xi, yi + 1)
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 50 TO 62
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, yi + 28)
    lr = node(xi + 1, yi + 29)
    ur = node(xi, yi + 1)
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR xi = 15 TO 26
    FOR yi = 1 TO 90
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    
```

```

    NEXT yi
NEXT xi
xi = 27
    FOR yi = 1 TO 13
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
    FOR yi = 14 TO 40 STEP 2
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, (yi + 14) / 2)
        lr = node(xi, yi + 1)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)
        elm = elm + 1
        ul = node(xi + 1, (yi + 14) / 2)
        ll = node(xi + 1, ((yi + 14) / 2) + 1)
        lr = node(xi, yi + 1)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)
        elm = elm + 1
        ul = node(xi, yi + 1)
        ll = node(xi + 1, ((yi + 14) / 2) + 1)
        lr = node(xi, yi + 2)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
    FOR yi = 42 TO 49
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi - 14)
        lr = node(xi + 1, yi - 13)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
    FOR yi = 50 TO 76 STEP 2
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, (yi + 22) / 2)

```

```

    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi + 1, (yi + 22) / 2)
    ll = node(xi + 1, ((yi + 22) / 2) + 1)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi, yi + 1)
    ll = node(xi + 1, ((yi + 22) / 2) + 1)
    lr = node(xi, yi + 2)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 78 TO 90
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, yi - 28)
    lr = node(xi + 1, yi - 27)
    ur = node(xi, yi + 1)
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR xi = 28 TO 31
    FOR yi = 1 TO 62
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi)
        lr = node(xi + 1, yi + 1)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
NEXT xi
xi = 32
FOR yi = 1 TO 13
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, yi)
    lr = node(xi + 1, yi + 1)
    ur = node(xi, yi + 1)
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi

```

```

FOR yi = 14 TO 26 STEP 2
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, (yi + 14) / 2)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi + 1, (yi + 14) / 2)
    ll = node(xi + 1, ((yi + 14) / 2) + 1)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi, yi + 1)
    ll = node(xi + 1, ((yi + 14) / 2) + 1)
    lr = node(xi, yi + 2)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 28 TO 35
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, yi - 7)
    lr = node(xi + 1, yi - 6)
    ur = node(xi, yi + 1)
    CALL printcor(elm, ul, ll, lr, ur, m)
NEXT yi
FOR yi = 36 TO 48 STEP 2
    elm = elm + 1
    ul = node(xi, yi)
    ll = node(xi + 1, (yi + 22) / 2)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi + 1, (yi + 22) / 2)
    ll = node(xi + 1, ((yi + 22) / 2) + 1)
    lr = node(xi, yi + 1)
    ur = 0
    CALL printcor(elm, ul, ll, lr, ur, m)
    elm = elm + 1
    ul = node(xi, yi + 1)

```

```

        ll = node(xi + 1, ((yi + 22) / 2) + 1)
        lr = node(xi, yi + 2)
        ur = 0
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
    FOR yi = 50 TO 62
        elm = elm + 1
        ul = node(xi, yi)
        ll = node(xi + 1, yi - 14)
        lr = node(xi + 1, yi - 13)
        ur = node(xi, yi + 1)
        CALL printcor(elm, ul, ll, lr, ur, m)
    NEXT yi
    FOR xi = 33 TO 35
        FOR yi = 1 TO 48
            elm = elm + 1
            ul = node(xi, yi)
            ll = node(xi + 1, yi)
            lr = node(xi + 1, yi + 1)
            ur = node(xi, yi + 1)
            CALL printcor(elm, ul, ll, lr, ur, m)
        NEXT yi
    NEXT xi
    CLOSE #2
    CALL writecor
    CALL writecon
    CALL boundary
    CALL cutelement
    PRINT "the node coordinates are in the file   : cor.dat"
    PRINT "the element connections are in the file : cnt.dat"
    PRINT "the boundary conditions are in the file : boundary.dat"
    PRINT "the cut-elements are in the file      : cutelem.dat"
    BEEP

END

SUB bottomnode (node, xc, yc)
k = 1
dx = 0: dy = 0: fx = 0: fy = 0
    PRINT #2, USING "#####"; node; k;
    PRINT #2, USING "#####. "; dx; dy; fx; fy
END SUB

```

```

SUB boundary
OPEN "c:\aaa\xx1. $$$" FOR INPUT AS #1
OPEN "c:\aaa\boundary.dat" FOR OUTPUT AS #2
DO
  INPUT #1, node, xc, yc
  IF xc = 0 AND (yc >= 0 AND yc <= 192) THEN CALL topnode(node, xc, yc)
  IF ((xc < 92 AND xc > 0) AND yc = 0) OR ((xc < 92 AND xc > 0) AND yc =
192) THEN CALL sidenode(node, xc, yc)
  IF (xc = 92 AND yc = 0) OR (xc = 92 AND yc = 192) THEN CALL
cornernode(node, xc, yc)
  IF xc = 92 AND (yc > 0 AND yc < 192) THEN CALL bottomnode(node, xc,
yc)
  IF EOF(1) = -1 THEN EXIT DO
LOOP
CLOSE #1
CLOSE #2
END SUB

```

```

SUB cornernode (node, xc, yc)
k = 0
dx = 0: dy = 0: fx = 0: fy = 0
  PRINT #2, USING "#####"; node; k;
  PRINT #2, USING "#####. "; dx; dy; fx; fy
END SUB

```

```

SUB cutelement
OPEN "c:\aaa\cutelem.dat" FOR OUTPUT AS #1
PRINT #1, USING "#####"; -1
x = 1090: i = 0
FOR y = 1 TO 6
  FOR z = x TO x + 19
    i = i + 1
    IF i > 10 THEN
      PRINT #1,
        i = 1
    END IF
    PRINT #1, USING "#####"; z;
  NEXT z
  x = x + 90
NEXT y
x = 1126
FOR y = 1 TO 6
  FOR z = x TO x + 19

```

```

        i = i + 1
        IF i > 10 THEN
            PRINT #1,
            i = 1
        END IF
        PRINT #1, USING "#####"; z;
        NEXT z
        x = x + 90
NEXT y
CLOSE #1
END SUB

SUB printcor (elm, ul, ll, lr, ur, m)
    IF elm >= 1073 THEN m = 2
    IF elm >= 1613 THEN m = 3
    PRINT #2, USING "#####"; elm; ul; ll; lr; ur; m
END SUB

SUB printnode (node, xc, yc)
    PRINT #1, USING "#####"; node;
    PRINT #1, USING "#####."; xc * 12; yc * 12
    PRINT #3, node, xc, yc
END SUB

SUB sidenode (node, xc, yc)
    k = 2
    dx = 0: dy = 0: fx = 0: fy = 0
    PRINT #2, USING "#####"; node; k;
    PRINT #2, USING "#####."; dx; dy; fx; fy
END SUB

SUB topnode (node, xc, yc)
    k = 3
    dx = 0: dy = 0: fx = 1000: fy = 0
    PRINT #2, USING "#####"; node; k;
    PRINT #2, USING "#####."; dx; dy; fx; fy

END SUB

SUB writecon
    OPEN "c:\aaa\yyy.$$$" FOR INPUT AS #1
    OPEN "c:\aaa\cnt.dat" FOR OUTPUT AS #2
    DO

```

```

    FOR x = 1 TO 2
        LINE INPUT #1, a$
        PRINT #2, a$;
        IF EOF(1) = -1 THEN EXIT DO
    NEXT x
    PRINT #2,
    LOOP
CLOSE #1
CLOSE #2
END SUB

SUB writecor
OPEN "c:\aaa\xxx.$$$" FOR INPUT AS #5
OPEN "c:\aaa\cor.dat" FOR OUTPUT AS #6
DO
    FOR x = 1 TO 3
        LINE INPUT #5, C$
        PRINT #6, C$;
        IF EOF(5) = -1 THEN EXIT DO
    NEXT x
    PRINT #6,
    LOOP
CLOSE #5
CLOSE #6
END SUB

```

VITA

Erhan Hosca was born on March 13, 1969, in Ankara, Turkey. He graduated with a B.Sc. in Mining Engineering from Middle East Technical University, Ankara , Turkey in 1993.

A. ERHAN HOSCA