MECHANICS OF INTERSEAM FAILURE IN MULTIPLE HORIZON MINING

by

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CHAPTER I

INTRODUCTION

There are many mineable, contiguously placed coal seams in the Appalachian Coalfields. Historically, the selection of the mining sequence has been based primarily on seam ownership, availability and economics. However, the mining of any seam without consideration for the ground control problems that can occur may seriously affect subsequent operations in the coal seams above and below the mined area. Such practices are often reflected in the recovery, cost and safety hazards in the mining of subsequent seams.

The ground control mechanisms controlling multi-seam mining in the Appalachian Coalfields has been isolated into these four areas:

--- massive interseam failure
--- pillar load transfer
--- subsidence
--- arching

The objective in this study is to determine the factors controlling the massive interseam failure in the Appalachian Coalfields and to produce firm mining guidelines that will facilitate optimum seam extraction, safety and minimum mining costs (Haycocks and Karmis, 1981).

Interseam failure might occur in a multiple horizon mine as an alternative to the formation of a subsidence trough over an excavation. Highly inclined shear or shear tensile failures may develop which usually results in block movement of the ground into the lower
excavations. This phenomenon has been recorded by several observers (Barrientos and Parker, 1974; Holland, 1951; and Stemple, 1956).

In order to understand the mechanisms involved in the interseam failure of a contiguous seam operation, the following parameters will be considered in this research:

i) The effect of horizontal to vertical load ratio.

ii) The effect of relative stiffness of the layers.

iii) The effect of changing other rock properties.

iv) The effect of proximity of other seams.
CHAPTER II

LITERATURE REVIEW

2.1 General

Multi-seam mining problems have existed for a very long time. British mines have been operating in multi-seam conditions for a considerable period of time (Finlay, et al, 1933; Dunham, et al, 1978; and Spedding, 1976). Quoting Dunham and Stace (1978):

"In the British Coalfields, it is becoming increasingly uncommon to mine in virgin areas and it has been estimated that every working face will be affected by interaction effects from previous workings at least once during its life."

Multi-seam mining problems are not limited to Britain but are experienced worldwide, and the United States is experiencing these problems in many regions.

Holland (1951) lists the following factors below as being of paramount importance in the solution of problems of the stability of lower or upper beds after the main bed is mined:

i) Mining methods in extracting the beds.

ii) Extraction ratio and uniformity of beds concerned.

iii) The intervals between the beds under consideration.

iv) Thickness of beds.

v) Nature of strata between beds.

Further, Holland (1951) observed that in 38 case studies undertaken in the Appalachian Coal region, about 75% of them had a shear or shear tensile failure after an underlying bed had been mined. Figure 1 is an
Figure 1. Massive Interseam Failure Caused by Removal of the Underlying Pillars (after Stemple, 1956)
example of massive interseam failure caused by removal of underlying pillars. Some of these case studies had water and air entering the cracks allowing the shales in the immediate roof of the underlying bed to weaken. Other problems which may be encountered when an underlying bed is mined first are as shown in Figures 2, 3 and 4. To explain the phenomena, Peng and Chandra (1980) proposed the idea of a dome shape to describe the zone of caving—the zone in which failure by shear and block movement occurs. The maximum height of this dome was empirically assumed to be 30 to 50 times the mining height.

In observations carried out on the multi-seam mines of the Pocahontas Coalfields in Southern West Virginia, which applies to coalfields in Virginia and Kentucky, Lazer (1965) concluded that, when the lower seam is mined first prior to mining the upper seam, interaction between beds is almost non existent if the interval between workings are greater than 50 feet. He further observed that, within the normal limits, the thickness of the lower mined out seam has no effect on the upper seam mining and the interval between seams may be as little as 50 feet. The paper by Hasler (1951) also supports these observations.

The complexity of interaction due to seam interval is manifested in the paper by Dunham and Stace (1978) where they noted that in British Coalfields maximum seam interval beyond which interactive effects are no longer felt is quoted from 210 feet to 750 feet. Dunham and Stace (1978) also emphasize that the influence of seam interval in multi-seam for any given situation is very much complicated by a variety of other considerations.
Figure 2. Shear Failure in Redstone Seam Over Robbed Lower Seam (after Holland, 1951)
Figure 3. Bending, Shearing and Readjustment of Level Caused by Subsidence in the Lower Seam (after Holland, 1951)
Figure 4. Bending in Humph Coal After Mining a Lower Seam
(after Holland, 1951)
Figure 5. Model of Strata Disturbance Above a Fully Extracted Underlying Panel (after Peng and Chandra, 1980)
Hasler (1951) studied the multiple seams of the Central Pennsyl-
vania bituminous coal region, especially in the Cambria County. He
observed that in the cases of the lower of the two or more beds of coal
in vertically adjacent areas which was mined before the extraction of
the upper bed, the problems encountered are:

i) Unavoidable damage to the upper bed—-and the causes
   of these damage are subsidence, fracture, and parting
   of the overlying strata and beds of coal within
   varying ranges.

ii) Damage to overlying bed due to large and extensive
    pillars of coal left in the lower worked out bed.
    The effects are listed as bumps in the floor,
    crushing of the coal in the upper bed, undue
    fractures in the coal.

The discussion above has been centered on coal, but it should be
emphasized that multiple seam mining is not restricted to coal but to
metal mining and other minerals as well. Hedley (1974) in the analysis
of multi-seam mining at Elliot Lake uranium mines, lists the causes of
the failure in the interseams as, buckling due to high horizontal
forces, tensile failure at the center of the roof in the lower bed, and
the shear or tensile failure in the interseam (usually occurring at the
edge of the pillar). According to the observation made at these mines,
the latter cause was the major mechanism of failure in the Elliot Lake
uranium mines. A graph of parting thickness (interseam thickness)
against stope span (width of opening) was plotted from observations of
Elliot Lake uranium mines as in Figure 6. From the graph, Hedley
Figure 6. Relationship Between Interseam Thickness and Width of Opening (after Hedley, 1978)
Elliot Lake uranium mines as in Figure 6. From the graph, Hedley (1954) postulates the formula below as a rule of thumb in the designing of the multiple seam mines at Elliot Lake.

\[ L \leq 5t_2 \]  \hspace{1cm} (1)

where \( L \) = maximum width of opening, ft.
\( t_2 \) = interseam interval, ft.

In reviewing most of the literature on multi-seam mining, it immediately becomes obvious that the character of the beds between the coal seams is very important in any evaluation of the problems attached to them. Figure 7 illustrates the importance of the character of the innerburden to the stability of the other seams. There are four sequences in which multi-seam mines can be worked and they are (King, et al, 1972):

i) The lower seam is mined out first prior to mining the superincumbent seams.

ii) Simultaneous mining of upper and lower seams.

iii) The upper seam is mined prior to mining the underlying seam.

iv) Combinations of the above (Hasler, 1951; Stemple, 1956; and Lazer, 1965).

The sequence of mining to be considered in this study is the situation where the lower seam is mined prior to the extraction in the upper seam.

Many of the published works available on multiple seam mining, depending on which part of the world the investigation was made, con-


Figure 7. Relationship of Innerburden Thickness and % Sandstone for Room and Pillar System in the Appalachian Coal Region (after Haycocks and Karmis, 1981)
tain some recommendations in extraction of these contiguous beds. The author will summarize these suggestions of mining in an upper bed after the extraction of the lower seam as below:

i) Extraction in the lower seam should be complete with no pillars or large remnants left in place (Holland, 1971).

ii) Pillar lines should be kept long and straight to reduce shearing of the overlying strata at the edges of the barrier pillar (Holland, 1951).

iii) As in (i) above if pillars are to be kept in the lower seam, it is advisable to columnize them, especially for developments and long-life entries. It is also recommended that whatever remnant pillars are left should be uniformly distributed (Peng and Chandra, 1980).

Reporting of observations in a mine is very important in studies of this kind; bearing in mind the above statement, King, et al, (1972) in their survey of the interaction of several workings on each other in the Midland Coalfields in Great Britain listed these conditions as the result of the phenomenon:

i) increased closure of mine openings due to the interaction producing an increase in the intensity of the force field surrounding the opening;

ii) deterioration in the natural strength of the strata surrounding the opening owing to the effects of the
increased rock pressures and/or lateral strain;

iii) increased roof control difficulties on working faces and roadway closure arising from interaction of subsidence of a lower working face; this is referred to as "live" interaction and is common in multi-seam workings.

Stemple (1956) in his report of the observations in coalfields of Eastern United States, stated that the damage that can be expected in the upper seam because of previous excavation in a lower one depends on the thickness of the underlying bed, the method of mining and the thickness of the innerburden. Shearing of the upper coal bed was also observed and is thought to occasionally facilitate the extraction of that seam (Figure 8). The recommendation for the limitation of excessive damage in the upper bed in this case was the complete mining of the lower seam prior to the extraction of the overlying bed.

The most common phenomena observed in Stemple's study were:

-- cracking or horizontal parting in the overlying strata.
-- vertical displacement of the upper seam which usually translates into separation of the bed.
-- roof falls, floor heaves and pillar crushing or squeezing.

Maximum disturbance in the upper seam was generally observed when isolated, groups of pillars, or solid coal (barrier pillars, chain pillars) were left in the lower seam. This caused the upper seam to shear along the coal line in the lower seam.
Figure 8. Damage in Upper Coal Seams (after Stemple, 1956)
less disturbance is caused when the lower seam is thin rather than thick. The nature of the strata between seams has an influence on failure too.

Further observation in the longwall method of mining in contiguous seams shows the shearing action in the gob region. If longwall mining system is used, an isolated pillar left in the lower seam acts as an unsupported cantilever beam and thus can bend 100 to 300 feet before fracturing (Figure 9). The severity of the disturbance, however, depends on the nature and thickness of strata in between the beds and thickness of the coal (Engineers International, 1981).

Various factors are involved in the failure mechanism of an underground excavation and more so in practical experience. Most of these factors which have been mentioned earlier, however, complicate the analysis of examples of actual cases in the field. Engineers International, Inc. (1981) collected some data from mine visits and literature review, mostly from the Appalachia, and presented them in a graph (Figure 10) comparing mining of both top and bottom seams. Figure 10 shows the relationship of extraction ratio to the interseam distance (parting thickness). Data points from each mining operation were connected vertically to facilitate interpretation.

Interseam distance is basically the underlying parameter in any study of multi-seam mining for the causes of structural problems. The extraction ratio decreases as the height of the interseam becomes smaller or if the nature of the parting is weak (Engineers International, 1981). Fracturing of the upper seam and bumps created by pillars
Figure 9. Shearing Action in the Gob Region of a Longwall Panel
(after Engineers International, 1981)
Figure 10. Effect of Parting Thickness on Extraction When Mining Above a Worked-Out Seam (after Engineers International, 1981)
left in the lower seam—due to high stresses induced and shearing of the intact and the bedding interface—are the major causes of lower extraction ratios in the upper seam.

There are many examples in which the shearing mechanism of the innerburden affects the successful mining of contiguous beds. As has been mentioned earlier in the observations made of the problems encountered in multiple-seam mining, roof falls are one of the major difficulties encountered in the excavation process. Cox (1974) observed that most of the major roof failures are associated with structural fracture zones, massive shale beds of lenticular structure or thinly laminated sand-shales. Apart from the obvious weakness presented by jointing and faulting, massive shales and laminated sandy shales create a low shear strength roof structure suggesting mostly shearing mechanism in its failure. The conclusions in this paper suggest that shear failures of the rock are possible if short bolts are used in roofs of low shear strength as shown in Figure 11. The potential of shear failure along vertical fracture planes (joints, etc.) are also stated (Cox, 1974). The stages of failure in the roof showing the shearing action when a moderate and severe transverse jointing are encountered as in Figures 12 and 13.

An example of positive use of the shearing mechanism of failure in mining has been in the excavation of ore using the block caving technique. This method of mining entails the induction of caving to facilitate the recovery of the ore. The caving is induced by undercutting the ore and thus allowing shearing from both ends of the opening. To limit
Figure 11. Typical Failure of a Bolted Mine Roof
   (after Cox, 1974)
Figure 12. Stages of Roof Failure Showing the Shearing Action When Moderate Transverse Jointing are Encountered (after Adler, 1973)
Figure 13. Stages of Roof Failure Showing the Shearing Action When Severe Transverse Jointing are Encountered (after Adler, 1973)
greater dilution the deposit must be massive, it must have strong walls from which ore can part easily. Planes of weakness in the ore can help greatly in this method. The size of the undercut depends on factors such as the planes of weakness in the ore, the height of the ore available for extraction and the output of the mine.

2.2 Geological Factors

As usually happens in most ground control problems, geology of the area under consideration plays a major role in its analysis. The geological factors very much dictate the extent of failure in the interseam of any multiple horizon mine.

Massive shear failures have been postulated to be the mechanism of failure in the interseam of two or more vertically adjacent beds when the lower one is mined first (Holland, 1951; Stemple, 1956). In the study of taphrogenesis, a term formulated by KRENKEL to define the formation of Grabens\(^1\), it was found that the massive shear failures and tensile failures are the most predominant means of formation of the world's rifts.

The geological history of most of the world's rifts is still clouded in controversy; there is no agreement over how the rifts evolved, nevertheless most of the parties involved in the controversy acknowledge the fact that somewhere along the history of the formation of the rifts, massive shear and tensile failure occurred to cause the rifts.

\[^1\text{Grabens in German means ditch and is used frequently in geology to mean rifts.}\]
Studying the rifting process really gives an insight into the phenomenon of massive shear failures and their relation to interseam failure. Burek (1981), in writing about warping in the rifts of Arabia, North East Africa and the continents which surround the North Atlantic rifts, noted that continental warping and shearing are due to horizontal, compressional stresses caused by the injection and spreading of mantle derived—new oceanic crust (mantle diapirism, ocean-floor spreading). Burek (1981) also found that lateral movements along diagonally arranged shear zones apart from causing compressional shield folding also causes major faults and fault systems. Evidence of large tectonic forces (or stresses) was observed in most of these regions. Confirmation of the above findings has been made by other workers in this field (Illies, 1968; 1972).

Large tectonic stresses (lateral) have been postulated to be a factor in the formation of massive shear failure. This has been true in the formation of rifts, especially after the idea of plate tectonics was introduced as the fundamental explanation of the world in its present state. D'elia, et al, (1971) made a study of the mechanical behavior or a highly metamorphosed Miocene mudstone and found that areas of high tectonic activity produced much sheared material. Their discovery was rationalized thus, that the tectonic compression, which in this region is east-west in direction, causes the mudstone to undergo large shear deformations which gave rise to the faults and the shear zones. Mohr (1972) in his studies on the rifts in East Africa (Afar, Ethiopia) confirms the fact that tectonic activity in the region is a factor in
the rifting process.

Chang, et al, (1977) described the modes in which fractures in a rock occur; and in their determination of the factors involved stressed the importance of the influence of the change in the temperature and pressure inside the earth caused by tectonic movement in faulting. The five modes of fracturing were: shear-tension, shear-compression, uniform compression, uniform tension and uniform shear.

The influence of local geology in massive shear failure is further detailed in the report on massive rock failure at the Bautsch Mine, Galena, Illinois to the United States Bureau of Mines by Touseull and Rick (1974). The report concedes that unequal compression of the clay seam between areas closer and farther from the free face did increase the amount of load on supports in the area and thus might have caused the initial failures in pillars experienced. Another mechanics which they reported was the loss of cohesion and the bedding planes in the bentonite seam, which might have been the major factor in many of the small roof failures that occurred. The wall rock alteration that occurred near the mineralized zone was also a factor in most of the failures.

In discussing some of the pertinent questions pertaining to multi-seam mining, Stemple (1956) mentioned the importance of the strata in the roof of the upper seam when the underlying bed has been mined. It has been suggested then that less trouble will be experienced for a hard sandstone roof than a soft shale roof. The importance of the character of the intervening formation (interseam strata) was also
stressed in his study.

2.3 Stability Analysis

There are many types of stability analysis which can be applied to excavations in a massive rock. The majority of these methods evolved from techniques evolved for pit slopes and so a brief discussion of the pit slope stability analysis will be undertaken.

Slope stability analysis is usually treated as a limiting equilibrium mechanics problem. A failure surface is assumed and a free body is sliced from the slope. With known or assumed values of the forces acting upon the free body, the shear resistance of the soil or rock needed for equilibrium can be calculated. A factor of safety can then be determined by comparing the shear resistance to the estimated shear strength (Whitman and Bailey, 1967).

From this basis many methods have been developed, for example the Friction Circle method, Fellenius or Simplified Bishop methods, Morgenstern-Price method and the wedge method. The details in these analyses are beyond the scope of this study and will not be elaborated on further.

A new approach of stability analysis, which has been successfully used for openings in massive rocks, was developed by the United States Bureau of Mines (Wang and Sun, 1970). The accuracy of most of the conventional slope stability analyses mentioned earlier has become questionable because of the assumptions made in the calculations. The Fellenius method assumes each slice to be in equilibrium without forces acting on the sides, whilst the Bishop method assumes each slice to be
in equilibrium, with horizontal resultant forces acting on the sides (Whitman and Bailey, 1967). Static equilibrium, however, is not satisfied completely in any of these two methods and thus causes errors in the magnitude of 60 percent and 7 percent in the Fellenius and the Bishop methods, respectively.

The Bureau of Mines method of stability analysis was also treated as a limiting equilibrium problem. The method uses the finite element and Mohr-Coulomb failure criteria in the analysis (Wang and Sun, 1970; Wang, et al, 1971). The procedure usually employed in limiting equilibrium methods are that (Scott, 1980):

i) Failure occurs as a result of shear on a surface in the geological material and the failure condition is assumed to be satisfied on that surface.

ii) The failure surface is assumed and a number of surfaces are examined.

iii) For each surface, the failure load (or stress which will just cause the shear failure) is computed. Safety factors can also be computed.

iv) A systematic search is then made to find the particular surface for which the safety factor is the least.

The stress field of the structure under consideration can be obtained using the finite element method of stress analysis. Then, along a possible fracture surface, the normal and shear stresses at every point is computed using the basic strength of material equations. The resisting strength is then calculated for the Mohr-Coulomb equation,
knowing the normal and shear strengths. To calculate the factor of safety for all the points on the possible failure surface, the ratio of the resisting strength to the total shear force is evaluated. The total shear force is obtained by integrating the shear stresses along the failure surface (Wang and Sun, 1970).

The usual method of stability analysis of an underground structure has been based on the critical stress concept based on the maximum compressive and tensile stresses determined to exist around an excavation (Obert and Duvall, 1967). In this method, the check for stability is for the rock strength to be equal to or greater than the product of the appropriate critical stress and the safety factor. This system of stability analysis although widely used has been found unacceptable as a criteria for structural failure, that involves a separation of material from walls of an excavation. The critical stress method of stability analysis, however, works for initiation of cracks on the periphery of the excavation (Wang, et al, 1971). This new method of stability analysis has been used in a single opening in massive rocks and has been found to be consistent with the field observations of rib slabbing and roof arching (Wang, et al, 1971).

This type of analysis has been investigated for a single opening (Wang, et al, 1971) but it has also been shown that it can be applied to an advancing longwall face (Kidybinski and Babcock, 1973).

2.3.1 Mohr-Coulomb Failure Criterion

The clear definition of failure in rock mechanics studies is surrounded by controversy, but Silverman (1957) defines it in a way that
will be acceptable to all parties:

"Failure may be defined in several ways: as the attainment of the yield point; the appearance of the slip on the external surface of the specimen; or in the case of the so-called brittle materials, it may coincide with actual rupture. For materials which do not have a definite yield point, a certain percentage of residual deformation may be defined as failure."

To predict failure of rock many theories have been propounded and they are usually based on the following simple assumptions (Lundborg, 1968):

i) Maximum principal stress.

ii) Maximum principal strain.

iii) Maximum shear stress.

iv) Maximum shear strain.

v) Maximum strain energy.

vi) Maximum distortional strain energy

or maximum octahedral shear stress.

The Mohr-Coulomb failure criteria is based on the assumption of maximum shear stress. The theory postulates that the shear strength increases with increasing normal stress on the failure plane (Desai and Siriwardane, 1981). For a two-dimensional case, the theory stipulates that failure will occur in a material along a surface element when the magnitude of the shear stress becomes equal to the sum of the resistances due to cohesion and friction (Wang, et al, 1971).

\[ \tau = c + \mu \sigma_n \] (2)

Where \( \sigma_n \) is the normal stress acting on the surface element.
\( \mu \) is the coefficient of internal friction.

\( \tau \) is the shear stress acting on the surface element.

\[ \mu = \tan \phi \]

Where \( \phi \) is the angle of internal friction.

To express the Mohr—Coulomb failure criterion in terms of principal stresses, reference will be made to Figure 14. From Figure 14,

\[ \frac{(\sigma_1 - \sigma_3)}{2} = AB + BF \]  

(3)

Rewriting,

\[ \frac{(\sigma_1 - \sigma_3)}{2} = OAsin\phi + Ccos\phi \]  

(4)

hence,

\[ \frac{(\sigma_1 - \sigma_3)}{2} = \frac{(\sigma_1 + \sigma_3)}{2} \sin\phi + Ccos\phi \]  

(5)

The failure criteria ignores the effects of intermediate principal stress and does not allow for hardening behavior. This theory, however, takes into account the frictional aspects of the rock.

The Mohr—Coulomb failure criteria appears to be attractive to the engineer because it considers the actual behavior of materials including its inhomogeneity and anisotropy (Silverman, 1957).

2.3.2 Finite Element Method

The basic concept of the finite element method is the idealization of an elastic continuum as an assemblage of discrete elements interconnected at their nodal points (Wang and Sun, 1970). As usual, the force-deflection or stress-strain relationship and the requirement for compatibility and equilibrium are used in obtaining the stiffness properties of the elements (Wang and Sun, 1970). The basic element
Figure 14. Mohr–Coulomb Failure Criterion (after Desai and Siriwardane, 1981)
(equilibrium) equation used for the finite element analysis is (Desai and Abel, 1972),

\[
\begin{bmatrix} k \end{bmatrix} \{q\} = \{Q\} \tag{6}
\]

where \([k]\) is the element property matrix,

\(\{q\}\) is the vector of nodal displacements,

\(\{Q\}\) is the vector of element nodal forcing parameters.

The element equations are then assembled to obtain a relationship of the form (Desai and Abel, 1972),

\[
\begin{bmatrix} K \end{bmatrix} \{r\} = \{R\} \tag{7}
\]

where \([K]\) is the assemblage property matrix,

\(\{r\}\) is the assemblage vector of nodal displacements

\(\{R\}\) is the assemblage vector of nodal forcing parameters.

The boundary conditions, surface traction and specific loading conditions on any point can be incorporated into the analysis for solutions of displacements. Secondary quantities such as strains and stresses in an element can then be calculated using a transformation matrix as shown below (Desai and Abel, 1972):

\[
\{\sigma\} = [C] [B] \{q\} \tag{8}
\]

where \(\{\sigma\}\) is the stress in an element

\([C]\) is the stress--strain matrix (might be Hooke's Law if linear elastic constitutive law is to be used).

\([B]\) is the transformation matrix.

Complete discussion of the finite element method is beyond the scope of this thesis. Further reference to this method of analysis can be obtained from Desai and Abel, 1972, and Zienkiewicz, 1977, and all
such programs are computerized.

The output from most programs are in the form of displacements at nodes, the vertical and horizontal stresses, the principal stresses and the angles they make and the shear stress involved in an element. Thus, with such an array of stress components given, the finite element method is important for many uses other than analysis of stresses.

2.3.2.1 Advantages of the Finite Element Method

Before the discussion on finite element method is ended as such, it will be expedient to state the advantages it has over other methods of stress analyses (Wilson, 1965).

i) The method is completely general with respect to geometry and material properties.

ii) Complex bodies composed of many different materials are easily represented.

iii) Anisotropic materials are automatically included in the formulation and thus allows filament structures to be handled easily.

iv) Displacement or stress boundary conditions may be specified at any nodal point within the finite element system.

v) Arbitrary thermal, mechanical and accelerated loads can be specified too.

vi) Precision wise, it has been shown mathematically that the method converges to the exact solution as the number of elements is increased. Thus, the desired
degree of accuracy can be obtained.

vii) In the formulation of the finite element method, symmetric, positive-definite matrix which usually occurs in a banded form is usually obtained and thus can be solved with a minimum of computer storage and time.

2.3.2.2 Modeling Criteria for the Finite Element Method

The most important aspect of the finite element method is the discretization of the model body. This process is essentially an engineering judgment exercise and the importance of it cannot be overemphasized. The decision on the number, shape, size and configuration of the elements should be made in such a way as to simulate as closely as possible the original body (Desai and Abel, 1972).

Although it has been mentioned that engineering judgment is needed in the discretization process, some guidelines are, nevertheless, suggested to facilitate the decision-making process (Desai and Abel, 1972).

i) In the location of nodes, subdivision lines and planes are usually placed where abrupt changes in geometry, loading and material properties occur.

ii) A finer subdivision is necessary in regions where stress concentrations are expected.

iii) Curved boundaries can be approximated as piecewise linear by the sides of the elements adjacent to the boundary, if straight-sided elements are to be employed. Isoparametric elements with curved sides can be used.
iv) To minimize the solution time and the storage requirement for the overall stiffness matrix, the band width is made as small as possible. The band width is given as,

\[ B = (D + 1) f \]  

where \( B \) is semi band width.

\( D \) is the maximum largest difference occurring for all elements of the assemblage.

\( f \) is the number of degrees of freedom at each node.

In this case the numbering system is very important in terms of computer time and storage requirements. Figure 15 (b) shows the right way to number the nodes and elements.

v) The aspect ratio of the elements affects the finite element solution. The aspect ratio is defined as the ratio of the largest dimension of the element to the smallest dimension. Long, narrow elements are usually avoided when drawing a mesh for a body. The graph in Figure 16 shows the inaccuracy of solutions as a function of the aspect ratio.

vi) For a body of infinite lateral dimensions, side boundaries can be assumed and side nodes restrained to move only in the vertical direction and not horizontally.

Kulhawy (1974) showed that for geotechnical problems, such as are being considered in this study, arbitrary, linear strain quadrilateral
Figure 15. Numbering of Nodes to Reduce Band Width
(after Desai and Abel, 1972)
Figure 16. Innaccuracy of Solution as a Function of Aspect Ratio (after Desai and Abel, 1972)
elements are the most appropriate. His paper recommends a minimum of 125-150 elements for the analysis of simple structures in homogeneous rock and that the boundaries of the finite element mesh should be located at least six radii away from the center of the opening to minimize the boundary effects. (Figure 17).

2.4 Brittle Model

Physical modeling is slowly gaining popularity in the solution of very complex rock mechanics problems. Theoretical and field-measurement approaches have been used extensively in the solution of ground control problems, but these approaches have their limitations. The theoretical studies are based on the theories of elasticity and plasticity, which sometimes do not apply directly to a particular problem, because of the simplifying assumptions that must be made to obtain a solution (Barron and Larocque, 1963). Field measurements adapt the theory to the ground conditions, but the interpretation of the measurements is complicated because of the unknown and often uncontrollable variables which influence the measured phenomena. The inaccessibility of most of the structure for measurement purposes also is a problem (Barron and Larocque, 1963).

Although physical models have some limitations, it nevertheless allows for realistic empirical and theoretical equations to be deduced. It also allows for all variables which influence the phenomena to be measured. The main attractions of physical models are convenience, speed and low expense in the investigation of complex engineering problems (Whittaker, 1974).
Figure 17. Effect of External Boundary Location on Accuracy of Finite Element Method Solution (after Kulhawy, 1974)
Various physical models have been built to simulate diverse rock mechanics problems. Everling (1964) performed experiments to simulate the behavior of the ground surrounding a roadway, the support characteristics of different systems in a roadway, and to investigate the break phenomena and deformation processes in the ground responsible for the changes in the cross-section of the roadway. Model dimension was 6.6 feet by 6.6 feet by 1.3 feet and five hydraulic props on each of the four narrow sides were used to apply the required load. Each of the props has its own pump capable of exerting 60 tons of force. The top cover of this model, which was constructed at the Mining Research Station at Essen-Kray in West Germany, was stressed against the model by four props—which helped to achieve the plane strain conditions experienced underground. Geometrical scale of 1/10 was used and the strengths of coal, shale and sandstone were modeled on a strength scale of 1/10 using variations in mix of Portland cement, fused alumina cement and quartz powder.

Hobbs (1966) also performed a series of experiments using scale models to determine the movement of strata in the vicinity of mine roadways underground, controlling such factors as rock strength, pressure, roadway size, roadway shape, support design, packing system design, pack width and ribside position. The facility designed for this case at the National Coal Board Mining Research Establishment at Isleworth in England could test models of dimensions 2 feet by 2 feet by 5 inches. The model was loaded using 12 rams, 3 on each side. Each of the rams had a capacity of 3.14 tons and were all powered by an elec-
tric pump and relief valves system. Preliminary tests on the rams showed that there was little variation between the applied ram loads at the same ram pressure for individual cylinders. A geometrical scale factor of 1/50 was used. The model material is a mixture of sand, plaster and water.

The profile measuring head (N.C.B., M.R.E. Isleworth type no. 943) was used in most of the measurements in Hobbs's (1966) physical model. Everling (1964), however, used distance measuring devices and permanent photographic records in the test on the physical model.

Barron and Larocque (1963) reports on a physical model design to simulate a vertical section taken through the orebody bounded by the opposite faces of the pillar in the Wabana Iron Ore Mine in Newfoundland, Canada. The main objective was to investigate the following problems:

1) The stress distributions in the model under various conditions, as the rooms are developed.

ii) The effects of discontinuities in the ground.

The dimensions of the Wabana Mine model were 6 feet by 6 feet by 1 foot and had three 50-ton capacity rams to apply load on each side of the model. Four 2-ton rams were used to apply restraining load to the face of the model—to simulate plane strain conditions. The geometrical scale factor for the model was 1/70 and the materials used here were a variation of the water and plaster using retarders and fillers to achieve the required physical properties.

Rosenblad (1969) reported on the use of a model of approximately
22 inches by 22 inches by 6 inches to investigate the behavior of jointed rocks. The equipment developed, the materials used and the instrumentation used are all obtained in detail in the papers by Rosenblad (1968, 1969).

Sandbox models have been used successfully to simulate the possible distribution of stress in the region of coal-mine face workings as described by Harris (1974).

2.4.1 Comparison of Model Material and Natural Rock

To carry out any model study it is very important to scale the geometrical factors as well as the mechanical properties of the materials and loads involved (Barron and Larocque, 1963). The technique of dimensional analysis is usually used to specify the requirements for similitude between the model and the prototype. The basis of this technique is to collect all the physical quantities, constants, and characteristics pertaining to the problem and applying certain rules, usually the Buckingham–Π theorem, to form dimensionless parameters. The dimensionless parameters for the model must have the same value as the prototype according to the similitude condition (Barron and Larocque, 1963).

It must be emphasized here that it is usually very difficult to obtain a perfect similitude between physical models and mining conditions, although such models can achieve sufficient accuracy to contribute qualitatively and quantitatively to the solution of rock mechanics problems. Usually, some of the independent dimensionless variables, which have less significant influence on the test results, are allowed
to deviate from the correct values (Rosenblad, 1968).

Mandel (1963) describes an equivalent material as the softer model material, chosen in such a way that its mechanical properties of elastic, plastic and viscous deformation may be deduced from those of the material of the structure by a change in the scales of the stresses, times and deformations. The paper goes on to classify similitude into simple and extended types. Simple similitude is when the scale of the displacements is that of lengths, so that the deformations are equal at two homologous points in the structure and in the model. When the displacement is small a scale different from lengths can be adopted—extended type of similitude.

Hobbs (1966) after going through the dimensional analysis drew up a table to indicate the scale factors used in those model tests and also to show how other parameters are related to the basic scale factors of length, mass and time. Thakur (1968) proposed that the time scale factor for equivalent materials is equal to the square root of the geometrical scale factor under both elastic and plastic conditions.

\[ \tau = \sqrt{\lambda} \]  \hspace{1cm} (10)

The above conclusion is in agreement with the work of Hobbs (1966).

2.4.2 Properties of Model Material

Some general guidelines are given in the literature as to the properties of model materials needed in any study (Rosenblad, 1968).

i) The material should be economical, costwise.

ii) It should be easily obtainable.

iii) It should be reproducible from one batch to the next.
<table>
<thead>
<tr>
<th>Physical Parameter</th>
<th>Dimension</th>
<th>Scale Factor (a)</th>
<th>Scale Factor (b)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>L</td>
<td>$\lambda$</td>
<td>1/50</td>
</tr>
<tr>
<td>Mass</td>
<td>M</td>
<td>$\mu$</td>
<td>11/25x10^{-5}</td>
</tr>
<tr>
<td>Time</td>
<td>T</td>
<td>$\tau$</td>
<td>1/\sqrt{50}</td>
</tr>
<tr>
<td>Density</td>
<td>ML^{-3}</td>
<td>$\mu\lambda^{-3}$</td>
<td>≈ 11/20</td>
</tr>
<tr>
<td>Strength</td>
<td>ML^{-1}T^{-2}</td>
<td>$\mu\lambda^{-1}\tau^{-2}$</td>
<td>1/90</td>
</tr>
<tr>
<td>Stress</td>
<td>ML^{-1}T^{-2}</td>
<td>$\mu\lambda^{-1}\tau^{-2}$</td>
<td>1/90</td>
</tr>
<tr>
<td>Young's Modulus</td>
<td>ML^{-1}T^{-2}</td>
<td>$\mu\lambda^{-1}\tau^{-2}$</td>
<td>1/90</td>
</tr>
<tr>
<td>Acceleration</td>
<td>LT^{-2}</td>
<td>$\lambda\tau^{-2}$</td>
<td>≈ 1</td>
</tr>
<tr>
<td>Poisson Ratio</td>
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<td>1</td>
</tr>
<tr>
<td>Angle of Internal Friction</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>
iv) The material should be similar to rock in all its pertinent properties.

v) Its static properties should not change with time.

vi) Its deformability should be sufficiently large to cause adequate response in displacement and strain measuring instruments.

vii) It should be possible for measuring instruments to be easily fixed to or embedded in the material.

Further requirements enumerated by Rowe and Base (1966) list these requirements below as very important in model material properties:

i) That there should be an affinity between the stress/strain relations of model material and that of the prototype.

ii) The value of Poisson's ratio and angle of internal friction for both the model and prototype materials should be the same.

iii) The model material should be capable of easy fabrication to the required form, preferably without inducing internal stress.

iv) The material strength should be such that a loading frame could be built to fail the model.
CHAPTER III

CASE STUDIES

Specific examples of multi-seam mines and their mechanism of failure are enumerated below. Six examples of cases in which the underlying seam was mined first are described in this chapter. All these examples were collected from the literature.

3.1 Case Study No. 1

This mine is located in McDowell County in West Virginia (Stemple, 1956). Two seams are worked in the mine—Pocahontas No. 6 and Pocahontas No. 3—in the upper and lower tiers. The method of mining is the room and pillar technique with 90% planned extraction ratio in the upper bed and 92.4% in the lower seam. The thickness of the upper and lower seams are 46 inches and 54 inches, respectively. The interval between the seams is between 140 and 150 feet, and it consists of shales and sandstones in basically the same proportions. Figure 18 shows the geological column in the mining area.

The observations made after the lower seam was mined were very favorable—the disturbance in the upper seam was minimal although a sizeable pillar was left in the lower seam. Surface subsidence in this mine was also very minimal—some cracks showing up but not significant. The relative stability experienced in this case might be due to thicker interseam and the sandstones involved in the innerburden. The same reasoning can also be espoused for the minimal surface subsidence experienced.

Mining Method:

Upper seam—room and pillar
Extraction 90%

Lower seam—room and pillar
Extraction 92.4%

Figure 18. Geological Column of Case Study No. 1
3.2 Case Study No. 2

The location of the mine is in Mercer County in West Virginia (Stemple, 1956). The mine is serviced by two seams—the Pocahontas No. 11 and Pocahontas No. 3. The seams have thicknesses of 54 inches and 68 inches in the lower and upper beds, respectively. The interval between the two seams is approximately 510 feet. The geological formation in the innerburden is composed of shales and sandstones with the coal seam Pocahontas No. 6 in it (Figure 19).

Observations of disturbance is similar to that in Example 1—minimal disturbance in the upper seam and minimal surface subsidence. The greater thickness of the innerburden might be the clue to the insignificant disturbance in the upper seam.

3.3 Case Study No. 3

The mine under consideration is located in Cambria County in the state of Pennsylvania (Hasler, 1951). The geological column of the area as given by the U.S. Geological Survey is shown in Figure 20. The seams under consideration here are the lower Kittanning and the Upper Freeport for the lower and upper beds. The bed thicknesses for the lower and the upper seams are 42 inches with an interval of approximately 180 feet. The room and pillar method of mining is used in this case.

There was not a marked disturbance in the upper bed after two years had elapsed from the time of the mining of the lower seam. Significant disturbance was, however, experienced in the upper bed in the area where pillars in the underlying seam has been extracted. It should, however, be noted that the pillars in the upper bed were columnized to that in

Mining Method:

Upper seam--room and pillar
Extraction 74%

Lower seam--room and pillar
Extraction 80%

Figure 19. Geological Column of Case Study No. 2
Scale, 1 inch = 150 feet

Figure 20. Geological Column of the Central Pennsylvania Coal Area, Cambria County (after Hasler, 1951)
the lower seam. A combination of factors can be used to explain why there is some stability in the upper bed in this case—the greater seam interval, the good planning reflected in the columnization of the pillars in the two seams and the hitherto unmentioned time factor.

3.4 Case Study No. 4

McDowell County, in West Virginia, figures again in the discussion of the examples of failures in the multi-seam mines (Stemple, 1956). The seams being worked in this mine are the Pocahontas No. 11 and Pocahontas No. 3 with seam thickness of 36 inches and an average of 90 inches, respectively. 75% and 90% are the extraction ratios for the upper and lower beds, respectively. The seam interval in this mine is about 470 feet and is composed of shales, coal seams, fireclays and several sandstone beds (Figure 21).

Disturbances observed in the upper bed are very significant with cracks which stretch to the lower seam. The roof and the coal in the upper seam were also observed to be fractured. These observations point to shear failure. Surface subsidence, which is in the form of wide cracks and the settlements, were also observed. The predominance of shales in the innerburden and also the high extraction ratio in the lower seam might account for the instability and the subsidence in the upper seam.

3.5 Case Study No. 5

Consideration of shear and shear tensile failure is made in this example. The location of the mine is in Raleigh County in West Virginia.

Mining Method:

Upper seam- room and pillar
Extraction 75 percent

Lower seam- room and pillar
Extraction 90 percent

Figure 21. Geological Column of Case Study No. 4

Mining Method:

Upper seam—room and pillar
Extraction 92%

Lower seam—room and pillar
Extraction 80%

Figure 22. Geological Column of Case Study No. 5
(Stemple, 1956). The mine has two seams, Sewell and Beckley in the upper and lower beds, respectively. These coal seams have thicknesses of 48 inches in the upper and between 33 to 165 inches in the latter and they are mined at 92% and 80% extraction ratios. The interval between the seams is also variable between 260 to 330 feet with mostly shales and sandstones in its geological formation. Figure 22 depicts the geological column of the mining area.

Significant damage was experienced in the upper bed even in the virgin areas. The damage is intensified in the development work of the upper seam. Breaks to the surface from the lower seam are essentially vertical at first, but later show slight positive inclination estimated at 10 degrees maximum—shear and shear tensile type failure and subsid- ence. The low interseam height is the greatest factor in the damage done in the upper seam.

3.6 Case Study No. 6

All the mines in the examples above have been using the room and pillar method of mining. The discussion in this example is, however, concerned with the failure mechanism experienced when longwall method of mining is used in multi-seam mines. The mine under consideration has two seams, the Upper Campbell Creek and the Eagle (Peng and Chandra, 1980). Mostly shales and sandstones form the interval between the seam and has a height of approximately 190 feet (Figure 23). The lower seam was mined using the longwall method, the upper bed was basically mined by the room and pillar method with some longwall practiced in some few areas.
LOCATION: W. Va.

Cover consists of mainly sandstone and shale

15' Shale
6'6" Campbell Coal
7'1" Shale
31'4" Sandstone
1'4" Powelton Coal
14'1" Shale
2'5" Powelton Coal
3'3" Shale
42'4" Sandstone

Mining Method:

Upper seam--room and pillar
Lower seam--longwall

16'11" Shale
3' Coal and Shale
11'7" Shale
45' Sandstone
9'11" Shale
7' Eagle Coal
3'10" Shale
Sandstone

Figure 23. Geological Column of Case Study No. 6 (after Peng and Chandra, 1980)
The panel entries of the upper seam were located directly above the gob areas left by the longwall panels in the lower bed. Fewer maintenance problems occurred because of the location of the panels in the upper seam in the de-stressed zone created by the mining of the lower bed. However, as the upper seam was advanced towards the limit of mining in the lower seam, greater roof control problems at the face started. Cracks left in the roof were as high as 20 feet. Severity of the problem kept increasing even after the face moved beyond the mining limit of the lower seam.
CHAPTER IV

PHYSICAL MODELING

Physical models have been used quite extensively to study various problems in rock mechanics; and for the purpose of this study this type of models are being used primarily to investigate the failure patterns that occur in multi-seam mining and to corroborate the results obtained with the computers.

This chapter will, therefore, describe in detail all the test equipment, the development of the model material and the procedure adopted in the tests.

4.1 Description of Test Equipment

The model apparatus is shown in Figure 24.

The model to be tested has a dimension of 24 inches by 24 inches by 6 inches and is bound on the four thinner sides by 1 inch thick steel. This arrangement around the model is to achieve the uniform loading conditions in the vertical and horizontal directions. The type of steel used in all these arrangements is the ASTM A36 type structural steel.

To apply the load on the model a hydraulic system was chosen because of convenience in its usage. The load is applied directly using twelve hydraulic cylinders manufactured by Applied Power, Inc. Three rams were placed on each side of the model, so that the uniform loading conditions on the sides can be simulated close enough. Each of these rams has a capacity of 25 tons and stroke of 4 inches.
Two hydraulic pumps were used in this assembly for easy manipulation of the pressure (or load) in the horizontal and the vertical sections of the model. It was also found that to use only one hydraulic pump for this assembly will amount to maintaining equal pressures (or load) on the vertical and horizontal sections of the model which will seriously limit the objectives of the experiment.

Although the two electric powered hydraulic pumps used in the model tests are physically different, they seem to have the same basic output specifications. The pump that controls the cylinders acting vertically has a rated delivery as shown in Table 2.

This pump has a three-way valving with an "on-off-jog" switch, it is also fitted with two safety relief valves—one factory set at 10,000 psi and the other is adjustable for pre-setting maximum operating pressures.

The rated delivery of the pump that controls the cylinders acting horizontally is as shown in Table 3. A two-way manual valve is a standard attachment to this hydraulic pump and has the usual two safety relief valves also incorporated. The motor specifications in this pump reads a bit different from the previous pump described—it is rated at 1½ horsepower to 8,000 psi for continuous operation and to 10,000 psi for intermittent use. A special electrical wiring was required, however, for the 3 phase, 230 volts with 60 Hz electrical specification on this motor. A full load rated current is 10 Amperes and it has a speed of 1725 revolutions per minute.

Rigid steel tubing was chosen in the connections around the cylin-
Table 2
Rated Delivery of ENERPAC EEM 435L Pump

<table>
<thead>
<tr>
<th>Flow Rate (Cubic in./min.)</th>
<th>Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>280</td>
<td>0</td>
</tr>
<tr>
<td>195</td>
<td>500</td>
</tr>
<tr>
<td>42</td>
<td>10,000</td>
</tr>
</tbody>
</table>
### Table 3

**Rated Delivery of ENERPAC PEM 3022C Pump**

<table>
<thead>
<tr>
<th>Flow Rate (Cu.in./min.)</th>
<th>Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>700</td>
<td>0</td>
</tr>
<tr>
<td>60</td>
<td>10,000</td>
</tr>
</tbody>
</table>
ders to reduce the occurrence of damaged hoses and the subsequent replacements that might accompany the experiment if rubber tubing were used throughout the assembly. High pressure 316 stainless steel tubing of 3/8 inch outside diameter and 1/4 inch inside diameter with a working pressure of up to 10,000 psi was used. Rubber hoses capable of 10,000 psi were used for the flexibility needed in the connection of the pressure gages and the pressure relief valves around the area of the pump.

An arrangement to reduce the time taken to extend the pistons of the vertical or horizontal cylinders was incorporated in the assembly. Lines coming from cylinders acting in the same direction were connected from both sides by a tee fitting. The other side of the tee fitting then connects the pump with all the necessary arrangement for the hydraulic gage and relief valve connection. It is assumed that there is no significant loss of pressure in the lines and so the pressure readings obtained from the lines--by the gages--are the same as that applied on individual cylinders.

4.2 Design of Loading Frame

The following factors were considered in the design of the loading frame, which obviously is the most important apparatus in the model tests:

i) The stability of the frame itself in terms of toppling during the tests.

ii) That the frame should be able to sustain the maximum load that could be applied without failing.
iii) That enough space could be left after construction for a reasonable model to be tested.

iv) The frame should be simple in design, economical with materials for its construction being readily available.

Structural steel I-beams W18 x 50 of ASTM No. A36 specifications were recommended for the basic frame using the guidelines above. The loading frame was designed to 81 inches high so that it can be moved into position in the laboratory. Another restriction imposed on the design of the frame was the choice of the capacity of the cylinders to be used. The choice of 25 ton cylinders was to allow reasonable loads to be applied to the model. The regulation of the load was another factor in the choice of these rams. These hydraulic cylinders were mounted on the loading frame by bolts through holes previously drilled on the latter.

The basic design of the frame as shown in Figure 24 was checked for maximum deflection, moment, maximum shear force and reactions for each beam. A fillet weld of at least 1/4 inch using E60XX electrodes and an electric arc of length of at least 24½ inches was recommended for the joints in this design to ensure stability of the frame.

An additional feature was added to the design of the loading frame. This feature which was designed to sit at the four corners of the model is shown in Figure 25. The basic function of these features which will be referred to simply as brackets were:

-- to support the model initially in the test position before any load can be applied.
to serve as the holder of the steel plates which are needed for the uniform loading required in the test.

-- to allow for some movement in the piston action of the cylinders before the test begins.

4.3 Instrumentation

The load on the model can be read off the hydraulic gages connected to the lines that feed the cylinders. However, the most important instrumentation in these tests are the permanent photographic records kept for the various experiments. Photographic records are very important in a study like this because of the fact that similitude of the model material to the prototype is not perfect. The imperfections in the similitude mean that measurements made on a physical model could not be assumed absolute. However, the incorporation of photographic records and the hydraulic gages in this work makes for a realistic qualitative analysis.

Two hydraulic gages which are capable of measuring up to 50,000 lbs. were connected to the hydraulic lines linking the cylinders acting horizontally or vertically. The gages are mounted in the lines using gage adaptors and auto-damper valves. This additional equipment added to the gage is to prevent any damaging dial snap-back if a sudden load release occurs.

4.4 Developing A Model Material

Wide varieties of materials have been used in physical models to quantitatively and qualitatively study assorted rock mechanics problems.
Stimpson (1970) reviews most of the up-to-date information on modeling materials. Sugar cubes have been used by Trollope in modeling of a trapezoidal opening in block-jointed media.

Hobbs (1966) used various mixes of sand, plaster and water as the model material for the tests he carried out. Cement paste in this case was rejected because of the difficulty experienced in maintaining a constant strength and the brittle behavior or a weak model material made with this type of mortar. Although the compressive strengths of the plaster-water mixes were found to be of the correct order, disadvantages were noticed in them, viz;

i) Shrinkage during the setting and hydration process.

ii) Excessive compaction when the compressive strength of the plaster was exceeded.

iii) Low density.

Fine-grained silver sand filler was used to reduce the above disadvantages for strengths of up to 150 psi.

Various mixture ratio of Portland cement, fused alumina cement and quartz powder were used by Everling (1964) to simulate the 1/10 strength ratio of coal, shale and sandstone which was used in the model study. Paraffin layers were incorporated in both the roof and the floor to simulate bedding planes.

Rosenblad (1968) reported on the development of a model material for the study of the failure modes in intact and jointed rock masses. The material used was the rounded Valley, Nebraska river sand, hydrocal B-11 gypsum cement, and water in 7.6:1:1.4 proportions by weight,
respectively. This material had an unconfined compressive strength of 610 psi and a direct tensile strength of about 85 psi, it has a void ratio of 0.6 and a dry unit weight of 120 lb. per cu. ft.

For the simulation of the Wabana Iron Ore Mine in Canada, Barron and Larocque (1963) used some variation of the water/plaster of Paris ratio and some retarders and fillers. The geometrical scaling factor for this material was 70 and that of the force is 13.7.

Barton (1970) developed a model material of red lead-sand/ballotini-plaster-water to study rock slope failure. The model material here had an unconfined compressive strength as low as 5 lbf/in$^2$ and Young's moduli ranged between 350 and 560 lbf/in$^2$.

Stimpson (1970) attempted to classify in a simplified way the modeling materials which can or have been used for rock and soil mechanics problems. This classification is as in Figure 26; the fact that the mechanisms of rock and rock-like material deformation in compression showed that dilation was important was used in this classification.

Close scrutiny of the classification in Figure 26 shows the variety of model materials which could have been used for this study. The basic factors that prevailed finally for the choice of the material in this study was:

i) The ability to satisfy the similitude conditions.

ii) The ease of fabrication of the model.

iii) The ability of the material to set reasonably fast.

iv) The availability and the relatively quick delivery of the material.
Figure 20. Classification of Modeling Materials (after Stimpson, 1970)
v) The workability of the material.

vi) The ability to use fillers to make a weak model which will fail under test.

The material chosen for this study was a commercial cement—POR-ROK cement, manufactured by Hallemite, Lehn & Fink Industrial Products Division of Sterling Drugs, Inc. and ordinary building sand as the filler material. This cement sets in about 15 minutes and approximates various rock properties by the addition or exclusion of various proportions of sand. The more sand is added the lower the strength of the model material. Less water is also needed for this type of cement—3 oz. of water for every 1 lb. of POR-ROK used.

4.4.1 Laboratory Testing

The choice of the mix proportions and the method of mixing was achieved by laboratory testing—unconfined compression tests. For a model of dimensions 24 inches by 24 inches by 6 inches, with three 25 ton cylinders acting uniformly on the sides, 350 psi is the maximum stress that can be applied on it. Thus, to achieve failure of the model, the strength of the models to be tested should be below 350 psi.

Cylindrical specimens of the different proportions of the cement and the filler chosen were tested. The length to diameter ratio for all the samples tested was made to be about 2.0 (Hobbs, 1967). Figure 27 shows the relationship between the uniaxial compressive strength and the cylinder length for cylinders of 2.5 cm diameter (Hobbs, 1967). Figure 27 proves the necessity of using the length/diameter ratio of 2, if the inevitable increase in strength of cylin-
Figure 27. Relation Between Compressive Strength and Cylinder Length (after Hobbs, 1967)
ders of L/D ratio less than 2 is to be avoided. Hobbs (1967) attributes the increase in strength in the shorter samples to the influence of the end restraint, which causes the stress system to be essentially triaxial throughout the whole volume of the rock specimen. The lesser probability of a gross weakness being present in the shorter specimen is also cited as a factor.

Uniaxial compressive strength tests were conducted on the specimens of sand and POR-ROK cement with different proportions of mix. The results of those mixtures after 28 days of curing under room temperature conditions are summarized in Table 5.

Three samples of each type of mixture were tested and averaged out for the uniaxial compressive strength. All the testing was done using the million pound capacity M.T.S. stiff testing machine at the Mining Department of Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

4.5 Procedure

A mold of inside dimensions of 24 inches by 24 inches by 6 inches was constructed with wood to make the concrete models for these tests. The concrete models had a composition of 70% by weight sand and 30% by weight POR-ROK cement. A block of wood 9 inches by 6 inches by 1\(\frac{1}{2}\) inches was placed in the mold 7\(\frac{1}{2}\) inches from one end before the pouring of the concrete was done. This block of wood created a hole to simulate a mined area.

The concrete was mixed by hand and left under room conditions to cure (Table 4). The tests were carried out with the side with shorter
Table 4

Test Results of Uniaxial Compressive Strength of Different Mixtures of Sand and POR-ROK Cement

<table>
<thead>
<tr>
<th>% Sand</th>
<th>Diameter (in.)</th>
<th>Length (in.)</th>
<th>Failure Load (lb.)</th>
<th>Displacement (in.)</th>
<th>Failure Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>2.699</td>
<td>4.426</td>
<td>22,500</td>
<td>0.01575</td>
<td>3932.7</td>
</tr>
<tr>
<td>20</td>
<td>2.695</td>
<td>4.262</td>
<td>23,500</td>
<td>0.01550</td>
<td>4119.6</td>
</tr>
<tr>
<td>30</td>
<td>2.686</td>
<td>4.213</td>
<td>24,500</td>
<td>0.01650</td>
<td>4323.8</td>
</tr>
<tr>
<td>40</td>
<td>2.690</td>
<td>4.187</td>
<td>18,750</td>
<td>0.0154</td>
<td>3299.2</td>
</tr>
<tr>
<td>50</td>
<td>2.693</td>
<td>4.149</td>
<td>13,250</td>
<td>0.0150</td>
<td>2326.2</td>
</tr>
<tr>
<td>60</td>
<td>3.334</td>
<td>6.346</td>
<td>16,770</td>
<td>0.0130</td>
<td>1920.5</td>
</tr>
<tr>
<td>70*</td>
<td>3.330</td>
<td>6.324</td>
<td>4,000</td>
<td></td>
<td>459.3</td>
</tr>
<tr>
<td>80*</td>
<td>3.298</td>
<td>6.572</td>
<td>1,600</td>
<td></td>
<td>187.3</td>
</tr>
<tr>
<td>90*</td>
<td>3.330</td>
<td>6.380</td>
<td>1,000</td>
<td></td>
<td>114.8</td>
</tr>
</tbody>
</table>

*3 days curing at room temperature.
distance to the hole on the bottom cylinders and the reverse carried out with the second model. The cylinders acting horizontally were first activated up to a maintained load of 2000 lb. (387.82 psi); and the cylinders which act in the vertical direction are advanced until model failure. Photographs are taken at the beginning and end of the tests.
CHAPTER V

COMPUTER MODELING

The limiting equilibrium approach as published by Wang, et al, (1971) was used in the analysis. This method of stability analysis as has been stated earlier has the great advantage of the versatility of the finite element method to achieve realistic solutions to ground control problems. Plane stress and plane strain two-dimensional finite element analysis can be made for an underground structure of given geometry, loading and material properties. Usually in practice, plane strain analysis is done assuming that the length in the third plane is infinite. The basis of the finite element method has been discussed in a preceding chapter.

The stress field around the underground structure under consideration can be determined from the finite element analysis. A possible fracture surface can then be calculated from the principal stress directions obtained from the finite element method, using the linear Mohr-Coulomb failure criterion. The possible fracture surface is computed bearing in mind that at any point, the tangent to the surface is oriented such that the quantity $(|\tau| + \mu \sigma_n)$ is a maximum at an angle $\theta$, from the maximum principal stress direction. $\theta$, however, is related to the coefficient of internal friction as shown below (Obert and Duvall, 1967; Wang, et al, 1971)

$$\tan 2\theta = \frac{1}{\mu}$$  \hspace{1cm} (11)

Smooth curves to connect the tangents will produce a pattern of
Figure 28. Stress at a Point on a Fracture Surface (after Wang et al. 1971)
the possible failure surfaces over the entire structure. Two families of fracture surfaces intersect each other at an angle of $2\theta$ or $180^\circ - 2\theta$ (Wang, et al, 1971).

A typical fracture surface and the definition of the parameters is as shown in Figure 28. The equations used in this analysis are as below (Wang, et al, 1971):

Normal stress,

$$\sigma_n = \frac{1}{2} (\sigma_x + \sigma_y) - \frac{1}{2} (\sigma_x - \sigma_y) \cos 2\phi + \tau_{xy} \sin 2\phi$$  \hspace{1cm} (12)

Shear stress,

$$\tau_{nm} = \tau_{xy} \cos 2\phi - \frac{1}{2} (\sigma_x - \sigma_y) \sin 2\phi$$  \hspace{1cm} (13)

Where $\sigma_x$, $\sigma_y$, $\tau_{xy}$ are the horizontal, vertical and shear stresses, respectively. (They are obtained from the finite element stress analysis).

$\sigma_n'$, $\tau_{nm}$ are the normal and shear stresses acting on the fracture surface (Figure 28).

and $\phi$ is the fracture surface orientation (Figure 28).

The above two equations are used in the calculation of safety factor of the fracture surface at a point. Integrating over all the points along the fracture surface, the safety factor relationship becomes (Wang, et al, 1971),

$$\text{Factor of safety} = \frac{\Sigma (c + \mu \sigma_n)}{\Sigma \tau_{nm}}$$  \hspace{1cm} (14)

This factor is the stability index; the higher it is the more stable the structure becomes (Wang, et al, 1971).
The procedure detailed above was all incorporated into the two-dimensional finite element computer program developed by C. S. Desai and J. F. Abel (1972). See Appendix A. Some assumptions were made in the algorithm in the modification of the finite element program to incorporate the stability analysis. The analysis was made for each element of the mesh. Only one point on the possible fracture surface was considered and this point was assumed to be at the centroid of the element. The fracture surface orientation, which is the tangent to the surface or the tangent from the vertical axis (Figure 28), is calculated accurately using the principal stress orientation and the angle given by the coefficient of internal friction. Factors of safety are calculated for each element. These factors of safety are then contoured using the General Purpose Contouring Program (GCPD) developed by Batten and made available by the California Computer Products, Inc.

5.1 Finite Element Mesh Generation

Five standard models were analyzed in this as shown in Figures 13, 14, 15, 16 and 17. These models were constructed bearing in mind the modeling criteria for the finite element as described earlier and also documented by Kulhawy (1974). The initial model (Model I) shown in Figure 13 was to simulate the situation in the Pittsburgh or Big Vein seam at Georges Creek Region, Maryland as reported by Stemple (1956) and Holland (1951) and illustrated in Figure 1.

The geological columnar stratigraphic section used in the modeling procedure was simplified. The column is shown in Figure 29. Shorter computer time and reduced modeling complexity was the main aim of the
Figure 29. Simplified Geological Column Used in the Analysis
simplification. The Pittsburgh or Big Vein seam used in the analysis was 14½ feet thick with an overlying Redstone seam of 4 feet. The innerburden consisted mainly of shales with some intercalations of sandstones. The innerburden was, however, modeled as shales alone with material properties modified to allow for the sandstone intercalations. To reduce the boundary effects in the model a sandstone overburden was incorporated and a shale floor to the underlying coal seam was also added. The lateral boundaries were placed sufficiently distant from the opening to minimize the boundary effects of the model which can significantly affect the accuracy of the results obtained from the finite element method.

To extend the studies on Model I a second model with an opening in the upper bed of the same length as the lower seam was constructed—Model II. Model III was constructed to simulate room and pillar mining which is very prevalent in this area of the Appalachian Coalfields to allow some practical recommendations to be made. The pillars in this latter model were columnized—the pillars in the lower layer were placed exactly below the upper layer. The last two models were variations of the staggered pillars that could be expected in practice.

These meshes for the finite element program were 240 feet in length and 75 feet high. A vertical stress of 1000 psi was loaded on the top of the model to simulate the overburden pressure (cover load). No vertical displacement in the nodes at the lower boundary of the model was permitted. The side nodes of the model were restricted in their movement in the horizontal direction. All the other nodes in the model
were free to move in all directions.

The numbering system used here was quite contrary to that espoused in an earlier discussion. The band width in the finite element method using Equation 9 should be minimized as far as possible as recommended by Desai and Abel (1972). The nodes and elements are, however, numbered laterally, although numbering vertically would have produced a smaller band width. The meshes were numbered this way to take some advantage of the automatic generation of nodal and element data by the computer code being used. Numbering laterally also makes it easy to remove elements to simulate excavation.

The material properties used in this study were taken from the literature (Agbabian, 1978; Obert and Duvall, 1969) and they are as shown in Table 5.

5.1.1. Model I

As has been mentioned earlier, Model I was to simulate the situation in the Pittsburgh or Big Vein seam at Georges Creek Region, Maryland as shown in Figure 1. The situation was, however, simplified to the mesh shown in Figure 30. The model has an overall dimension of 240 feet by 75 feet with an 80 feet by 9.5 feet opening in the lower coal seam (See Figure 30). Shales make up the floor of the opening, with coal roof. The shale floor is 15 feet thick and an interseam of shale of 29.5 feet. Five feet of coal was left on top of the opening—roof of opening.

The coal seam in the upper bed is 4 feet thick, 12 feet from the top. 1000 psi pressure was applied to the top of the model. This
Table 5

Material Parameters Used in the Analysis
(after Agbabian, 1978; Obert and Duvall, 1969)

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Modulus $E \times 10^8$ psi</th>
<th>Poisson Ratio $(v)$</th>
<th>Density $(p)$ pc</th>
<th>Cohesion $(C)$ psi</th>
<th>Angle of Internal Friction $\phi$ degrees</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shale</td>
<td>1.61</td>
<td>0.10</td>
<td>158.4</td>
<td>167040.0</td>
<td>55.0</td>
</tr>
<tr>
<td>Coal</td>
<td>0.72</td>
<td>0.23</td>
<td>128.2</td>
<td>61200.0</td>
<td>15.0</td>
</tr>
<tr>
<td>Sandstone</td>
<td>2.97</td>
<td>0.13</td>
<td>144.8</td>
<td>819360.0</td>
<td>43.23</td>
</tr>
</tbody>
</table>
pressure had to be converted to concentrated loads at the nodes at the
top of the mesh. To achieve the concentrated loads at these nodes, the
principle of equal influence of loads at nodes was used. The idea of
equal influence area is to assume that the load at each node on top of
the model acts halfway to the left and also to the right of the node
in question.

This model contained 574 nodal points and 504 elements.

5.1.2 Model II

Model II was similar to Model I except for the addition of an open-
ing in the upper seam with a dimension of 80 feet by 4 feet. The mesh-
plot of this model is as shown in Figure 31. The geology of Model II
was the same as Model I. The assumption of 1000 psi pressure on top
of the model is also used in this case.

574 nodal points and 487 elements were under consideration in this
model.

5.1.3 Model III

This model has an overall dimension of 240 feet by 75 feet and has
four openings in the two coal seams (See Figure 32). The two openings
in the lower seam have dimensions of 20 feet by 9.5 feet each with a
60 foot pillar in between the two openings. The other two openings in
the upper coal seam were constructed so as to produce a columnized pil-
lar system as in practice. The openings are each of dimensions 20 feet
by 4 feet. This model is similar to Model I in the case of the geology,
except that there are two openings in the lower seam and upper seam as
opposed to only one opening in the lower one. Two upper seam openings are also incorporated to simulate room and pillar situations.

This model had 574 nodal points and 504 elements.

5.1.4 Model IV

Model IV has the same dimension and geology as Model I and was aimed at simulating room and pillar system as it applies to multi-seam mining. Staggered pillars were considered with its center to center dimensions of rooms as 80 feet. This means that pillars are 60 feet and openings are 20 feet in length which is the usual dimensions used in mines in the Appalachian Region. Two openings were located in the lower seam and three in the upper bed.

Using the same numbering system as before, a mesh of 574 nodal points and 500 elements were considered. The loading and boundary conditions were as previously used for the three models mentioned earlier. The mesh for Model IV is as in Figure 33.

5.1.5 Model V

Model V is similar to Model IV except that the locations of the openings were somehow reversed. Two openings were located in the upper seam and three in the lower one. 574 nodal points and 500 elements were under consideration.

The meshplot of Model V is shown in Figure 34.
Figure 33. Model IV
Figure 34. Model V
5.2 Computer Codes Used in the Analysis

Three sets of computer programs were used in the present study, viz,

i) The Mesh-plot program.
ii) The Finite Element program.
iii) The General Purpose Contouring program.

5.2.1 The Mesh-plot Program

Apart from using the Mesh-plot program in this analysis as a pictorial view of the mass of data input to the computer, this program was also used as the only check on the input data errors. The finite element code used in this analysis, which has been listed in Appendix A, has its own internal data error checks. The Mesh-plot program, however, allows one to spot the errors in the computer plot that is generated easily.

This program is written in FORTRAN IV with packaged plotting subroutines available at the Virginia Tech Computing Center. Two plotters can be used with this program—the Versatec 1200 electrostatic plotter and the Calcomp 1051 drum plotter. The input data to this program is in the same format as that of the finite element program, thus its usefulness as a check on the input data of the latter program which is bigger and expensive to use.

5.2.2 The Finite Element Computer Code

There are lots of computer codes used for the finite element analysis, but the code selected for this study is that developed and
published by Desai and Abel (1972). This code is relatively simple and because of its simple and generalized nature is not very efficient for large-scale problems. The code is limited to linear, elastic, plane strain or plane stress analysis of isotropic bodies. Only one loading case may be accommodated for each problem. Notwithstanding all these limitations, the program has been successfully applied to many solid mechanics problems.

The elements used are 4-Constant strain triangle quadrilaterals and/or constant strain triangles. Various subroutines are employed in this code to carry out most of the computational steps. These subroutines are controlled by the main routine which reads the title of the problem, computes the semi-band width, solves the overall equilibrium equations and prints the displacements.

5.2.3 The General Purpose Contouring Program (GCPG)

The General Purpose Contouring Program (GCPG) is capable of representing graphically the functions of two independent variables as contour diagrams. GCPG was originally developed by George W. Batten, Jr. of the University of Houston, Texas and is available on the Versatec 1200 electrostatic and the Calcomp 1051 plotters.

The program can be used in a wide variety of applications ranging from the generation of contour maps of gravitational fields and strata depths in the realm of geophysics to the depiction of temperature and barometric pressure in the field of meteorology. GCPG has been used in the depiction of contour maps of stress around an opening, (Bhattacharya, 1980).
There are two basic modes in which data could be input into GPCP:

--- data consist of the function values at the mesh points of a rectangular array (gridded data).

--- function data at arbitrary points within the region of interest (irregular data).

In the case of the random data points, GPCP generates gridded data by analytically constructing a smooth surface passing through every data point. Gradient information can also be specified to facilitate the determination of the mesh point estimates derived from the random data.

GPCP is very flexible in that the user could specify the manner in which the contour lines generated could be plotted—bold lines, dash lines, lines with or without labels, etc. Areas could be blanked out so that no contours are plotted in those regions. Annotations, in the form of lines and alphabetic symbols can also be included in the plotted maps.

GPCP was made available as a proprietary package program from the Virginia Tech Computing Center, therefore a listing of the program was not available.

5.3 Procedure

The finite element mesh which has been discussed earlier is obviously very important in this analysis. After a reasonable mesh has been conceived, the whole formulation is digitized and input into the computer. The nodes were numbered from left to right—the reasons for that are given in an earlier section. The number of node, its coordinates and the boundary conditions are all input together. The element
data describes the location of each element with respect to the nodal points which had been supplied earlier. The material type of which the element is part is also included in the data cards describing the element.

To facilitate the input procedure, the computer automatically generates the nodal point data if the intermediate nodal points fall on a straight line and are equidistant. The first and last points on the line, however, will have to be specified. The computer code used in this case will assume automatically $\text{KODE} = 0$ (boundary condition, where loading conditions in the x and y directions are prescribed). Automatic generation of element data also exist for elements which are on a line in such a way that their corner node indices each increase by one compared to the previous element. The computer code will, however, assume the same material type in its automatic generation.

Material properties used in this analysis which has been stated in Table 5 will have to be specified even before the digitization process of the nodal and element data. The boundary conditions for the model are also incorporated in the data fed into the computer.

The output from the modified finite element method of analysis gives us an array of stresses ranging from the vertical stresses to the angle of the principal stresses. The safety factor in every element (occurring at the centroid of the element) is also calculated. However, for easy access to the part of the output concerning the safety factors at the centroid of the elements, the values of the coordinates and their corresponding safety factors and displacement are stored on a disk.
The data stored on disk is fed into GPCP to produce contours of the safety factors. The data which is stored on the disk are prepared to make it easier for the input process into the contouring program. This means that the data is stored in the same format as that for GPCP.
CHAPTER VI

RESULTS

6.1 Computer Modeling

The results obtained from the stability analysis done on the computer is in the form of contour maps and thus the interpretation of them is very important in this study. From the theory of the technique implemented in this study, fracture will be most likely to occur first along the surface that has the lowest factor of safety, which will be referred to herein as the critical fracture surface (Wang, et al, 1971). This technique does not, however, use the absolute values of the safety factors, but gives a good exhibit of the trend of the shear failure in a structure.

Observation from the results from all the models described earlier will be undertaken in this chapter. Tests carried out especially with Model I will be explained and observations made from the results obtained will be stated. It should be emphasized, however, that the observations stated in this text are valid for the material properties and constitutive law of rocks (which, in this case, is linear elastic) used.

6.1.1 Model I

The description of the model has been stated before in Section 5.1.1. The model basically has one opening of 80 feet by 9.5 feet in the lower coal seam. The main objective for the construction of this model was to simulate the effect of mining the lower coal seam on the
unmined upper coal bed. Figure 35 shows the safety factor contours obtained using the material properties tabulated in Table 5. Further experiments (or tests) on modulus, Poisson ratio, density, angle of internal friction and cohesion were undertaken to investigate the mechanism of failure in the interseam. The sensitivity of the material properties used in this analysis was tested to fully understand the failure mechanism in the interseam. The observations in these tests will be discussed later on in the chapter.

The critical failure surface, which is the lowest safety factor contour line, is the 20 contour line in this case (Figure 35). It must, however, be borne in mind that a lower safety factor contour other than the chosen contour could be obtained from Figure 35 but for the fact that the contouring program is very efficient with this contour interval--20.

The approximate angle the shear failure line makes with the horizontal is about 70°. Field observations have confirmed this angle as very common in cases as that being considered in this model. As commonly observed underground, the critical failure surface as it is in this contour plot occurs at the edge of the opening. This phenomena has been observed by workers like Aggson (1979).

Observing closely the contour plot obtained from Model I, higher safety factors tend to be experienced in the middle portion of the interseam. As in almost all the cases the critical fracture surface around the opening is neither the nearest nor the most distant of all possible fracture surfaces that begin and end on the boundary of the
excavation (Wang, et al, 1971). Illustrating how useful this method of analysis is, safety factor values obtained in the center of the immediate roof of the opening were very small indeed, thus strengthening the fact that tensile failures could occur in those areas.

It is noticeable that the contours at the boundary of this model and many other contour plots in this study are not uniform (i.e., almost vertically straight contours) contour lines. This is because of the limitation on the storage capacity of GPCP, very large values of safety factors had to be removed from all the data sets for the program to work. Most of the large values, incidentally, were at the boundaries, as expected, thus the non-uniformity of the boundary contours.

The region of high safety factors is very extensive in the middle part of the interseam layer although the initial failure at the edge of the opening is the most critical of all the failure regions. The extent of the predicted failure in the upper seam for this model was judged to be an important determination especially in checking the interactive effects of seams. To determine the extent of this failure, the distance between the 20 safety factor contour at the floor of the upper bed was measured, bearing in mind the scale factors used in the drawing of the contours. It was especially important in the tests carried out and described in later sections that some consistency in the determination of the extent of the predicted failure in the upper seam was established. This, therefore, explains the use of the floor of the upper coal seam as the reference.
6.1.1.1 **Effect of Modulus**

This test on Model I was undertaken to evaluate the effect of modulus changes on the mechanism of failure in this study. To achieve this objective, the ratio of the modulus of the shales to that of the coal \( \frac{E_{\text{shale}}}{E_{\text{coal}}} \) was varied and the effects studied. The moduli of the coal seams were, however, kept constant and that of the shales varied. This was undertaken with the understanding that the shales were the most important geological formation in the study because of its predominance in the interseam.

The rationale behind the use of the ratio of the modulus was for a simplified analysis, that is, the use of simple integers in the graph plots and description. \( \frac{E_{\text{shale}}}{E_{\text{coal}}} \) ratios of 2, 4, 8 and 10 were undertaken in this study.

The plot on Figure 36 is a clear indication as to the trend of the extent of failure in the upper seam when the lower seam is mined, as the interseam modulus is increased. It is noticeable that a change in the ratio of the modulus of the rocks in the model affects the extent of failure in the overlying seam. As the modulus of the shale in the interseam is increased the extent of failure in the upper seam generally increases as well.

Figure 37 shows the critical fracture surfaces which occur with changes in the modulus of the innerburden.

Individual contour plots for the changes in modulus made in this analysis are shown in Appendix B. As the modulus of the interseam increases, the failure surface in the middle portion of the shales in
Figure 36. Effect of Modulus of the Interseam on Failure in the Upper Seam
Figure 37. Critical Fracture Surface for Varying Modulus of the Innerburden
this region becomes thinner. The thinning effect in the middle region of the interseam can be explained by the brittle nature which the shale assumes as modulus increases. The brittle nature of the interseam also explains the expansion of the angle of fracture in the interseam; the angle increases from 70° to $E_{\text{shale}}/E_{\text{coal}}$ ratio of 2 to 90° when the ratio is 10.

Observing the middle of the immediate roof of the excavation in the lower seam, it can be noticed that as the $E_{\text{shale}}$ value increases the values of the safety factor contours also increase.

6.1.1.2 Effect of Poisson's Ratio

Poisson's ratio plays an important role in the solution of the finite element method of stress analysis, especially in the consideration of lateral stresses which have been found to be very important in this case (shear failure). It was, therefore, of great interest to try and evaluate the effect of changes in the Poisson's ratio in the interseam on the stability in the interseam.

For easy analysis, the Poisson's ratios of the coal seams was kept constant and that of the shales which dominates the interseam was varied. The ratio of the Poisson's ratio of the shales to that of the coal seams were used for the same reasons as mentioned in the previous section. The ratios used in this analysis were 0.3, 0.5, 1.0 and 1.5. It was not possible to increase the ratio beyond 1.5 as the maximum Poisson's ratio of 0.5 was being approached and the degree of plasticity after that point will be too high, thus unrealistic for any practical purpose.
Figure 38 shows the relationship between the extent of the predicted failure in the upper seam when the lower bed is mined and the gradual increase in the Poisson's ratio of the interseam as in Model I. Generally from the graph, there is an increase in the lateral extent of failure in the upper seam as the Poisson's ratio of the interseam is increased.

Figure 39 shows the critical fracture surface plots obtained for changes in the Poisson's ratio of the interseam. The thinning out of the middle section of the interseam encountered in the previous section on the incremental modulus effect does not occur in these contour plots. Actually, the middle region of the interseam bloats out from $70^\circ$ to about $90^\circ$ failure angle as the Poisson's ratio in the interseam increases. The higher degree of plasticity as the Poisson's ratio is increased may account for this bloating effect in the middle section of the interseam experienced in this case.

The middle of the immediate roof of the opening appears much "safer" as the Poisson's ratio of the interseam is increased. This means that as Poisson's ratio of the interseam is increased the immediate roof of the opening assumes higher values of safety factors. The increase in plasticity may also account for this favorable region in the immediate roof.

6.1.1.3 Effect of Density

The weight of the rocks above an opening is very vital to the stability of the excavation. This factor was studied varying the density of the shale in the interseam of Model I. For the use of
Figure 33. Effect of Poisson's Ratio of the Interseam on Failure in the Upper Seam
Figure 39. Critical Fracture Surfaces for Varying Poisson's Ratio of the Innerburden
simple integers in the analysis of this factor the ratio of density of
dense to that of coal \( \rho_s/\rho_c \) were used. The ratios considered in
this analysis are 0.5, 1.0, 1.5 and 2.0. To achieve these ratios the
density of coal was kept constant and the density of shale varied.

The plot on Figure 40 shows the relationship between the lateral
extent of the predicted failure in the upper seam when the lower bed
is excavated and the ratio of the density of shales to that of coal in
Model I. The plot obtained in this exercise shows that as the density
in the interseam is increased, the lateral extent of the failure in
the upper seam decreases until \( \rho_s/\rho_c = 1.4 \) then it starts to increase.

After the \( \rho_s/\rho_c = 1.4 \) has been reached, it can safely be inferred
that the weight of the interseam plays a greater role on the failure
pattern which occurs. Other factors such as the stiffness of the mate-
rial and the lateral stresses must have had a lot more influence on the
interseam failure before this ratio of densities was reached.

Safety factor contour plots of the various ratios are as shown in
Appendix B. Figure 41, however, shows the critical failure surfaces of
the model when the density of the innerburden is changed. In a general
observation of the contour plots obtained from this computer analysis,
it can be noticed that there is not much difference in the basic fail-
ure pattern for \( \rho_s/\rho_c \) ratio of 0.5 and 1.0. The angle of failure as
obtained for both contour plots is approximately the same. The only
significant difference that is apparent in the two plots is the lateral
extent of the failure in the upper seam which is shown in Figure 40.
Figure 40. Effect of Density of the Interseam on Failure in the Upper Seam
Figure 41. Critical Failure Surfaces for Varying Density of the Innerburden
Dramatic differences in the contour plot of $\frac{\rho_s}{\rho_c}$ ratio of 1.5 and 2.0 are experienced, nevertheless. Apart from the apparent large differences in the lateral extent of the failure in the upper seam, the middle section of the upper seam which usually has a series of high safety factor contours in all the plots are different. Those high safety factor contour areas in the upper seam seem to flatten out in structure until the $\frac{\rho_s}{\rho_c}$ ratio gets to 2.0 when the values get lower and flatter.

It should be realized that the area between the critical failure surfaces contain some competent rocks. This is to say that the mechanism of this massive failure which happens in the interseam in the multiple horizon mining is mainly shear in nature, and this is justified by the high values of safety factors encountered in between the failure surface.

6.1.1.4 Effect of Cohesion

The parameter cohesion plays an important role in the calculation of shear stresses. It is also one of the important parameters considered in the linear Mohr-Coulomb failure criteria. Cohesion, therefore, was considered a very vital parameter to study. The values of cohesion in the interseam were varied with that of the other seams or materials kept constant. The usual ratio of cohesion of shales to that of coal was used in this analysis in a bid to simplify the interpretation of the results. The ratios used were 1.0, 2.0 and 3.0.

The results obtained from this analysis have been summarily plotted in Figure 42. This plot shows the trend of the lateral extent of
Figure 42. Effect of Cohesion in the Interseam on Failure in the Upper Seam
failure in the upper seam when the lower bed is mined as cohesion in the interseam is increased. Actual safety factor contours are shown in Appendix B. The four figures shown in this case gives the impression that the parameter cohesion (of the interseam layer) has no effect on the manner in which the whole structure fails. Figure 43 shows the critical failure surface for changes in the cohesion of the innerburden. It was suspected, however, that the cohesion factor in the calculation of the safety factor was insignificant as compared to the normal stresses (Equations 2 and 14). All observations mentioned under Section 6.1.1 (on Model I) apply in this case. In fact, both of the exercises have identical safety factor contour plots.

6.1.1.5 Effect of Angle of Internal Friction

The linear Mohr-Coulomb equation and the safety factor equation in Equations 2 and 14 show that the angle of internal friction (\( \phi \)) is one of the factors which needs consideration in any stability analysis like in this study. As usual, \( \phi \) of the interseam layer which is predominantly of the rock type shales was varied keeping that of the coal constant. The usual ratio of \( \phi_{\text{shale}} / \phi_{\text{coal}} \) were considered; in this case, 1.0, 2.0, 3.0, 4.0 and 5.0.

The results obtained in this exercise are summarized in Figure 44. The other contour plots were ignored because they were all identical to that shown in Figure 45. The graph on Figure 44 also depicts the relationship between the angle of internal friction (\( \phi \)) of interseam layer and the lateral extent of failure in the upper layer, when the lower bed is mined. There seems to be no apparent relationship, that is, \( \phi \) of the
Figure 43. Critical Fracture Surfaces for Varying Cohesion of the Innerburden
Figure 44. Effect of Angle of Internal Friction of the Interseam on Failure in the Upper Seam
interseam has no effect on the failure pattern experienced in the model studies.

In leaving this discussion on the effect of $\phi$ on interseam failure, it will be expedient to mention that the contour plots of Model I (using the material properties in Table 5) is identical to the plots obtained in this exercise.

6.1.1.6 Effect of Proximity of Seams

Hasler (1951) and Lazer (1965) observed in the coal mines in the Pocahontas Coalfields that interactions were non-existent when intervals between seams are greater than 50 feet. Other intervals have been observed in other coalfields which might imply that the geology of the area might be a factor in the interaction of the contiguous seams.

With these observations in mind, Model I was expanded vertically in the interseam to study the effect of the bed interval. Various meshplots were drawn to accommodate seam intervals of 29.5 feet (which is the standard Model I), 49.5 feet, 69.5 feet, 89.5 feet and 119.5 feet. These meshplots were designed on the same basis and guidelines as mentioned earlier. They all had an opening of 80 feet by 9.5 feet in the lower seam with 5 feet of coal as the immediate roof. The upper seam was unmined. The horizontal lengths of all the meshes were 240 feet, although the vertical lengths for them varied in each case. The 29.5 feet interval had 75 feet vertical length as against 165 feet for 119.5 feet seam interval.

The boundary conditions used in all these cases were the same as that described before. This means that a 1000 psi pressure was applied
on the top of each of the mesh created and distributed uniformly on the top. There was no vertical displacement allowed in the nodes at the lower boundary of each mesh in this case. The nodes at the side boundaries of the meshes were only allowed to move vertically and not horizontally, all other nodes were, however, allowed to move freely in any direction.

The same numbering system as described under Model I was used and the material properties previously specified in Table 5 were used in this analysis.

Safety factor contours obtained after the finite element analysis are shown in Appendix B. The trend of the lateral extent of failure in the upper seam when the lower bed is mined, as the seam interval increases is depicted in Figure 46. Figures 47, 48, 49 and 50 show the critical failure surfaces which have been isolated from the safety factor contours. The critical failure surfaces could not be superimposed because of differing scales. The usual safety factor contour assumed for the critical failure surface could not be applied in this case because in most of the cases, the 20 contour line did not run into the upper seam. The lowest safety factor contour that could reach into the upper seam and still show the trend of failure in it as the bed interval was increased was found to be the 60 contour line.

An interesting observation was made from Figure 46, that the lateral extent of the failure predicted by this method in the upper seam decreases as the distance between the bed increases until the interval reaches about 52 feet, then it starts to increase. It was believed
Figure 46. Effect of Proximity of Two Seams on Interseam Failure
Figure 47. Critical Failure Surface (Interseam Distance = 49.5 ft)
from studying the graph and the safety factor contour plots that the
mined lower bed ceases to be a greater factor in the failure of the
upper seam after the interval between the two seams is beyond about
52 feet. Consistent reduction in the lateral length of the failure in
the upper seam when increasing the interval between the beds until the
52 foot mark gave rise to the above inferences. Gradual increment of
the failure region after the 52 foot interval was also observed; but to
make the inference more justifiable, a rapid increment in failure in
this region after the 90 foot interval was experienced showing that
there was more than the mined lower seam involved in the cause of the
failure and that its involvement was diminishing rapidly as the interval
between the beds increases.

The boundary contours in the contour plots shown in Appendix B
show clearly the uniformity of safety factors at the boundary of the
area of analysis. The uniformity of the safety factor contours show
that boundary effects in this finite element model is almost non-exist-
ent. It can also be noticed that the high safety factor region in the
middle of the interseam which is to be expected gets thinner with every
increase in the seam interval. The low safety factors, which occur at
the edges of the opening, is also a manifestation of the high stress
concentrations around that region which contribute greatly toward shear
failure in that region.

6.1.2 Model II

This model has an 80 feet by 9.5 feet opening in the lower seam
and also a 80 feet by 4 feet opening in the upper one. Basically, this
model was to simulate the simultaneous mining of contiguous seams and for a check on the failure patterns. The critical failure surface of this model is as shown in Figure 51. The critical failure surface here is indicated by the 20 safety factor contour line.

The critical failure surface in this case is clearly manifested in that the break from the lower seam to the upper seam can be noticed easily. The failure angle obtained is about 80° to the horizontal. The volume of rock involved in the failure region in this case is quite greater than that shown in Model I. The slabbing of the rib of the opening in the lower seam is also noticed.

The boundaries to this model have fairly high safety factor values except that they are not uniformly looking contours as usually expected. That may be due to stress redistribution, with higher stresses occurring in the areas of the opening.

6.1.3 Model III

This model was constructed to investigate the many recommendations made by workers like, Peng and Chandra (1980), Hasler (1951) and Zachar (1952) that any pillars left in a multi-seam mine should be columnized. The critical failure surface obtained from this model is shown in Figure 52. The 100 safety factor contour is shown to illustrate further the fracture pattern.

The plot obtained from the model justifies the rationale behind the recommendation of columnization of pillars in a multi-seam mine. Due to the many high safety factors encountered in this model study, the contour interval was increased to 50 and thus the lowest safety factor
Figure 52. Critical Failure Surface — Model III
contour encountered is 50. It can be realized from the plot that the failure region in this model is very much confined to where the openings are and that pillars left are almost intact in terms of failure.

The pillars in the lower seam seem almost intact and although the pillars left in the upper one tend to be smaller than the original, it has some stability as against those to be described in Models IV and V. Observing carefully the failure that might occur around the openings it can be noticed that there are some high safety factor areas in the middle of any two openings vertically adjacent.

6.1.4 Model IV

Staggered pillars in multi-seam mining was modeled in the remaining meshes to investigate the possibility of leaving the pillars in a staggered formation. It was noted that in the literature these pillar formations had been discouraged (Stemple, 1956), although it was conceivable that the staggered pillars could be stable and not cause much ground control problems if enough time could elapse after the complete excavation of the lower seam before the upper one is mined.

The meshplot for this model has been discussed in Section 5.1.4 and shown in Figure 33. The critical failure surface obtained for this model, however, is shown in Figure 53. The 40 safety factor contour has been added to clearly define the failure surface.

The contour plots obtained in this model analysis confirms the fact that there has been no recommendation for staggered pillar system for multi-seam mining. The critical failure surface, which in this case is the 20 safety factor contour, runs from an upper opening into
Figure 53. Critical Failure Surface — Model IV
a lower one making both openings unstable. Some very undesirable ground control problems can also be noticed quite clearly in this case, the critical failure surface which forms on the ribside of the lower opening can also be a sign of rib slabbing. The kind of stability experienced when the pillars were columnized in the previous test could not be attested to in this model.

6.1.5 Model V

This model which is described in Section 5.1.5 and with its mesh-plot shown in Figure 34, was constructed to observe the differences in the kind of ground control problems it poses as compared to Model IV. The same rationale as described in the previous section was also behind this model's construction.

Observation of the contour plot obtained working with this model, which is shown in Figure 54, shows that ground control problems associated with this model is identical to those experienced with Model IV.

6.2 Physical Model

Two tests were done in this analysis, because of the technical difficulties experienced in the preparation of the models, especially in the creation of layered blocks. These tests and other tests carried out, however, suggest that the design and construction of the loading frame and all the accessories are in good condition and conforming to the design criteria mentioned in Chapter 4.

The two tests showed that when a load of 2000 lb. (387.82 psi) was applied in the horizontal direction a similar load (i.e., 2000 lb.) in
the vertical direction was needed to break the model. The model also broke initially in the vertical direction in the end with the shorter distance to the hole in the pattern expected from the computer and literature study (Figure 55 and Figure 1). A photograph could not be taken in the initial stages of the model failure but Figure 55 represents what happened initially.
Figure 55. Initial Fracture Pattern of Model Test
CHAPTER VII

SUMMARY

The case studies carried out in this investigation prove that when pillars of the upper seam are superimposed on the pillars left in the lower seam, greater stability in the overlying seam is achieved. Investigating cases where the underlying seam has been mined prior to mining the upper seam, it was found that less disturbance occurs in the upper seam when the innerburden is thick. It was also found that the extraction ratio has to be small when the innerburden is not thick or when the nature of the interseam is weak, if stability in both seams is to be maintained. The case studies also revealed that isolated pillars left in the lower seam act as an unsupported cantilever beam which can bend 100 to 300 feet before fracturing in longwall mining.

Most of the energy and time of this research have been devoted to the design and construction of a loading frame for the physical modeling aspect of this study. Two samples were tested and they exhibited the same shearing pattern as was found from the literature and the computer model studies. A loading frame capable of testing a model block of 24 inches by 24 inches by 6 inches, with a maximum pressure of 10,000 psi, was however, designed and constructed.

Computer analysis was performed on a multi-seam mine modeled to simulate as closely as possible the situation experienced in Figure 1. Five different models were constructed to attempt to study all the conditions to be expected in a multi-seam mine with the underlying seam
mined first prior to the extraction in the overlying seam. Different tests were also carried out on the first model (Model I) such as the investigation of the effect of modulus, Poisson's ratio, density, cohesion and the angle of internal friction of the innerburden on its failure. The critical failure surface obtained using the lowest safety factor contour shows a shear failure line approximately 70° to the horizontal, which confirms what has been observed in the field.

When the modulus of the innerburden was increased systematically it was found that failure in the upper seam also increases. Increasing the Poisson's ratio of the innerburden also increased the extent of the failure in the upper seam significantly. There was a cluster of high safety factors in the middle section of the innerburden when increasing the Poisson's ratio and this was attributed to the high degree of plasticity and the high lateral stresses which come into effect in this test. The density of the innerburden also has an effect on the failure in the upper seam. It was inferred from the tests that the weight of the innerburden comes into play in the failure of the upper seam after $\rho_{\text{shale}}/\rho_{\text{coal}}$ is greater than 1.4. The frictional factors—cohesion and angle of internal friction—were found to have no significant effect on the failure in the upper seam when the lower seam is mined first.

The effect of proximity of seams on the failure in the upper seam was investigated using innerburden thicknesses of 29.5 feet, 49.5 feet, 69.5 feet, 89.5 feet and 119.5 feet (Model I was used in all these cases). The computer analysis showed that the mined lower seam ceases to be a significant factor in the failure of the upper seam after the
innerburden thickness exceeds 52 feet.

When an opening was constructed in the upper seam, it was found from the computer analysis that the critical failure surface ran from the lower seam into the upper one; and also slabbing of the ribs of the openings were noticed. Columnization of pillars in a multi-seam mining has been recommended by many workers and it was found in this study the best way to leave pillars in a multi-seam mining. The pillars in the lower seam seems almost intact with slight robbing of pillars in the upper seam; the relative stability experienced superimposing pillars in multi-seam mines explains why it is recommended by most workers in this area. Instability in the pillars left in both seams was experienced when staggered pillars were modeled and analyzed using the computer.
8.1 Conclusions

Three methods of analysis have been used in the study of the mechanics of interseam failure when a lower seam is mined first prior to the mining of the upper seam. The three methods are: computer modeling, case studies from the literature and physical modeling. The following conclusions were drawn from the above methods of analysis:

1. Using the material parameters in Table 5, which are very realistic for mines in the Appalachian Coalfields, the finite element method used suggests that shear failure is most likely when the innerburden is less than 52 feet. This conclusion is also consistent with what has been observed in the Appalachian Coal region (Hasler, 1951; Lazer, 1965).

2. As the modulus of the innerburden increases, the tendency to fracture through to the upper seam becomes greater. This finding implies that, although massive formations such as sandstones are preferred in the innerburden, these may result in extensive fracture planes under some mining conditions.

3. The computer analysis also shows that as the Poisson's ratio of the innerburden increases the failure in the upper seam also increases. Thus, shear failure through to the upper seam is dependent on the lateral stresses and some degree of plasticity of the innerburden.
4. Increase in the density of the innerburden greatly increases the innerburden instability due to shear failure. This was expressed in the computer analysis when density was varied.

5. Cohesion and angle of internal friction (frictional factors) in the innerburden have no significant effect on the stability of the upper seam.

6. When a room and pillar method of mining is used in the extraction of the seams in a multiple seam mine, the pillars in the upper seam should be superimposed on the pillars left in the lower seam. The investigations in the field and the computer analysis have found this consideration to be very important for stability in the upper seam in this case. Thus, staggering of pillars should be discouraged.

7. Longwall panels in the upper seam when placed in the de-stressed zones created by the mining of the lower seam minimizes damage in the overlying bed.

8.2 Recommendations

More tests on the physical model with a particular mine and its conditions in mind would be very helpful to this study. This will invariably include a study of similitude conditions and all the different scale factors pertaining to the mine.

Further measurements on the physical model such as displacements (convergence) in the openings will also be very helpful if complete study is to be achieved in the laboratory. Linear Variable Differential Transformers (LVDT) in conjunction with the Data Acquisition System already acquired could be used to monitor the displacements in
the model during the tests.

Further work on making the POR-ROK Cement/sand model into layers for close modeling of situations in the field needs to be done. This might be achieved by making the model in slabs and placing them in the frame with an adhesive.

Finally, further field studies can be conducted to study the effect of time on the interseam failure.
REFERENCES


Jones, D. C. "Morris Creek is Now a 3-Seam Producer," Coal Mining and Processing, March 1969, pp. 32-36.


APPENDIX A

MODIFIED FINITE ELEMENT PROGRAM USED
IN THE COMPUTER ANALYSIS
**Introduction to the Finite Element Method - Desai-Abel**

**Example Code (Program) for Plane Strain/Stress**

**Modified for Stability Analysis by Eddie Barko**

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**Logical Out**

```
* **dimension title(18)**
* common nnp,nel,nmat,nslc,ndpt,nbody,mtyp,
  ie(6),pr(6),rd(6),th(6),ie(1500,5),
  2x(1500),y(1500),ulx(1500),vly(1500),kde(1500),isc(80),jsc(80),
  3surtr(80,2),surtry(80,2),cohes(6),phi(6)
* common/one/ qk(10,10),q(10),b(3,10),c(3,3),bt(3,6),xq(5),yq(5)
* common/two/iband,neq,r(3000),ak(3000,140),xxs(600)
* data maxel, maxnp, maxmat, maxbw, maxslc
  1/1500,1500,6,140,80/
```
1 CONTINUE
   IBAND = 2*(MAXDIF + 1)
   NEQ = 2*NRP
   IF(IBAND.GT.MAXBW) GO TO 900
   IF(ISTOP.GT.0) GO TO 999
   CALL ASEMBL(ISTOP)
   IF(ISTOP.GT.0) GO TO 999
C
C TRIANGULARIZE STIFFNESS MATRIX, EQ. (2-2), KKK=1
   CALL BANSOL(1,AK,R,NEQ,IBAND,MAXDOF,MAXBW)
C
C SOLVE FOR DISPLACEMENTS CORRESP. TO LOAD VECTOR R, EQ.(2-3), KKK=2
   CALL BANSOL(2,AK,R,NEQ,IBAND,MAXDOF,MAXBW)
   PRINT 300, (I, R(2*I-1),R(2*I),I=1,NRP)
   WRITE(7,3922)(X(I),Y(I),R(2*I-1),R(2*I),I=1,NRP)
   3922 FORMAT(2F10.3,2E15.7,I10)
C
   CALL STRESS
   GO TO 9999
900 PRINT 901, IBAND, MAXBW
   GO TO 9999
100 FORMAT(15,1X,18A4)
200 FORMAT(/8H1PROBLEM,15,3H..,18A4/)
300 FORMAT(37H1OUTPUT TABLE 1.. NODAL DISPLACEMENTS //
   1   13X,4HNODE, 9X, 11HU = X-DISP.,9X,11HV = Y-DISP./
   2   (5X,112,2E20.8))
901 FORMAT(/12H BANDWIDTH =,I4,25H EXCEEDS MAX. ALLOWABLE =,I4//
   1   30H GO ON TO NEXT PROBLEM )
999 STOP
END
C********************************************************************
C SUBROUTINE DATAIN
C********************************************************************

SUBROUTINE DATAIN(MAXEL,MAXNP,MAXMAT,MAXSLC,ISTOP)
COMMON NNP,NEL,NMAT,NSLC,NOPT,NBODY,MTYP,
IE(6),PR(6),RO(6),TH(6),IE(1500,5),
2X(1500),Y(1500),ULX(1500),VLY(1500),KODE(1500),ISC(80),JSC(80),
3SURTRY(80,2),SURTRX(80,2),COHES(6),PHI(6)

ISTOP = 0
READ 1,NNP,NEL,NMAT,NSLC,NOPT,NBODY

PRINT 100,NNP,NEL,NMAT,NSLC,NOPT,NBODY

C CHECKS TO BE SURE INPUT DATA DOES NOT EXCEED STORAGE CAPACITY
IF(NNP.LE.MAXNP) GO TO 201
ISTOP = ISTOP + 1
PRINT 251, MAXNP
201 IF(NEL.LE.MAXEL) GO TO 202
ISTOP = ISTOP + 1
PRINT 252, MAXEL
202 IF(NMAT.LE.MAXMAT) GO TO 203
ISTOP = ISTOP + 1
PRINT 253, MAXMAT
203 IF(NSLC.LE.MAXSLC) GO TO 204
ISTOP = ISTOP + 1
PRINT 254, MAXSLC
204 IF(ISTOP.EQ.0) GO TO 205
PRINT 255, ISTOP
STOP

205 READ 2, (E(I),PR(I),RO(I),TH(I),I=1,NMAT)
PRINT 101
PRINT 51, (I,E(I),PR(I),RD(I),TH(I), I=1,NMAT)

C
C READ IN COHESION AND PHI VALUES OF THE MATERIALS
C
WRITE (6,505)
DO 10 I=1,NMAT
READ (5,500)(COHES(I),PHI(I))
WRITE (6,510)(COHES(I),PHI(I),I)
500 FORMAT (2F10.2)
510 FORMAT (2F10.2,115)
10 CONTINUE
505 FORMAT (2X,8HCOHESION,7X,3HPHI,4X,11HMATERIAL NO)

C
C READ AND PRINT NODAL DATA (REF. 1)
PRINT 103
N=1
5 READ 3, M,KODE(M),X(M),Y(M),ULX(M),VLY(M)
  IF(M-N)=6,7
4 PRINT 105, M
PRINT 52,M,KODE(M),X(M),Y(M),ULX(M),VLY(M)
  ISTOP= ISTOP +1
  GO TO 5
7  DF = M + 1 - N
6 PRINT 52,N,KODE(N),X(N),Y(N),ULX(N),VLY(N)
  RX=(X(M)-X(N-1))/DF
  RY=(Y(M)-Y(N-1))/DF
  KODE(N)=0
  X(N)=X(N-1)+RX
  Y(N)=Y(N-1)+RY
  ULX(N)=0.0
  VLY(N)=0.0
  GO TO 5

EFE00960  EFE00970  EFE00980  EFE00990  EFE01000  EFE01010  EFE01020  EFE01030  EFE01040  EFE01050  EFE01060  EFE01070  EFE01080  EFE01090  EFE01100  EFE01110  EFE01120  EFE01130  EFE01140  EFE01150  EFE01160  EFE01170  EFE01180  EFE01190  EFE01200  EFE01210  EFE01220  EFE01230  EFE01240  EFE01250  EFE01260  EFE01270
N=N+1
IF(M-N)9,6,8
9 IF(N.LE.NNP) GO TO 5
C READ AND PRINT ELEMENT PROPERTIES, TABLE 6-4
PRINT 106
13 L=0
14 READ 15,M,(IE(M,I),I=1,5)
16 L=L+1
IF(M-L)117,17,18
117 PRINT 118,M
PRINT 53,M,(IE(M,I),I=1,5)
   ISTOP=ISTOP+1
   GO TO 14
18 IE(L,1)=IE(L-1,1)+1
   IE(L,2)=IE(L-1,2)+1
   IE(L,3)=IE(L-1,3)+1
   IE(L,4)=IE(L-1,4)+1
   IE(L,5)=IE(L-1,5)
17 PRINT 53,L,(IE(L,I),I=1,5)
   IF(M-L)20,20,16
20 IF(NEL-L)21,21,14
21 CONTINUE
C
C READ AND PRINT SURFACE LOADING (TRACTION) CARDS
   IF(NSLC.EQ.0) GO TO 31
30 PRINT 108
   DO 40 L=1,NSLC
      READ 41,ISC(L),JSC(L),SURTRX(L,1),SURTRX(L,2),SURTRY(L,1),
         SURTRY(L,2)
40 PRINT 42,ISC(L),JSC(L),SURTRX(L,1),SURTRX(L,2),SURTRY(L,1),
         SURTRY(L,2)
31 IF(ISTOP.EQ.0) GO TO 999
PRINT 900, ISTOP

1 FORMAT(615)  
100 FORMAT(35HINPUT TABLE 1.. BASIC PARAMETERS //  
1   5X, 40H NUMBER OF NODAL POINTS. . . . . . . . . . . . ,15/  
2   5X, 40H NUMBER OF ELEMENTS. . . . . . . . . . . . . ,15/  
3   5X, 40H NUMBER OF DIFFERENT MATERIALS . . . . . . . ,15/  
4   5X, 40H NUMBER OF SURFACE LOAD CARDS. . . . . . . ,15/  
5   5X, 40H 1 = PLANE STRAIN, 2 = PLANE STRESS. . . . . ,15/  
6   5X, 40H BODY FORCES(1 = IN -Y DIREC., 0 = NONE),15)  
251 FORMAT(////33H TOO MANY NODAL POINTS, MAXIMUM = ,15)  
252 FORMAT(////30H TOO MANY ELEMENTS, MAXIMUM = ,15)  
253 FORMAT(////30H TOO MANY MATERIALS, MAXIMUM = ,15)  
254 FORMAT(////40H TOO MANY SURFACE LOAD CARDS, MAXIMUM = ,15)  
255 FORMAT(////28H EXECUTION HALTED BECAUSE OF,15,13H FATAL ERRORS/)  
2 FORMAT(4E10.3)  
101 FORMAT(36HINPUT TABLE 2.. MATERIAL PROPERTIES //  
1   10H MATERIAL,5X,10H MODULUS OF,6X,9HPOISSON'S,7X,  
     28HMATERIAL,7X,  8HMATERIAL /  
     34X,6HNUMBER,5X,10HELASTICITY,8X,7H RATIO,8X,7HDENSITY,6X,  
     49HTHICKNESS )  
51 FORMAT(110,4E15.4)  
103 FORMAT(34HINPUT TABLE 3.. NODAL POINT DATA //  
1   5X,5H NODAL,48X,7X-DISP.,8X,7Y-DISP./  
25X,5HPNT,6X,4HTYPE,14X,1HX,14X,1HY,8X,7HLOAD,8X,7HLOAD)  
3 FORMAT(215,4E10.3)  
105 FORMAT(5X,17HERROR IN CARD NO.,15/)  
52 FORMAT(2110,4E15.4)  
106 FORMAT(34HINPUT TABLE 4.. ELEMENT DATA  //  
1   11X,31HGLOBAL INDICES OF ELEMENT NODES/3X,7HELEMENT,  
     27X,1H1,7X,1H2,7X,1H3,7X,1H4,2X,8HMATERIAL)  
118 FORMAT(5X, 25HERROR IN ELEMENT CARD NO.,15/)
15 FORMAT(615)
  53 FORMAT(I10,4I8,I10)
108 FORMAT(I17H1INPUT TABLE 5.. SURFACE LOADING DATA //
    117X, 33HSURFACE LOAD INTENSITIES AT NODES/
    24X,6HNODE I,4X,6HNODE J,10X,2HXI,10X,2HXJ,10X,2HYI,10X,2HYJ)
  41 FORMAT(215,4E10.3)
  42 FORMAT(2110,4E12.4)
  900 FORMAT(/ //45H ASSEMBLY AND SOLUTION WILL NOT BE PERFORMED,,I5,
    121H FATAL CARD ERRORS    )
  999 RETURN
         END
C****************************************************************************************************
C SUBROUTINE ASEMBL                          *
C****************************************************************************************************
C SUBROUTINE ASEMBL(ISTOP)
COMMON NNP, NEL, NMAT, NSLC, NOPT, NBODY, MTYP,
  LE(6), PR(6), RO(6), TH(6), IE(1500, 5),
  2X(1500), Y(1500), ULX(1500), VLY(1500), KDE(1500), ISC(80), JSC(80),
  SURTRX(80, 2), SURTRY(80, 2)
COMMON/ONE/ QK(10, 10), Q(10), B(3, 10), C(3, 3), BT(3, 6), XQ(5), YQ(5)
COMMON/TWO/ IBAND, NEQ, R(3000), AK(3000, 140)
DIMENSION LP(8)
C
C REWIND 1
C INITIALIZE
ISTOP = 0
C INITIALIZE PARTS OF MATRICES C AND BT
BT(1, 4) = 0.0
BT(1, 5) = 0.0
BT(1, 6) = 0.0
BT(2, 1) = 0.0
BT(2, 2) = 0.0
BT(2, 3) = 0.0
C(1, 3) = 0.0
C(2, 3) = 0.0
C(3, 1) = 0.0
C(3, 2) = 0.0
C INITIALIZE OVERALL STIFFNESS MATRIX AK AND OVERALL LOAD VECTOR R
DO 2 I = 1, NEQ
R(I) = 0.0
DO 2 J = 1, IBAND
2  AK(I, J) = 0.0
C
C COMPUTE ELEMENT STIFFNESSES AND LOADS ONE BY ONE
C
DO 10 M=1,NEL
IF(IE(M,5).GT.0) GO TO 11
   ISTOP = ISTOP + 1
   GO TO 10
11 CALL QUAD(M,AREA)
   IF(AREA.GT.0.0) GO TO 16
      ISTOP = ISTOP + 1
   PRINT 20,M
C
C CONDENSE ELEMENT STIFFNESS FROM 10X10 TO 8X8, EQ.(5-64), AND ELEMENT
C LOADS FROM 10X1 TO 8X1, EQ.(5-640). (REF.2)
16 IF(IE(M,3).EQ.IE(M,4)) GO TO 26
   DO 31 J = 1,2
      IJ= 10-J
      IK = IJ+1
      PIVOT = QK(IK,IK)
   DO 32 K = 1,IJ
      F = QK(IK,K)/PIVOT
      QK(IK,K)=F
   DO 33 I=K,IJ
      QK(I,K)=QK(I,K)- F*QK(I,IK)
   33 QK(K,I) = QK(I,K)
   32 Q(K) = Q(K)-QK(IK,K)*Q(IK)
   31 Q(IK) = Q(IK)/PIVOT}
C
C STORE MULTIPLIERS, PIVOTS, CONDENSED LOADS, STRAIN-DISP. AND STRESS-STRAEFE02650
C MATRICES ON SCRATCH TAPE NO. 1 (TO BE USED LATER TO COMPUTE STRAINS ANEFE02660
C STRESSES )
C
EFE02360
EFE02370
EFE02380
EFE02390
EFE02400
EFE02410
EFE02420
EFE02430
EFE02440
EFE02450
EFE02460
EFE02470
EFE02480
EFE02490
EFE02500
EFE02510
EFE02520
EFE02530
EFE02540
EFE02550
EFE02560
EFE02570
EFE02580
EFE02590
EFE02600
EFE02610
EFE02620
EFE02630
EFE02640
EFE02650
EFE02660
EFE02670
26 WRITE (1) ((QK(I,J),I=1,10),J=1,10), Q(9), Q(10),
1((B(I,J),J=1,10),I=1,3), ((C(I,J),I=1,3),J=1,3), XQ(5), YQ(5)
C
C ASSEMBLE STIFF. AND LOADS, DIRECT STIFF. METHOD, SEC. 6-5.
C
      LIM=8
      IF(E(M,3),EQ.E(M,4)) LIM = 6
      DO 40 I=2,LIM,2
         IJ = I/2
         LP(I-1) = 2*IE(M,IJ) - 1
         LP(I) = 2*IE(M,IJ)
      DO 50 LL=1,LIM
         I = LP(LL)
         R(I) = R(I) + Q(LL)
      DO 50 MM=1,LIM
         J = LP(MM) - I + 1
         IF(J.LE.0) GO TO 50
         AK(I,J) = AK(I,J) + QK(LL,MM)
      50 CONTINUE
      40 CONTINUE
      10 CONTINUE
C
C ADD EXTERNALLY APPL. CONC. NODAL LOADS TO R
      DO 55 N=1,NNP
         IF(KODE(N),EQ.3) GO TO 55
         K=2*N
         IF(KODE(N),EQ.1) GO TO 57
         R(K-1) = R(K-1) + ULX(N)
      IF(KODE(N),NE.0) GO TO 55
      57 R(K) = R(K) + VLY(N)
      55 CONTINUE
C
C CONVERT LINEARLY VARYING SURFACE TRACTIONS TO STATIC EQUIVALENTS,
C AND ADD TO OVERALL LOAD VECTOR R, EQ.(5-61A).
   IF(NSLC.EQ.0) GO TO 60
   DO 61 L = 1,NSLC
      I = ISC(L)
      J = JSC(L)
      II=2*I
      JJ=2*J
      DX = X(J) - X(I)
      DY = Y(J) - Y(I)
      EL = SQRT(DX**2 + DY**2)
      PXI=URTRX(L,1)*EL
      PXJ=URTRX(L,2)*EL
      PYI=URTRY(L,1)*EL
      PYJ=URTRY(L,2)*EL
      R(II-1)=R(II-1)+PXI/3.0 + PXJ/6.0
      R(JJ-1)=R(JJ-1)+PYI/6.0 + PYJ/3.0
      R(II)=R(II)+PYI/3.0 + PYJ/6.0
      R(JJ)=R(JJ)+PYI/6.0 + PYJ/3.0
   CONTINUE
   61
C INTRODUCE KINEMATIC CONSTRAINTS (GEOMETRIC BOUNDARY CONDITIONS),
C EQ.(6-18).  REF. 1.

C DO 70 M=1,NNP
   IF(KODE(M).GE.0.AND.KODE(M).LE.3) GO TO 72
      ISTOP = ISTOP + 1
   GO TO 70
   72
   IF(KODE(M).EQ.0) GO TO 70
   IF(KODE(M).EQ.2) GO TO 71
   CALL GEOMBC(ULX(M),2*M-1)
   IF(KODE(M).EQ.1) GO TO 70
   71
   CALL GEOMBC(VLY(M),2*M)
70 CONTINUE
ENDFILE 1
     IF(ISTOP.EQ.0) GO TO 81
PRINT 100, ISTOP
20 FORMAT(/5X,17H AREA OF ELEMENT ,I5,I4H IS NEGATIVE //)
100 FORMAT(////42H SOLUTION WILL NOT BE PERFORMED BECAUSE OF ,I5,
     1 15H DATA ERRORS //)
81 RETURN
END
C***************************************************************
C SUBROUTINE QUAD
C***************************************************************
SUBROUTINE QUAD(M,TOTALA)
COMMON NNP,NEL,NMAT,NSLC,NOPT,NBODY,MTYP,
1E(6),PR(6),RO(6),TH(6),IE(1500,5),
2X(1500),Y(1500),ULX(1500),VLY(1500),KODE(1500),ISC(80),JSC(80),
3SURTRX(80,2),SURTRY(80,2)
COMMON/ONE/ QK(10,10),Q(10),B(3,10),C(3,3),BT(3,6),XQ(5),YQ(5)
COMMON/TWO/IBAND,NEQ,R(3000),AK(3000,140)
C
I= IE(M,1)
J= IE(M,2)
K= IE(M,3)
L= IE(M,4)
MTYP = IE(M,5)
TOTALA = 0.0
C
C CONSTRUCT STRESS-STRAIN MATRIX C, EQ.(3-16C). FOR PLANE STRAIN
C NOPT=1, AND FOR PLANE STRESS NOPT=2. PRESENT CODE IS FOR
C ISOTROPIC MATERIALS
IF(NMAT.EQ.1.AND.M.GT.1) GO TO 5
IF(NOPT.EQ.2) GO TO 2
CF = E(MTYP)/((1.0+PR(MTYP))*(1.0-2.0*PR(MTYP)))
C(1,1)= CF*(1.0-PR(MTYP))
C(1,2)= CF* PR(MTYP)
C(2,1)= C(1,2)
C(2,2)= C(1,1)
C(3,3)= CF*(1.0-2.0*PR(MTYP))/2.0
GO TO 5
2
CF = E(MTYP)/(1.0-PR(MTYP)*PR(MTYP))
C(1,1) = CF

158
C(1,2) = PR(MTYP)*CF
C(2,1) = C(1,2)
C(2,2) = CF
C(3,3) = CF*(1.0-PR(MTYP))/2.0

LIM = 4

IF(K.EQ.L) LIM = 3
XQ(5) = 0.0
YQ(5) = 0.0

DO 10 N=1,LIM
    NN = IE(M,N)
    XQ(N) = X(NN)
    YQ(N) = Y(NN)
    XQ(5) = XQ(5) + X(NN)/FLOAT(LIM)
    YQ(5) = YQ(5) + Y(NN)/FLOAT(LIM)
10

C INITIALIZE QUAD. STIFFNESS, LOAD VECTOR AND STRAIN-DISPLACEMENT VECTOR

DO 13 JJ = 1,10
    Q(JJ) = 0.0
13
DO 12 JJ = 1,10
    QK(JJ,JJ) = 0.0
12

DO 13 JJ = 1,3
    B(JJ,II) = 0.0
13

IF(K.EQ.L) GO TO 15

CALL CST(1,2,3,TOTALA)
    GO TO 999
15 CALL CST(1,2,5,AREA)
    TOTALA = TOTALA + AREA
CALL CST(2,3,5,AREA)
    TOTALA = TOTALA + AREA
CALL CST(3,4,5,AREA)
    TOTALA = TOTALA + AREA
CALL CST(4,1,5,AREA)
    TOTALA = TOTALA + AREA
    EFE03740
    EFE03750
    EFE03760
    EFE03770
    EFE03780
    EFE03790
    EFE03800
    EFE03810
    EFE03820
    EFE03830
    EFE03840
    EFE03850
    EFE03860
    EFE03870
    EFE03880
    EFE03890
    EFE03900
    EFE03910
    EFE03920
    EFE03930
    EFE03940
    EFE03950
    EFE03960
    EFE03970
    EFE03980
    EFE03990
    EFE04000
    EFE04010
    EFE04020
    EFE04030
    EFE04040
    EFE04050
TOTALA = TOTALA + AREA

999 RETURN
END
C******************************************************************************
C SUBROUTINE CST
C******************************************************************************
C SUBROUTINE CST(I,J,K,AREA)
COMMON NNP,NEL,NMAT,NSLC,NOPT,NBODY,MTYP,
  1E(6),PR(6),RO(6),TH(6),IE(1500,5),
  2X(1500),Y(1500),ULX(1500),VLY(1500),KODE(1500),ISC(80),JSC(80),
  3SURTRX(80,2),SURTRY(80,2)
COMMON/ONE/ QK(10,10),Q(10),B(3,10),C(3,3),BT(3,6),XQ(5),YQ(5)
COMMON/TWO/IBAND,NEQ,R(3000),AK(3000,140)
DIMENSION CB(3,6),LC(6),LT(3),TK(6,6)

C
LT(1)= I
LT(2)= J
LT(3)= K

C COMPUTE STRAIN-DISPLACEMENT MATRIX B FOR TRIANGLE, EQ. (5-35A)
BT(1,1)= YQ(J)-YQ(K)
BT(1,2)= YQ(K)-YQ(I)
BT(1,3)= YQ(I)-YQ(J)
BT(2,4)=XQ(K)-XQ(J)
BT(2,5)= XQ(I)-XQ(K)
BT(2,6)= XQ(J)-XQ(I)
BT(3,1)=BT(2,4)
BT(3,2)= BT(2,5)
BT(3,3)= BT(2,6)
BT(3,4)= BT(1,1)
BT(3,5)= BT(1,2)
BT(3,6)= BT(1,3)
AREA =(BT(2,4)*BT(1,3) - BT(2,6)*BT(1,1))/2.0

C COMPUTE C*B
DO 10 II = 1,3
DO 10 JJ = 1,6
  CB(II, JJ) = 0.0
DO 10 KK = 1,3
  CB(II, JJ) = CB(II, JJ) + C(II, KK) * BT(KK, JJ)
10
C
C COMPUTE (B**T)*C*B, EQ.(5-45A)
DO 12 II = 1,6
DO 12 JJ = 1,6
  TK(II, JJ) = 0.0
DO 12 KK = 1,3
  TK(II, JJ) = TK(II, JJ) + BT(KK, II) * CB(KK, JJ)
12
C
C ADD TRIANGLE STIFFNESS TO QUADRILATERAL STIFFNESS, EX.(6-2).
C ADD TRIANGLE STRAIN-DISPLACEMENT MATRIX TO QUADRILATERAL STRAIN-
C DISPLACEMENT MATRIX
DO 15 II = 1,3
  LC(II) = 2 * LT(II) - 1
15
DO 30 II = 1,6
  LL = LC(II)
  IF(AREA.EQ.0.0) GO TO 999
  FK = 1.0/(4.0*AREA)
  FB = 2.0*FK
DO 20 JJ = 1,6
  MM = LC(JJ)
20
  QK(LL, MM) = QK(LL, MM) + TK(II, JJ) * TH(MTYP) * FK
DO 30 JJ = 1,3
  B(JJ, LL) = B(JJ, LL) + BT(JJ, II) * FB
30
C
C DEVELOP BODY FORCE VECTOR, EQ.(5-61B)
IF(NBODY.EQ.0) GO TO 999
TBODF = AREA* RO(MTYP)* TH(MTYP)
BODYF = -TBODF/3.0
DO 35 II=1,3
    JJ = 2* LT(II)
    Q(JJ) = Q(JJ) + BODYF
35
999 RETURN
END
C******************************************************************************
C SUBROUTINE STRESS
C******************************************************************************

SUBROUTINE STRESS
COMMON NNP, NEL, NMAT, NSLC, NOPT, NBODY, MTYP,
1E(6), PR(6), RO(6), TH(6), IE(1500, 5),
2X(1500), Y(1500), U(LX(1500), VLY(1500), KDE(1500), ISC(80), JSC(80),
3SURTRX(80, 2), SURTRY(80, 2), COHES(6), PHI(6)
COMMON/ONE/ QK(10, 10), Q(10), B(3, 10), C(3, 3), BT(3, 6), XQ(5), YQ(5)
COMMON/TWO/IBAND, NEQ, R(3000), AK(3000, 140)
DIMENSION SIG(6), SIGI(3)

C REWIND 1
PRINT 300
NOLINE = 47

C RETRIEVE MULTIPLIERS, PIVOTS, MATRICES B AND C, AND CENTROIDAL COORD.
C FOR ELEMENT
DO 5 M = 1, NEL
READ(1) (QK(I,J), J=1,10), I=1,2, Q(9), Q(10),
1 (B(I,J), J=1,10), I=1,3, (C(I,J), J=1,3), I=1,3), XC, YC

C SELECT NODAL DISPLACEMENTS FOR THE ELEMENT
LIM = 4
IF(IE(M, 3) .EQ. IE(M, 4)) LIM = 3
DO 10 I = 1, LIM
II = 2*I
JJ = 2*IE(M, I)
Q(I-1) = R(JJ-1)*
10 Q(I) = R(JJ)

C RECOVER CONDENSED DISPLACEMENTS FOR THE QUADRILATERAL, EQ. (5-64G)
IF(LIM.EQ.3) GO TO 16
DO 15 K=1,2
   JK = K + 8
   IK = JK - 1
DO 15 L=1,IK
15 Q(JK) = Q(JK) - Q(K,L)*Q(L)
C
C COMPUTE ELEMENT STRAINS, EQ.(5-35A)
   LIM = 10
   FAC = 0.25
GO TO 17
16 LIM = 6
   FAC = 1.0
17 DO 20 I=1,3
   E(I) = 0.0
DO 20 J=1, LIM
20 E(I) = E(I) + B(I,J)*Q(J)*FAC
C
C COMPUTE ELEMENT STRESSES , EQ.(5-35B)
DO 30 I=1,3
   SIG(I) = 0.0
DO 30 J=1,3
30 SIG(I) = SIG(I) + C(I,J)*E(J)
C COMPUTE PRINCIPAL STRESSES AND THE ANGLE WITH THE POSITIVE X AXIS
   SP = (SIG(1)+SIG(2))/2.0
   SM = (SIG(1)-SIG(2))/2.0
   DS = SQRT(SM*SM+SIG(3)*SIG(3))
   SIG(4) = SP + DS
   SIG(5) = SP - DS
   SIG(6) = 0.0
   IF(SIG(3).NE.0.0.AND.SM.NE.0.0) SIG(6) = 28.648*ATAN2(SIG(3),SM)
1
C COULOMB FAILURE CRITERION (REFERENCE WANG, PANEK & SUN)
C THIS PROGRAM TAKES INTO ACCOUNT THE FACT THAT AT ANY POINT ON THE
C FRACTURE SURFACE, THE TANGENT IS ORIENTED SUCH THAT THE QUANTITY
C -(MU*SIGNML)* IS A MAXIMUM AT AN ANGLE, SIG(6), FROM THE
C MAJOR PRINCIPAL STRESS DIRECTION. (OBERT & DUVALL P. 303)
C
C ANGLE=0.0174533*PHI(MTYP)
COEFIN=TAN(ANGLE)
ADANGL=1/COEFIN
EPHI=1/2*ATAN(ADANGL)
ADPHI=0.0174533*SIG(6)+EPHI
SIGNML=1/2*(SIG(1)+SIG(2))-(1/2*(SIG(1)-SIG(2))*COS(2*ADPHI))+
1*(SIG(3)*COS(2*ADPHI))
SHSTRS=SIG(3)*COS(2*ADPHI)-(1/2*(SIG(1)-SIG(2))*SIN(2*ADPHI))
SFT1=(COHES(MTYP)+(SIGNML*TAN(ANGLE)))/SHSTRS
SFT=ABS(SFT1)
C PRINT STRESSES, 50 LINES PER PAGE
IF(NOLINE.GT.0) GO TO 54
PRINT 1000
   NOLINE = 49
54   NOLINE = NOLINE - 1
C RE-USE STORAGE SPACE IN UN-NEEDED ARRAYS FOR STRESSES & CENTROIDS
C
C DO 60 JJJ=1,5
ISUB1 = MOD(JJJ+1,2)*120 + M
ISUB2 = (JJJ+1)/2
60  AK(ISUB1,ISUB2) = SIG(JJJ)
STX=SIG(1)/144.
     STY=SIG(2)/144.
     STXY=SIG(3)/144.
STMAX = SIG(4)/144.
STMIN = SIG(5)/144.
WRITE(7,3987) XC,YC,SFT,M
3987 FORMAT(2F10.3,F15.3,I10)
WRITE(6,1010) M,XC,YC,STX,STY,STXY,STMAX,STMIN,SIG(6),SFT
ENDFILE 1
300 FORMAT(47H1OUTPUT TABLE 2.. STRESSES AT ELEMENT CENTROIDS //
11X,7HELEMENT,9X,1HX,9X,1HY,4X,8HSIGMA(X),4X,8HSIGMA(Y),4X,
2BHTAU(X,Y),4X,8HSIGMA(1),4X,8HSIGMA(2), 7X,5HANGLE,
31X,13HSAFETY FACTOR)
1000 FORMAT(1H1, 7HELEMENT,9X,1HX,9X,1HY,4X,8HSIGMA(X),4X,8HSIGMA(Y),
14X,8BHTAU(X,Y),4X,8HSIGMA(1),4X,8HSIGMA(2), 7X,5HANGLE,
22X,13HSAFETY FACTOR)
1010 FORMAT(18, 2F10.2,1P6E12.4,1E12.4)
RETURN
END
C*******************************************************************************
C SUBROUTINE GEOMBC
C*******************************************************************************
SUBROUTINE GEOMBC(U,N)
COMMON/Two/IBAND,NEQ,R(3000),AK(3000,140),XXS(600)
C THIS SUBROUTINE MODIFIES THE ASSEMBLAGE STIFFNESS AND LOADS FOR THE
C PRESCRIBED DISPLACEMENT U AT DEGREE OF FREEDOM N, EQ.(6-18B). (REF.1)
DO 100 M=2,IBAND
   K = N - M + 1
   IF(K.LE.0) GO TO 50
   R(K) = R(K) - AK(K,M)*U
   AK(K,M) = 0.0
50   K = N + M - 1
   IF(K.GT.NEQ) GO TO 100
   R(K) = R(K) - AK(N,M)*U
   AK(N,M) = 0.0
100 CONTINUE
   AK(N,1) = 1.0
   R(N) = U
RETURN
END
C*******************************************************************************
C SUBROUTINE BANSOL
C*******************************************************************************
C*******************************************************************************
SUBROUTINE BANSOL(KKK,AK,R,NEQ,IBAND,NDIM,MDIM)
C SYMMETRIC BAND MATRIX EQUATION SOLVER. (REF. 2)
C
C KKK = 1 TRIANGULARIZES THE BAND MATRIX AK, EQ. (2-2)
C KKK = 2 SOLVES FOR RIGHT HAND SIDE R, SOLUTION RETURNS IN R, EQ.(2-3)
C
DIMENSION AK(3000,140),R(1)
   NRS = NEQ - 1
   NR = NEQ
IF(KKK.EQ.2) GO TO 200
DO 120 N= 1,NRS
   M= N-1
   MR = MIN0(IBAND,NR-M)
   PIVOT = AK(N,1)
DO 120 L=2,MR
   CP= AK(N,L)/PIVOT
   I = M+L
   J = 0
DO 110 K=L,MR
   J = J + 1
   110 AK(I,J) = AK(I,J) -CP*AK(N,K)
   120 AK(N,L) = CP
GO TO 400
200 DO 220 N= 1,NRS
   M= N-1
   MR = MIN0(IBAND,NR-M)
   CP= R(N)
   R(N)=CP/AK(N,1)
DO 220 L=2,MR
EFE06170
EFE06180
EFE06190
EFE06200
EFE06210
EFE06220
EFE06230
EFE06240
EFE06250
EFE06260
EFE06270
EFE06280
EFE06290
EFE06300
EFE06310
EFE06320
EFE06330
EFE06340
EFE06350
EFE06360
EFE06370
EFE06380
EFE06390
EFE06400
EFE06410
EFE06420
EFE06430
EFE06440
EFE06450
EFE06460
EFE06470
EFE06480
I = M + L
R(I) = R(I) - AK(N,L)*CP
R(NR) = R(NR)/AK(NR,1)
DO 320 I = 1,NRS
   N = NR - I
   M = N-1
   MR = MIN(1BAND,NR-M)
   DO 320 K = 2,MR
   L = M+K
C STORE COMPUTED DISPLACEMENTS IN LOAD VECTOR R
320   R(N) = R(N) - AK(N,K)*R(L)
400 RETURN
END
APPENDIX B

SAFETY FACTOR CONTOURS OBTAINED FROM THE ANALYSIS
Figure B1. Safety Factor Contour Plot of Model I ($E_{\text{shale}}/E_{\text{coal}} = 4.0$)
Figure B2: Safety Factor Contour Plot of Model I (Ehale/Ecoal = 4.0)
Figure B3. Safety Factor Contour Plot of Model I \((E_{\text{shale}}/E_{\text{coal}} = 8.0)\)
Figure 32. Safety Factor Contour Plot of Model I ($\gamma_{\text{shale}}/\gamma_{\text{coal}}=1.5$)
Figure B9: Safety Factor Contour Plot of Model I ($\rho_{\text{shale}}/\rho_{\text{coal}}=0.5$)
Figure B10  Safety Factor Contour Plot of Model I ($\rho_{\text{shale}}/\rho_{\text{coal}}=1.0$)
Figure B11  Safety Factor Contour Plot of Model I ($\rho_{\text{shale}}/\rho_{\text{coal}}=1.5$)
Figure B14
Safety Factor Contour Plot of Model I (C_{shale}/C_{coal}=2.0)
Figure B15: Safety Factor Contour Plot of Model I (C_{shale}/C_{coal}=3.0)
Figure B.16 Safety Factor Contour Plot of Model I ($\Phi_{\text{shale}}/\Phi_{\text{coal}}=1.0$)
Figure B17  Safety Factor Contour Plot of Model I (Interseam Distance=49.5 ft.)
Figure B20  Safety Factor Contour Plot of Model I (Interseam Distance = 119.5 ft.)
Figure B21: Safety Factor Contour Plot of Model II
Figure B2.3: Safety Factor Contour Plot of Model IV
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MECHANICS OF INTERSEAM FAILURE IN
MULTIPLE HORIZON MINING

by

Eddie N. Barko

(ABSTRACT)

The mechanics of massive interseam failure in a multiple seam mine was investigated using three approaches: case studies, physical models and computer analysis. Specific examples of multi-seam mines with the underlying seam mined first prior to the mining of the overlying seam were studied with some design guidelines drawn from them. A loading frame capable of testing model blocks of 24 inches by 24 inches by 6 inches and also capable of applying up to a maximum of 10,000 psi of pressure on the models was designed and built.

In this investigation, factors that affect the stability of the overlying seam when the underlying seam is mined first were studied using the finite element method and the Mohr-Coulomb failure criteria. Critical failure surfaces obtained from the computer analysis were analyzed for columnized and staggered pillars in room and pillar mining with the columnized pillars favored over the staggered ones.