A PHOTOELASTIC INVESTIGATION OF THE RELATIONSHIP BETWEEN STRESSES AROUND MINE OPENINGS AND RESULTING FAILURES

BY

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ABSTRACT

The primary purpose of this investigation is to determine, by photoelastic analyses, the relation between stresses around underground openings and the failure resulting therefrom.

Models were made of $\frac{1}{4}$-in. plates of Columbia Resin CR-39, also known as Allite. The openings studied were: a 2 x 7-yd, rectangular, horizontal prototype opening with sharp corners; a 2 x 7-yd, rectangular, prototype opening with sharp corners and with its long axis 45° from the horizontal; a 4-yd, circular prototype opening; and a triangular prototype opening with a 4-yd base and 4-yd sides.

Photoelastic stress patterns were obtained from a circular polariscope. From the stress patterns obtained, the extent of failure was calculated for four different conditions: Case I - failure due to tension alone; Case II - failure due to tension and 1600-psi shear; Case III - failure due to tension and 1200-psi shear; and Case IV - failure due to tension and 800-psi shear.

Each model was subjected to a vertical load representing 900 ft of overburden; the stress pattern was then recorded. From the stress pattern the critical stresses (at which the failure was assumed to occur in each case) were calculated. Thereupon a new model was made having an opening equal to the original opening plus the area assumed to have failed. The new model was then subjected

to a vertical load again, and the procedure was repeated. In this way a series of models was made for each case, thus indicating the extent of failure.

To test the validity of the conclusions obtained from the photoelastic analyses for both stratified rock and solid rock, a number of verifying experiments were performed.

In this thesis the more important rock failure theories are discussed, followed by the author's concept that failure due to shear is of primary importance.

The dome theory, a study of natural arches, some observations of cleavage induced by mining in British coal mines, and a case of swelling of ground are discussed. After each discussion, the author expresses his opinion on the subject backed by conclusions obtained from the experiments described in this thesis. Also, the importance of horizontal support is emphasized as substantiated by findings obtained from the experimental work.

In the following a summary of the conclusions is given, which includes both the findings directly obtained from the experiments and the conclusions indirectly obtained by further reasoning.

1. If rock fails because of tension alone, a natural arch will result, and the walls will not be affected.

2. If rock fails because of both shear and tension, shear failure will be of primary importance.

3. After failure once starts, it ultimately results in the same shape of failed area, independent of the original shape of the opening.

4. Closure of mine openings takes place because of the slippage along shear planes.

5. Horizontal support is as important as vertical support, if not more so.

CONTENTS

INTRODUCTION

The primary purpose of this investigation is to determine, by photoelastic analyses, the relation between stresses around underground mine openings and the failure resulting therefrom.

Models were made of ¼ in. plates of Columbia Resin CR-39, also known as Allite. The openings studied were: a rectangular, horizontal opening with sharp corners; a rectangular opening with sharp corners and with its long axis 45° from the horizontal; a circular opening; and a triangular opening.

From the stress patterns obtained, the extent of failure was calculated for four different conditions: Case I- failure due to tension alone, Case II- failure due to tension and 1600-psi shear, Case III- failure due to tension and 1200-psi shear, and Case IV- failure due to tension and 800-psi shear.

Each model was subjected to a vertical load, and the stress pattern recorded. From the stress pattern the critical stresses (at which the failure was assumed to occur in each case) were calculated. Thereupon a new model was made having an opening equal to the original opening plus the area assumed to have failed. The new model was then subjected to a vertical load again, and the procedure repeated. In this way a series of models was made for each case, thus indicating the extent of failure.

To test the validity of the conclusions obtained from the photoelastic analyses for both stratified rock and solid rock, a number of verifying experiments were performed. A solid beam was subjected to a load and the stress pattern recorded. The beam was then split along the longitudinal axis, and the two resulting beams were subjected to the same load. Comparison of the stress pattern of the solid beam and of the two beams together showed considerable weakening of the beam after it had been split. The two beams were then tied together by means of rods placed in vertical holes drilled through them. The resultant stress distribution was then the same as in the case of the solid beam.

For the same purpose a model with a rectangular opening in a solid plate of Allite was subjected to a vertical load. The same load was applied to models composed of horizontal strips of Allite. The resultant stress patterns in all cases were virtually the same.

Several theories have been advanced in the literature (McCutchen, 1949; Pandya, 1950) to explain the failure of rock. The more important theories are defined later in the introduction and are followed by the author's concept of rock failure as proved later in the thesis.

Following these theories previous photoelastic and mathematic analyses are summarized. The difference between those studies and the study described in this thesis is discussed.

In the following chapter the laboratory work performed by the author is discussed.

In the conclusions, the dome theory, a study of natural arches, observations of cleavage induced by mining in British coal mines, and a case of swelling of ground are discussed. After each discussion, the author expresses his opinion on the subject backed by conclusions obtained from the experiments described in this thesis. Also, in the conclusions, the importance of horizontal support is emphasized as substantiated by conclusions obtained from the experimental work.

ROCK FAILURE THEORIES

In the following, the more important rock-failure theories are defined (McCutchen, 1949; Pandya, 1950). The author's concept of rock failure is discussed thereafter. By failure is meant the inelastic behavior of the material when stressed.

The maximum-stress theory, also known as the Rankine theory, states that failure begins only when the maximum normal stress across any plane exceeds the stress corresponding to the proportional limit of the material in a simple tension or compression test.

The maximum-strain theory, advanced by de St. Venant, postulates that the strain associated with the failure of a material in a simple tension or compression test must not be exceeded if the material is not to fail. For materials that follow Hooke's law, the maximum-stress theory and the maximum-strain theory are identical.

In the <u>maximum-strain-energy theory</u>, developed by Beltrami, by Huber, and later by Haigh, the modulus of resilience is used as the criterion of elastic failure. The maximum strain-energy stored per unit of volume by the body under any complex system of stresses must not exceed that which can be stored per unit volume by the body, as determined by a simple tension or compression test.

The <u>theory of inner friction</u>, developed by Coulomb, is based on the concept of failure by a sliding action within the material, and states that failure occurs whenever the maximum shear stress on any plane exceeds the combined resistance due to the shear strength and frictional resistance.

The <u>maximum-stress-difference or maximum-shear theory</u>, which was proposed by Guest and is a special case of Coulomb's theory, holds that the factor producing failure of elasticity is the greatest shear stress in the material or the greatest difference between the principal stresses.

The maximum-shear theory has been modified by Mohr (1928). Instead of placing reliance on the maximum-shear stress alone as a cause of failure, Mohr states that of all planes having the same normal component, that which sustains the greatest shear stress is the plane on which inelastic strain will occur. Derivations and explanations of Mohr's theory are thoroughly discussed by McCutchen (1949).

The Hanck-von Mises theory, sometimes called the shear-energy or the shear-distortion theory, is expressed in the equation

$$p^2 - pq + q^2 = p_c,$$

where p and q are the two-dimensional stresses and p_c is the critical stress.

The author of this thesis believes that rock will fail readily when subjected to tension. However, when the state of stress is compression the shear strength of the rock is surpassed before the compressive strength (Handin and Fairbairn, 1953). In a mine opening, the roof and floor will fail because of tension; the walls of the opening will fail because of shear. If the failed rock from the walls is allowed to cave, the span of the roof will be increased, thus causing a larger area to be under tension. Therefore, around mine openings, shear failure is of primary importance.

This concept will be proved in this thesis.

PREVIOUS STUDIES

Previous photoelastic studies (Sinclair and Bucky, 1940; Kafadar, 1943; Duvall, 1948; Pandya, 1950; Panek, 1951) dealt mainly with boundary stresses around mine openings. The studies concerned problems of a two-dimensional nature. These investigators started with certain assumptions and obtained an "ideal" mine opening or combination of openings. The "ideal" case was the opening with the smallest resultant tensile stress and the smallest resultant stress concentration at the boundary.

Similar results were obtained by strictly mathematical analyses (Carstarphen, 1933; Fenner, 1938; Mindlin, 1939; Terzaghi, 1943; Symposium on Rock Pressure..., 1947; Mindlin, 1948; Greenspan, 1949; Mohr, 1952).

An excellent review of the different mathematical studies was made by Schoemaker (1948).

The different photoelastic and mathematical studies indicated above differ mainly in their basic assumptions, but, of course, these basic assumptions directly affect the validity of the mathematical interpretations.

Whereas the studies indicated above dealt mainly with boundary stresses, in this investigation the stresses are considered at points away from the sides of the opening.

In the previous studies no attempt was made to show stress patterns after initial failure took place. In this investigation the effect of failure in its relation to new stress distributions resulting therefrom are studied.

LABORATORY WORK PERFORMED

In this chapter the laboratory work performed by the author is described. The theoretical considerations including the basic assumptions, initial state of stress, and the conditions are discussed first. A description of the equipment and material for the models is next given. The experiments performed are then outlined with sample calculations. At the end of this chapter the findings obtained from the experiments are summarized.

THEORETICAL CONSIDERATIONS

Assumptions

Homogeneity and Isotropy - One of the first assumptions usually made is that rock is homogeneous and isotropic. Many objections to this assumption have been raised, especially by practical mining engineers. Stress analyses are applied to steel structures. Although steel consists of many irregular crystals, these crystals are small in comparison with the steel structures, so for all practical purposes, steel is considered homogeneous. By the same reasoning, rock may be considered homogeneous as long as the crystals and grains are small in comparison with the size of the opening.

Likewise, rock may be considered isotropic if the crystals and grains are oriented at random.

Linear and Perfect Elasticity - The rock in which the opening is created is assumed to be linearly and perfectly elastic, i.e., it follows Hooke's law. Rocks are not perfectly elastic. However, tests have shown that for all practical purposes the straight-line relation between stress and strain may be used (Panek, 1951, p. 3).

In the ideal case of study the assumption is usually made that faults and other planes of weakness are not present. Geologic disturbances may change the entire picture.

Initial State of Stress - The initial state of stress is of major importance. In all calculations it is assumed (1) that the pressure at depth is caused by the weight of the overlying rock and (2) that the rock is free from remaining stresses such as from tectonic forces.

The state of stress at any point can be represented by three principal stresses. The magnitude of the vertical stress can be expressed as $p = \rho h$ in which ρ represents the specific gravity of the rock and h the depth below the surface. The strains in the horizontal direction are of necessity zero. The solution for the horizontal stresses becomes (Fenner, 1938) $q_1 = q_2 = \frac{m}{1-m} p$ in which m is Poisson's ratio.

The effect of Poisson's ratio can be observed when substituting its theoretical minimum and maximum value in the above formula. In the case of the minimum value, or $m = 0$, the horizontal stress would be zero; in the case of the maximum value, or $m = 0.5$, the horizontal stress would be the same as the vertical stress. The latter condition represents hydrostatic pressure.

For most rocks Poisson's ratio varies between 0.2 and 0.3 (Peele, 1947). Thus the horizontal stress varies then between 0.25 and 0.43 times the vertical stress.

To reconstruct the state of stress in the laboratory, materials should be used which have approximately the same Poisson's ratio as the rocks studied. The error introduced by not using a material with exactly the same Poisson's ratio has been proved to be very small (Frocht, 1948, p. 179-201). A vertical, uniformly divided load will then be necessary only to simulate the state of stress that actually exists underground.

<u>Relation between Model and Prototype</u>

The relationship between the model and the prototype is discussed by Panek (1951, p. 26-27). He concludes that the scale ratio (ratio of a linear model dimension to the corresponding linear prototype dimension) may be chosen at will; the only requirement is that the model be geometrically similar to its prototype.

Duvall (1948) stated if the largest dimension of the opening is less than $\frac{1}{4}$ of the width of the plastic plate, the error is less than 7 percent. In all experiments described in this thesis the ratio between the largest dimension and the width of the plate was at least 1 to 6.

<u>Openings Studied</u>

The openings studied were a horizontal, 2 x 7-yd, rectangular prototype opening with sharp corners; a 2 x 7-yd, rectangular prototype opening with sharp corners with its longitudinal axis at 45° with the horizontal; a 4-yd, circular prototype opening; and a triangular prototype opening with a 4-yd base and 4-yd sides. It was assumed that these openings represent cross sections of infinitely long, horizontal drifts or tunnels.

Conditions

The openings are located 900 ft below the earth's surface.

The average pressure gradient for rock is taken to be 1 psi per ft of depth. To produce this pressure gradient in the model, the sides of the plates used were shaped in a hyperbolic curve as shown in Fig. 1.

Four different conditions of failure have been assumed. In all cases the rock was considered to fail under even the slightest amount of tension; however, different values were assigned for the shear strength as follows:

Case I - failure due to tension alone

Case II - failure due to tension and 1600-psi shear

Case III - failure due to tension and 1200-psi shear

Case IV - failure due to tension and 800-psi shear.

All rock which failed above the bottom level of the opening was assumed to be removed. This was represented on the models by enlarging the original opening to include the area represented by failure and removal.

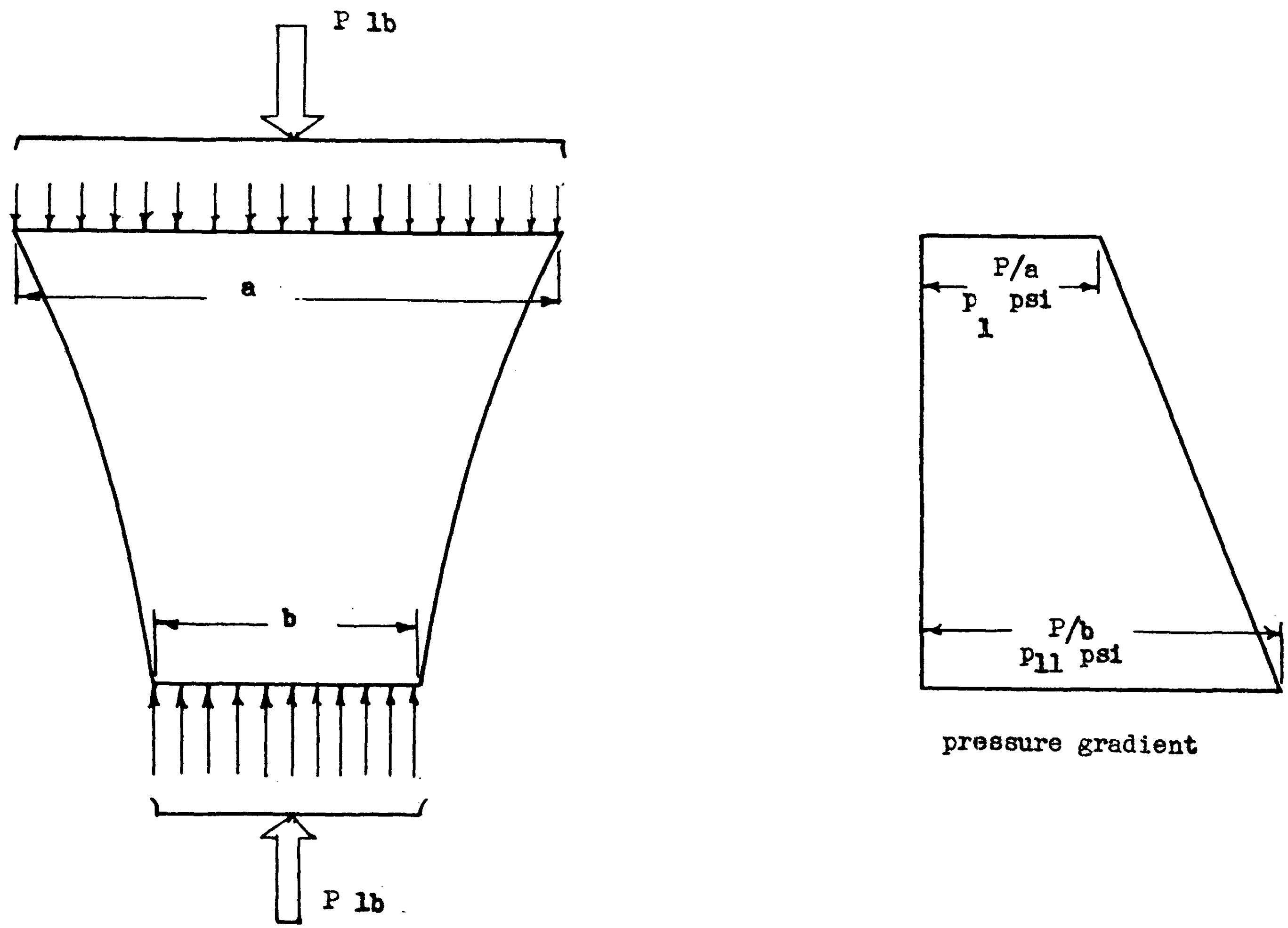


Fig. 1

EQUIPMENT AND MATERIALS USED

The models were made of $\frac{1}{4}$-in. plates of Columbia Resin CR-39, also known as Allite. The properties of this material are thoroughly discussed by Coolidge (1948).

The desired openings and outside shapes of the model were made by means of a jig saw, drills, files, sanding drums, mill cutters, sandpaper, and polishing powder.

Photoelastic stress patterns were obtained from a circular polariscope. The light source was a mercury-vapor light with a green filter, thus producing green, monochromatic light. Parallel rays of this light were produced by means of a condensor. The light was then passed through a polarizer, a quarter-wave plate, the model, a quarter-wave plate, and an analyzer. The produced light was next passed through a condensor and an image was formed on a ground-glass screen. The ground glass could be replaced by a film adaptor to record the results photographically. All lenses were 10 in. in diameter and could be evolved about their optical axes.

The model was placed in a loading device equipped with a leverage system to produce the required load.

EXPERIMENTS PERFORMED

The experiments performed may be divided as follows:

1. horizontal, 2 x 7-yd, rectangular prototype opening
   a. failure according to Case I
   b. failure according to Case II

c. failure according to Case III

d. failure according to Case IV

2. 2 x 7-yd, rectangular prototype opening with longitudinal axis at 45° with the horizontal

   a. failure according to Case IV

3. 4-yd, circular prototype opening

   a. failure according to Case III

4. triangular prototype opening with 4-yd base and 4-yd sides

   a. failure according to Case III

In each of the above experiments a model was made of the opening; the vertical, uniform load was applied; the stress pattern was recorded; the extent of failure for each particular case was calculated; the original opening in the model was enlarged to include the failed and removed rock; the new model was again loaded and the procedure was repeated. In this way a series of models was made to complete each experiment.

The results of the experiments were graphically plotted for comparison.

To test the validity of the results for both solid and stratified rock, a number of verifying experiments were performed. These experiments are described in the section "Solid Rock vs Stratified Rock".

As an example, experiment 1d with its calculations is as follows. The scale used was 1 in. = 8 yd, both horizontally and vertically. The horizontal mine opening was taken to be 2 x 7-yd with sharp, square corners. This was represented in the model as 0.25 x 0.865 in. The width of the model at the floor level was taken to be six times the width of the opening, or 6 x 0.865 = 5.19 in. At this level, the pressure in the prototype was 900 psi (depth x gradient).

In the model at a point 4 in. above the floor level of the opening, the vertical pressure had to represent 900 - 4 x 8 x 3 (in. x.scale x grad.) = 900 - 96 = 804 psi. The width of the model at that point was therefore 5.19 x $\frac{900}{804}$ = 5.81 in. In this way the width of the model was calculated at a number of points, and the sides of the model were thus constructed.

The distance between the bottom of the model and the floor of the opening was always kept at the equivalent of 6 yd in the prototype. The first model is shown in its actual dimensions in Fig. 2.

After the model was prepared, it was placed in the loading device, the proper load applied, and the stress pattern recorded. The proper load was calculated so that the critical shear failure was a distinct fringe line in the resulting stress pattern from the circular polariscope.

In the model as described above, the tension failure shows up independent of the magnitude of the load applied. If the first-order fringe is to represent the shear failure in Case IV (failure due to tension and 800-psi

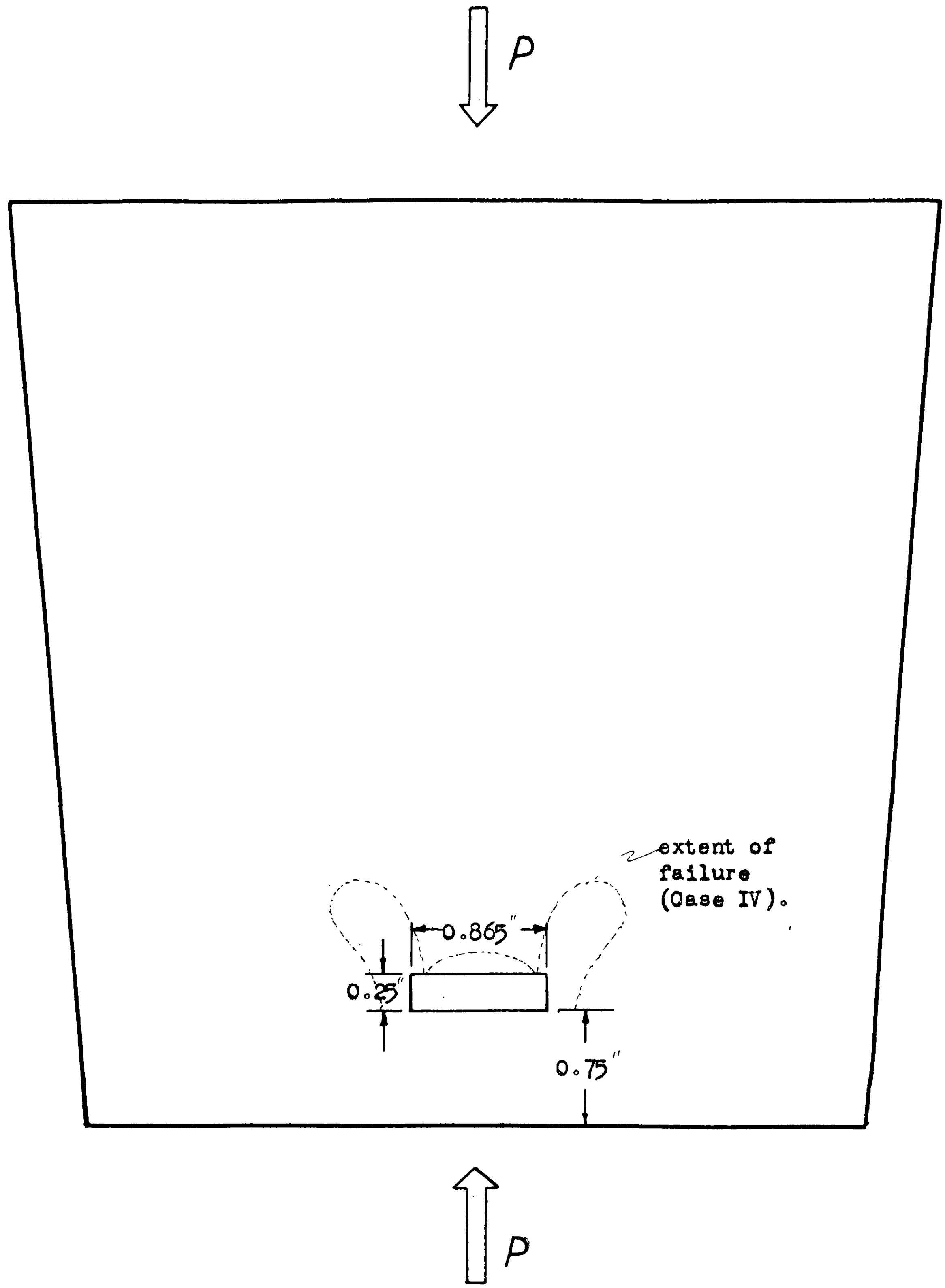


Fig. 2

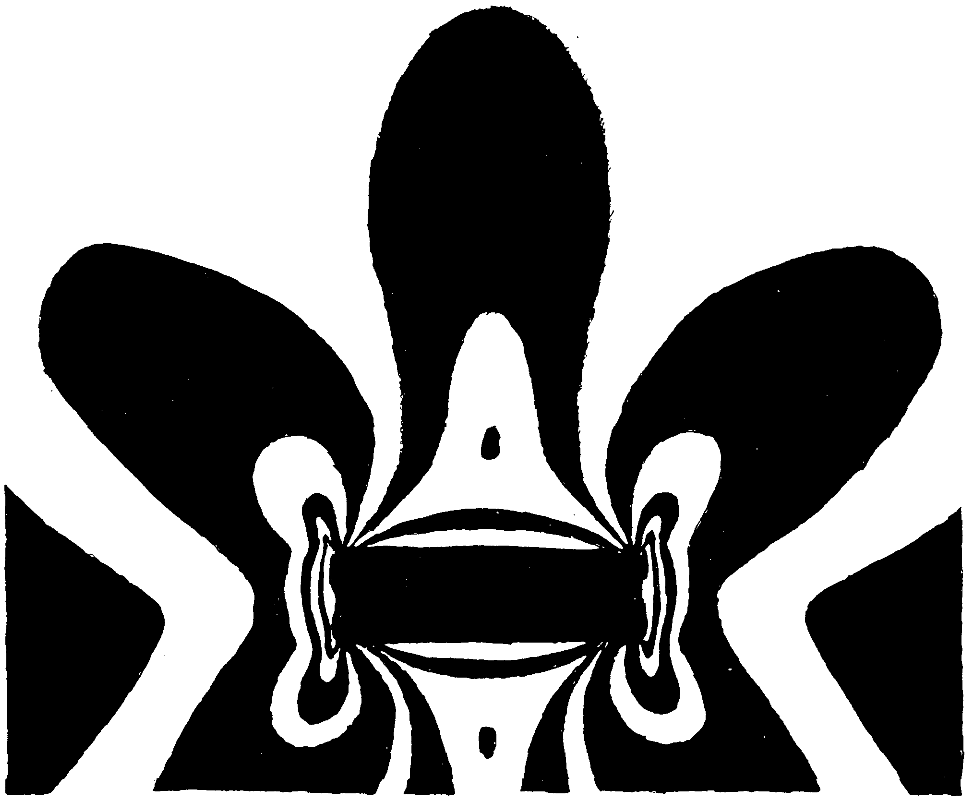
shear), the load to be applied may be calculated as follows.

The fringe value of CR-39 is 356 psi (Pandya, 1950). Therefore, 356-psi shear in the model has to represent 800 psi in the prototype. The required load becomes $\frac{356}{800} \times 900 = 400.5$ psi at the bottom level of the opening, or a total of 400.5 x 5.19 x 0.25 (thickness of plate) = 520 lb on the model. If the fringe of $1\frac{1}{2}$ order is to be the critical fringe, the load must be 780 lb. For each model two or more different loads were applied, and the results averaged for better accuracy. The fringe pattern of the first model is shown in Fig. 3 for a load of 1040 lb.

The area which failed, as calculated from the first model, is indicated in Fig. 2 for Case IV. Then a new model was made with the original opening plus the failed area as the new opening. Usually a different scale had to be used to keep the width of the plate six times as large as the largest dimension of the opening and still within reasonable limits.

With the new model, the same procedure was followed, and the extent of failure was calculated. In this way a series of models was made within the limits dictated by the outside dimensions of the plate.

For the horizontal, 2 x 7-yd prototype opening, a series of models was made for Cases I, II, III and IV. The resultant failure diagrams are shown in Figs. 4, 5,



AB=6"
CD=5"
P=1040 lb
opening
0.25x0.865"

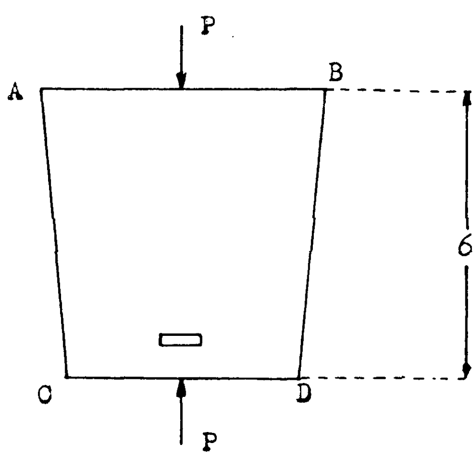


Fig. 3

6 and 7. The cumulative amount of failure by tension and by shear was planimetered each time a new model was made. These amounts are graphically represented in Fig. 8.

In Fig. 9, the results of experiment 2a are shown. The failed rock above the shaded lines between 0° and 135° was removed. It may be noted that the ultimate failure was virtually the same as that for the horizontal opening under similar conditions.

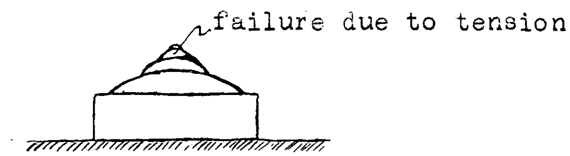
To show that the original shape of the opening did not affect the ultimate result of failure, Case III (failure due to tension and 1200-psi shear) was applied to the horizontal, rectangular opening, the circular opening, and the triangular opening. The results of the series of experiments are plotted in Fig. 10.

SOLID ROCK VS STRATIFIED ROCK

A number of experiments were performed to test the validity of the final conclusions obtained for both solid rock and stratified rock.

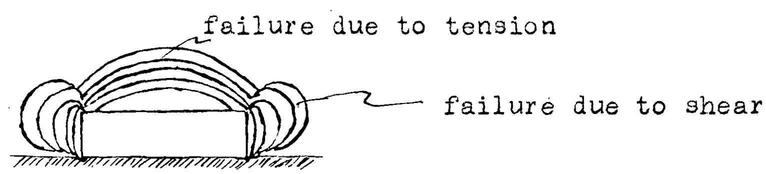
First, a beam 12 in. long, 1 in. high, supported by two knife edges 3 in. from the ends of the beam, was loaded vertically by a point load of 17 lb (Fig. 11). The resultant stress pattern from the circular polariscope is shown in Fig. 15.

The beam was then split in the longitudinal direction, so that two identical beams 12 in. long and 0.5 in. high were obtained.



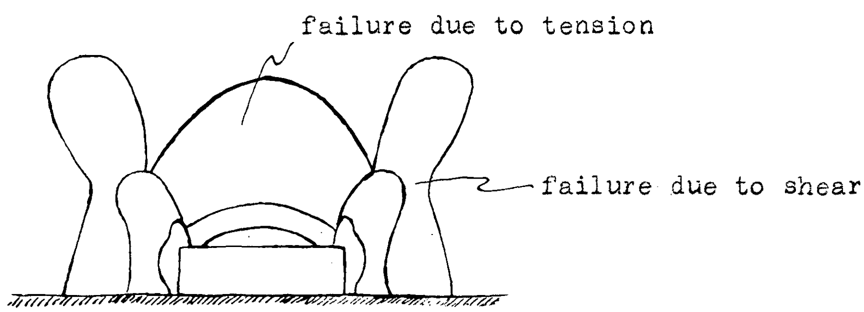
Failure due to tension.

Fig. 4



Failure due to tension and 1600.psi shear.

Fig. 5

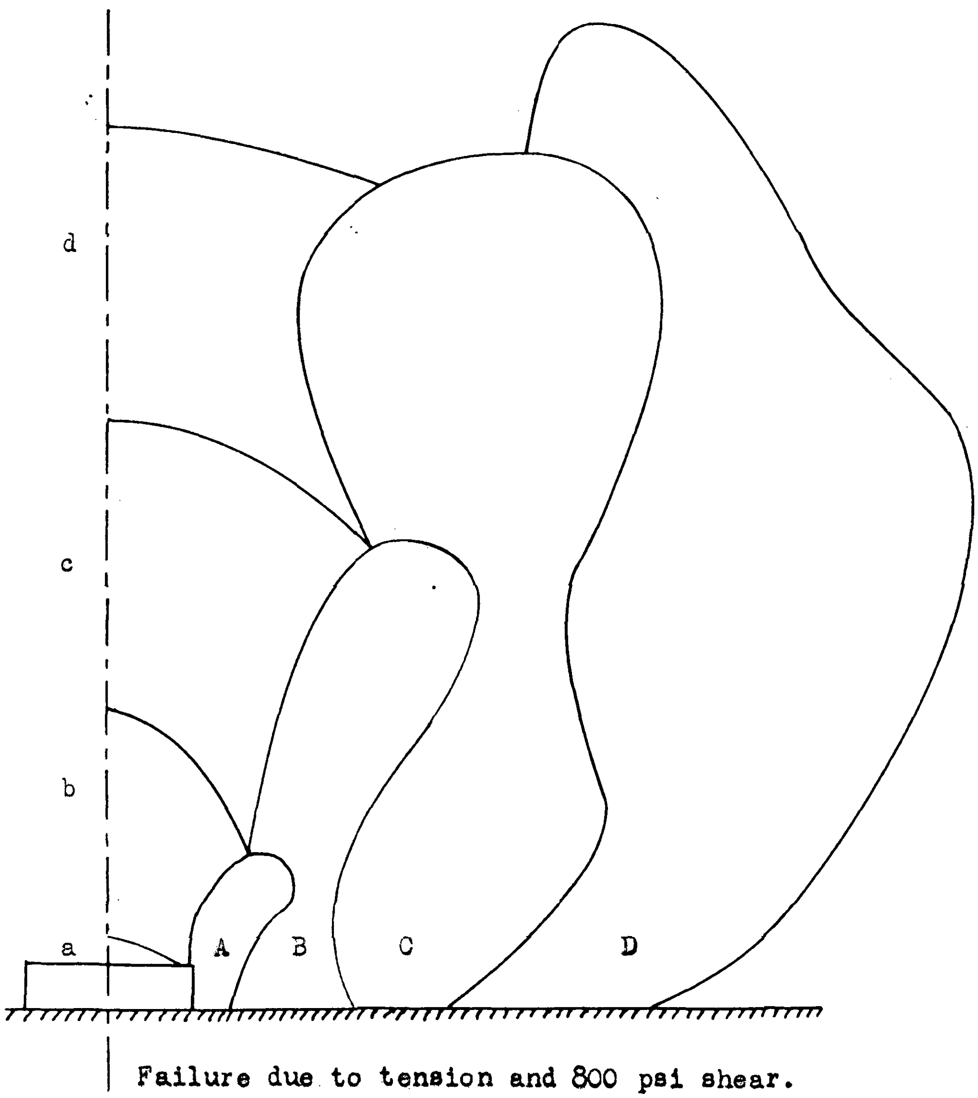


Failure due to tension and 1200 psi shear.

Fig. 6

scale: 1 inch=8 yard

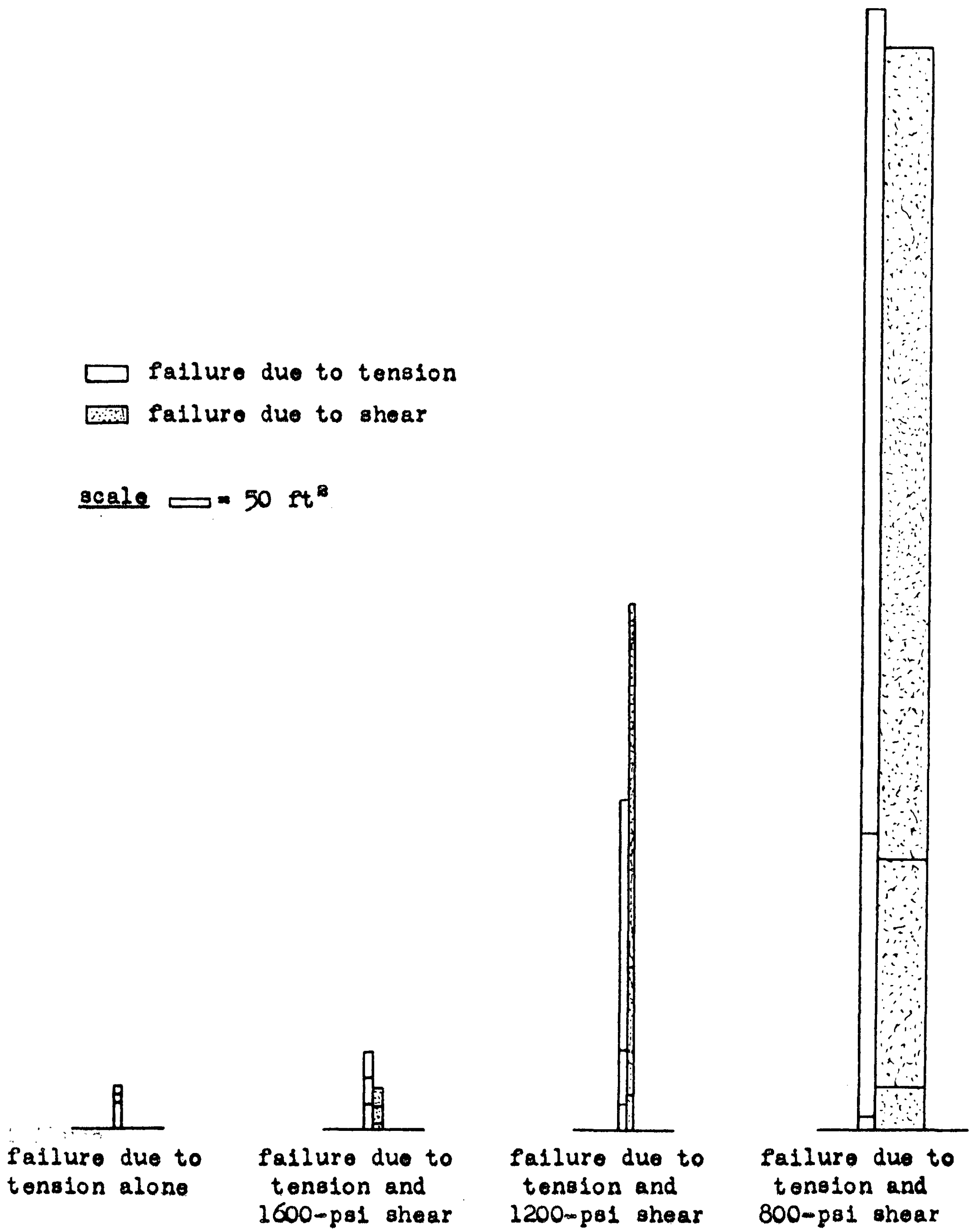
a,b,c,d--failure due to tension
A,B,C,D--failure due to shear



Failure due to tension and 800 psi shear.

scale: 1 inch=8 yard

Fig. 7



GRAPHICAL COMPARISON OF FAILURE AFTER A SERIES OF THREE MODELS FOR EACH CASE.

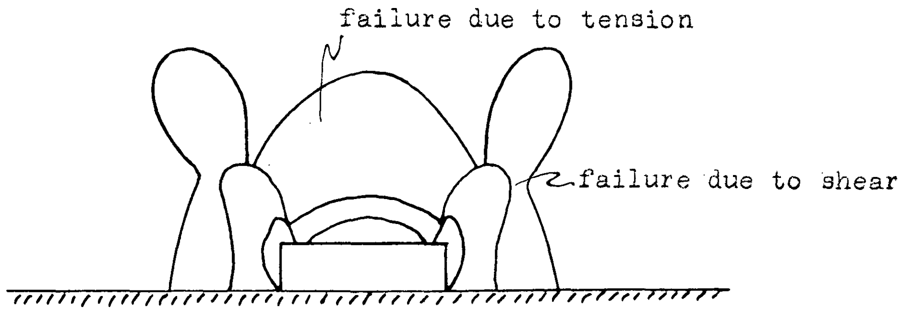
Fig. 8

a,b,c,d--failure due to tension
A,B,C,D--failure due to shear

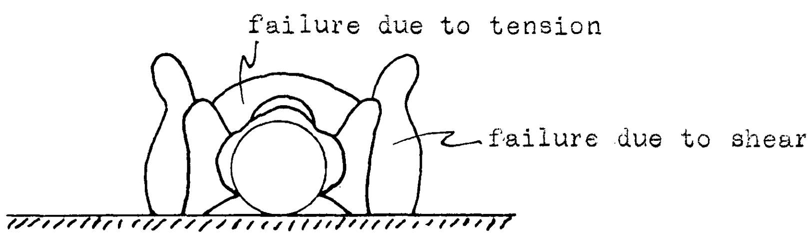
Failure due to tension and 800-psi shear

scale: 1 inch-8 yard

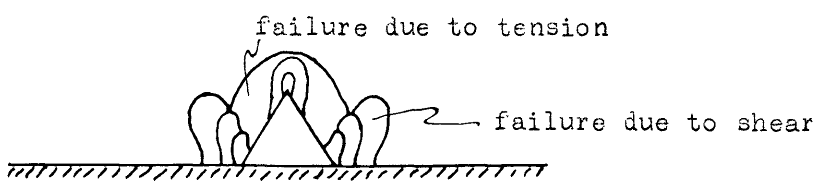
Fig. 9



Failure due to tension and 1200 psi shear.



Failure due to tension and 1200 psi shear.



Failure due to tension and 1200 psi shear.

scale: 1 inch=8 yard

Fig. 10

The two beams were then placed on the knife edges as shown in Fig. 12. A point load of 17 lb was again applied in the center. The resultant stress pattern from the circular polariscope is shown in Fig. 16.

Comparison of Fig. 15 and 16 shows that the fringes are more closely spaced in the case of the two 0.5-in. beams than in the case of the 1-in. beam, indicating a weakening upon splitting. As the moment of enertia varies as the third power of the height of the beam, the moment of enertia for the two smaller beams together is much smaller than for the large beam alone.

Vertical holes 1/16 in. in diameter were then drilled through the two beams as indicated in Fig. 13. Brass rods 1/16 in. in diameter were placed in the holes, the beams were again placed on the knife edges, and a point load of 17 lb was again applied in the center. The resultant stress pattern is shown in Fig. 17. Except for deviations from the holes in the plastic, the stress pattern approaches similarity with the stress pattern of the original 1-in. beam.

Two more holes were drilled through the two beams (Fig. 14), brass rods were brought in, and a point load of 57 lb was applied in the center. The resultant stress pattern is shown in Fig. 19. For comparison the 1-in. beam was also loaded with a 57-lb center load, the stress pattern of which is shown in Fig. 18. Again, similarity may be noted between these two stress patterns, with the

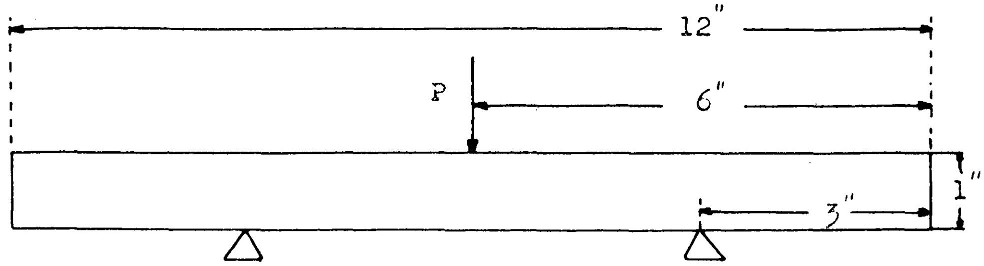


Fig. 11

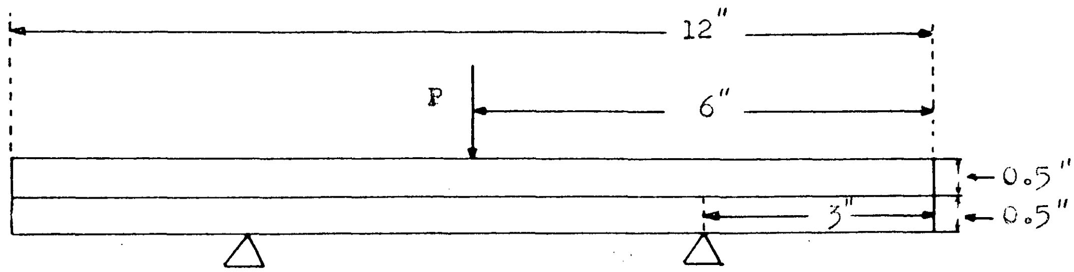


Fig. 12

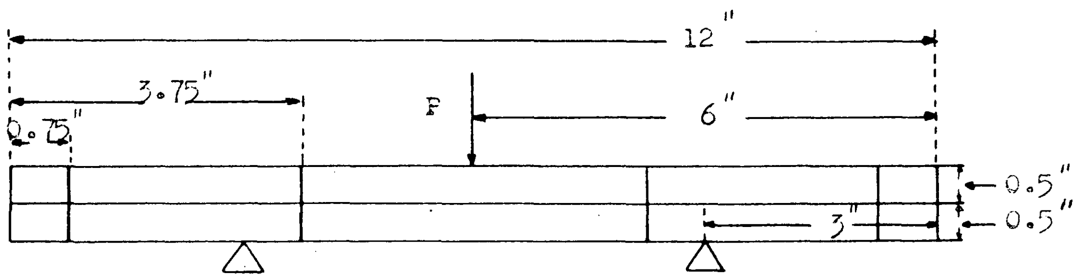


Fig. 13

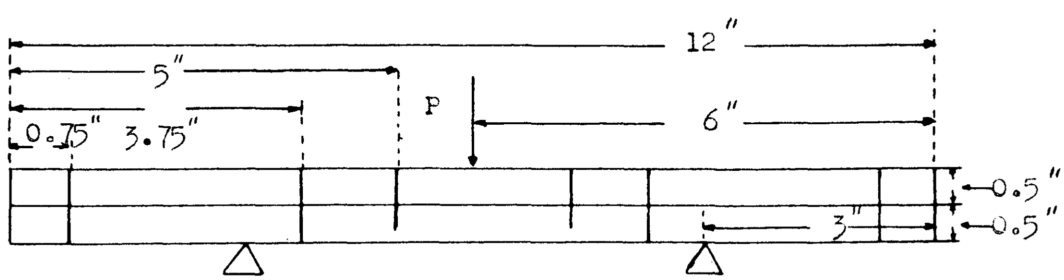


Fig. 14

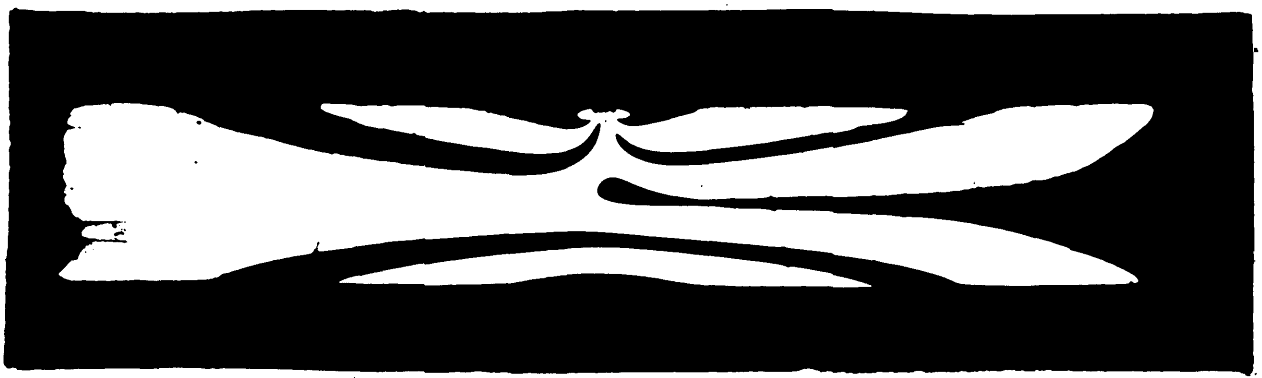


Fig. 15

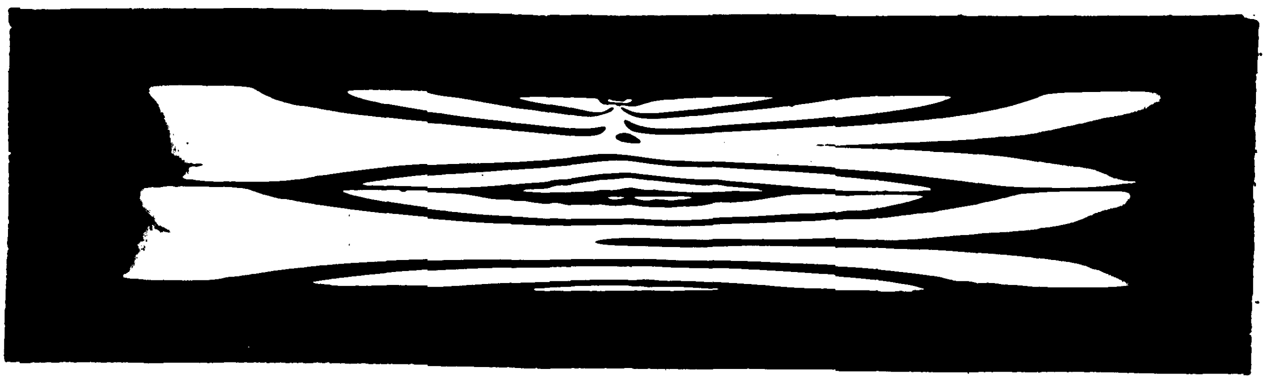


Fig. 16

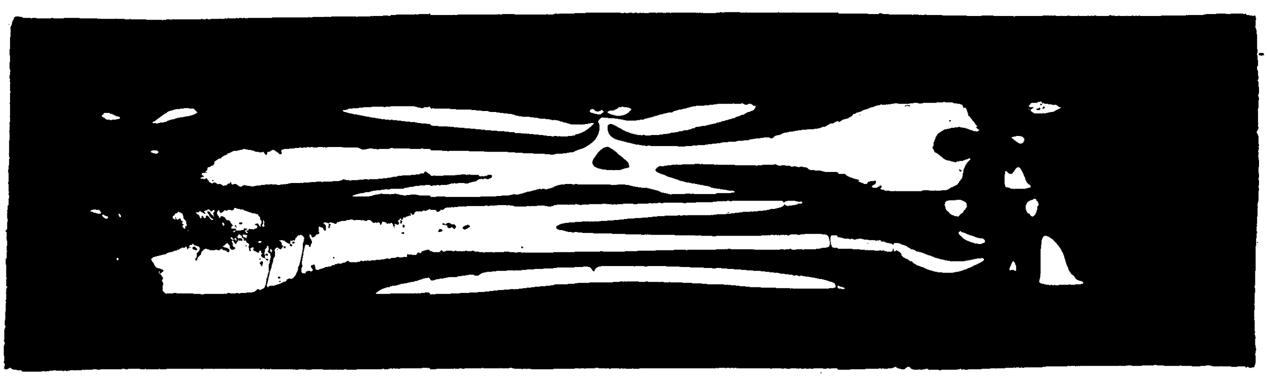


Fig. 17

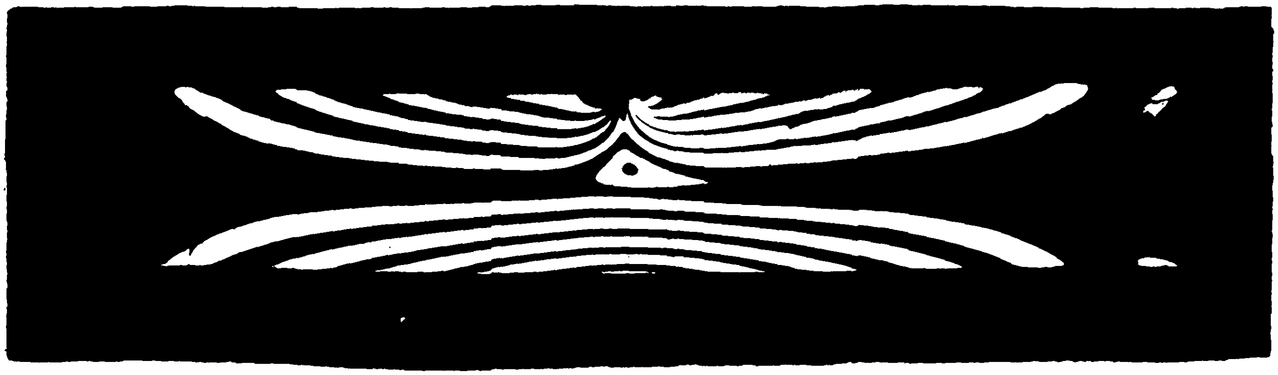


Fig. 18

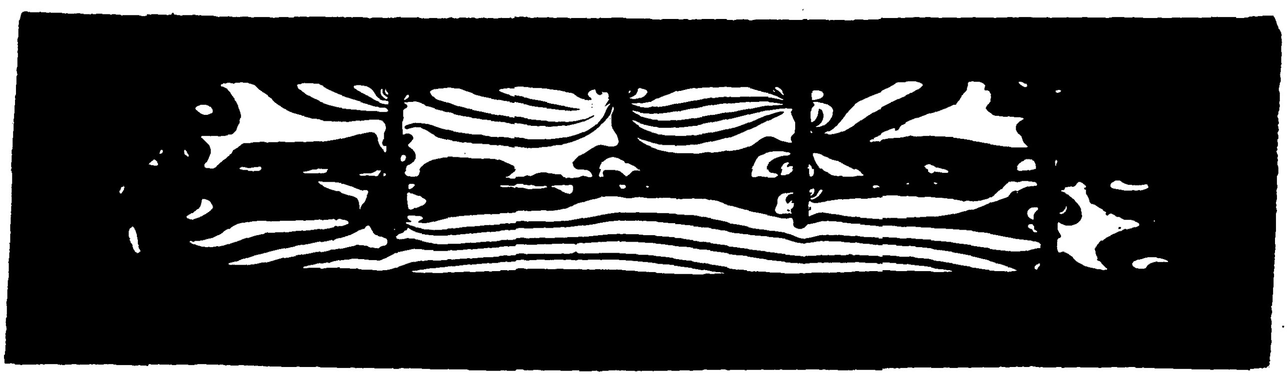


Fig. 19

exception of deviations around the vertical holes.

The next set of experiments is shown in Figs. 3, 20, 21 and 22. The dimensions and loading conditions are indicated in a diagram underneath the resulting stress pattern. It may be observed from these patterns that the models representing layered rock (Figs. 20, 21 and 22) produce virtually the same stress pattern as the model representing the solid rock (Fig. 3). Fig. 22 is a stress pattern of a model making use of aluminum and Allite. The fact that this pattern is virtually the same as the patterns produced by the Allite models indicates that Poisson's ratio may vary without invalidating the final result.

FINDINGS

From Figs. 4, 5, 6, 7 and 8, it may be concluded that if rock were to fail by tension alone, the failure would eventually come to a stop. In that case, the walls of the opening would be unaffected throughout the history of the opening.

When the rock starts to fail by some shear too, the walls of the opening are affected and therefore the span in the roof is increased, resulting in more failure by tension. If the rock is not supported, the failed rock falls out of place, and more failure results because of stresses which are set up around the new opening. As can be seen from Fig. 10, the ultimate shape of the caved opening will eventually be the same; however, the rate of failure is different.

AB=6"
CD=5"
P=1040 lb
opening
0.25x0.865"

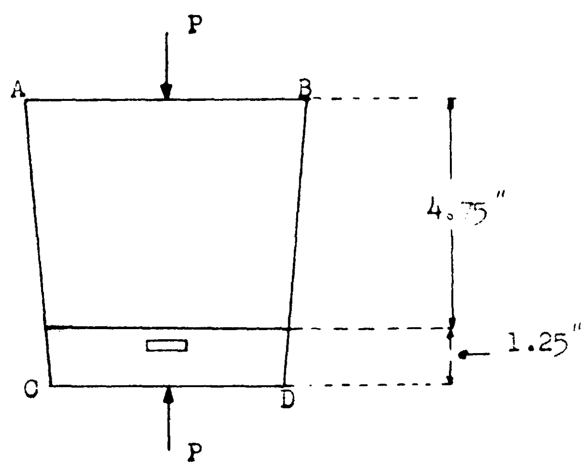


Fig. 20

AB=6″
CD=5″
P=1040 lb
opening
0.25x0.365″

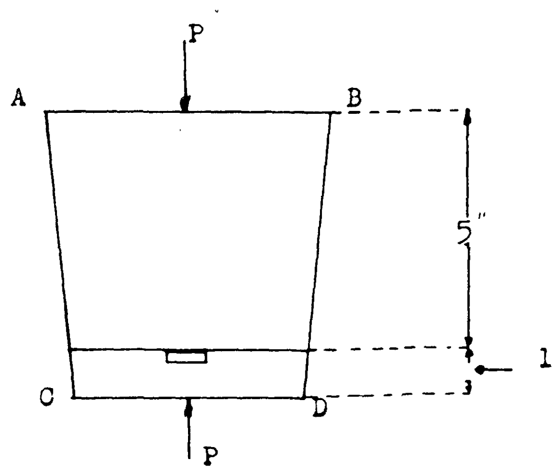


Fig. 21

AB=6"
CD=5"
P=1040 lb
opening
0.25x0.865"

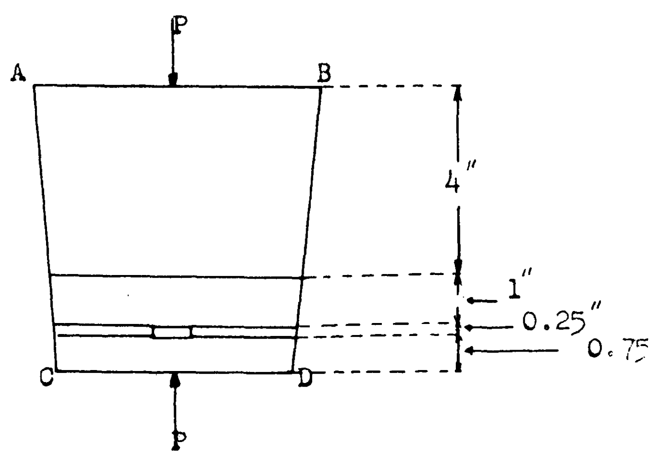


Fig. 22

Since failure in practice usually does not result in a dome-like surface, and from the results of the experiments described above, the conclusion can be made that failure due to shear is primary and failure due to tension is secondary.

From the experiments described under "Solid Rock vs Stratified Rock", the conclusion can be made that if friction between stratified layers is large enough, the rock may be considered to be solid. The frictional resistance equals ρhf, in which ρ is the specific gravity of the overlying rocks and f the frictional constant. From this, it may be reasoned that the frictional resistance at depth before mining is begun, is large enough to warrant the assumption that all rock may be considered to be solid.

CONCLUSIONS

In this chapter the dome theory, a study of natural arches, observations of cleavage induced by mining in British coal mines, and a case of swelling of ground will be discussed. Following each discussion the author expresses his opinion on the subject, backed by conclusions derived from the experiments described in this thesis.

Following these discussions the importance of horizontal support is emphasized, as substantiated by the findings obtained from the experiments.

At the end of this chapter a summary of conclusions is given, which includes both the findings directly obtained from the experiments and the conclusions indirectly obtained by further reasoning.

THE DOME THEORY DISCUSSED

The dome theory as advanced by Fayol was later developed by Crowle (1931), Dinsdale (1936) and Spalding (1937).

The dome theory postulates that when an opening is made in rock under stress, the rock in the top of the opening will often fail, and caving will continue until the opening has assumed a dome shape. Thereafter, equilibrium is apparently re-established. The oldest theory which attempted to explain the phenomenon is the cantilever theory, which postulates that the hanging wall consists of stratified layers that behave as independent beams with fixed ends and a uniformly distributed load. When failure occurs, inward shearing at the ends will cause the formation of a dome-shaped space.

The pressure-dome theory, which apparently evolved from the cantilever theory, merely asserts the formation of a pressure ring or dome around the excavation. The rock within the dome fails under (1) its own weight, (2) expansion and perhaps also (3) inherent stresses. If the hanging wall is not allowed to cave freely, it will throw its full weight on the supports of the opening.

The dome theory has been presented with many variations, and many different shapes of the pressure dome have been suggested. All interpretations are based, however, on the principle that the cause of rock failure lies within the dome (gravity, expansion and/or inherent stresses).

Spalding (1950) advanced the theory of closure. When a stope is opened, the weight of the rock within the hanging-wall fracture dome is taken chiefly by the supports. This weight squeezes the supports, and closure takes place.

The weight of the rock between the stress envelope and the fracture dome (Fig. 23)—or in the case where there is no fracture dome, the weight of all rock within the stress envelope—is carried by the rock itself acting as a beam and resting on the walls, thus bridging the stope.

Two forces cause closure by gravity: the dead weight

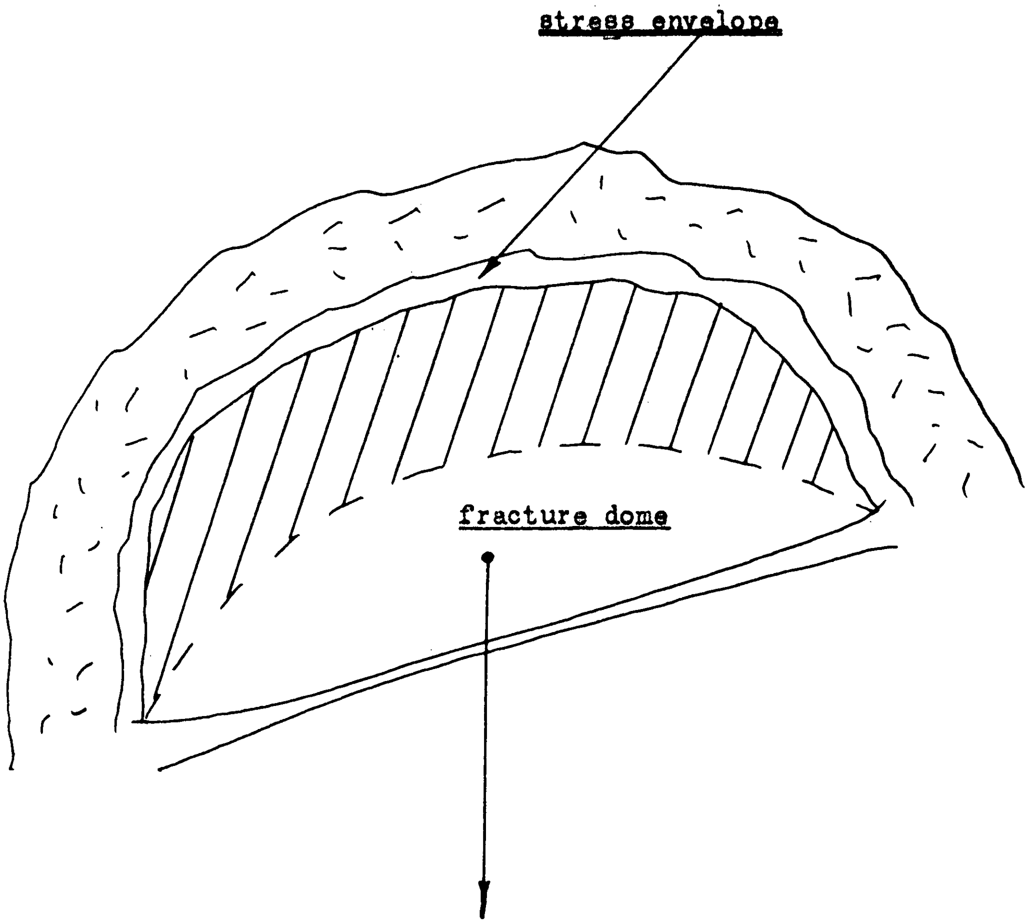


Fig. 23

of loose rock in the fracture dome and the force exerted by the solid wall bending under the weight of rock in the expansion dome.

This closure by gravity is, according to Spalding (1950), limited for several reasons. First, since the volume of rock within the hanging-wall portion of the stress envelope will not necessarily increase with depth, the forces which cause closure by gravity will be of the same order at all depths. Second, since gravity of necessity acts vertically, closure cannot take place in vertical stopes except perhaps where the rocks are so fractured as to become mobile. Third, there can be no movements due to gravity in the footwall.

Spalding therefore suggests that closure takes place because of the expansion of rock which goes from a compressed pre-mining condition to a stress-relieved post-mining condition.

A general criticism of the dome theory may be summarized in the following statements:

1. No adequate explanation has ever been given for the occurrence of the pressure dome.

2. The fact that the origin of the forces which cause failure in the dome lies within the dome has never been proved.

3. The shape of the suggested dome has never been theoretically calculated.

4. It is doubtful that the magnitude of the acting

forces assumed by the theory is sufficient to break up the rock within the dome, especially if the rock is homogeneous and has no planes of weakness.

5. The theory of fixed beams does not hold except in shallow mining in stratified layers.

6. The concept that expansion of compressed rock upon stress relief is responsible for closure cannot be justified, considering the value for Young's modulus for most rocks. Spalding noted, that complete closure might take place if the wall rock is so fractured as to become mobile. The author of this thesis believes that the mobile character of fractured rock in the walls of mine openings is very important in closure. The rock in the walls of the mine opening has been shown to be subjected to shear stress, which causes failure that may be macroscopic or microscopic. After failure, the rock can be deformed, and closure takes place.

STUDY OF NATURAL ARCHES

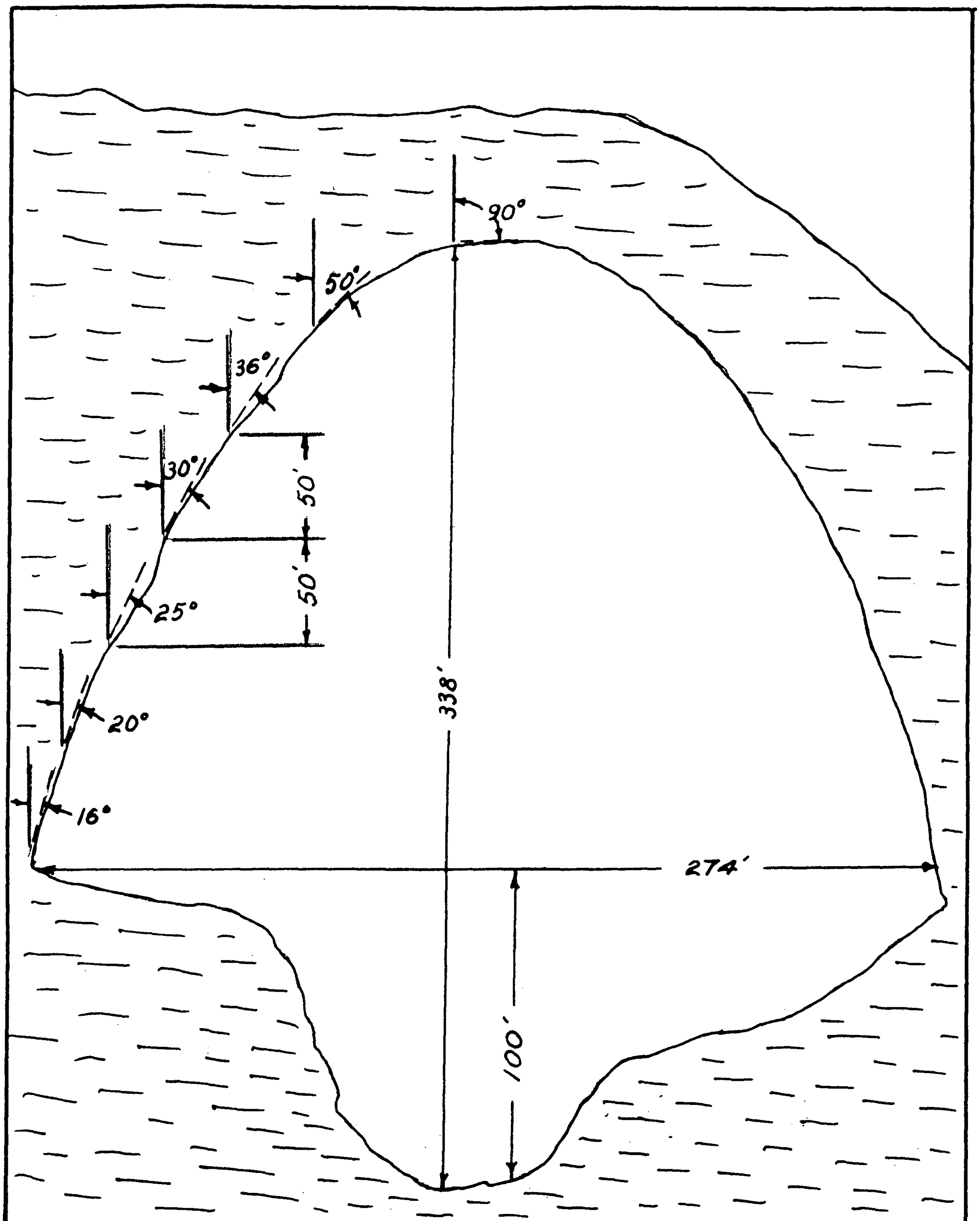
Cogan (1950) made a study of natural caves and natural bridges and prepared many pictures and diagrams of natural-arched formations. Among these were caves which were used by Neanderthalian and Magdalenian men tens of thousands of years ago, still showing footprints on the floor and printings on the wall. He reasoned that in his examples of caves and bridges, weathering and erosional agencies prefer to act on rock material which is not under stress or on that which is under the influence of tensile stress. The result that material not contributing to the support of the opening is easily weathered away, and the opening assumes the shape by which it can best withstand the prevailing stresses.

His examples of arching correspond with photoelastic determinations by Duvall (1948); i.e., narrow openings are best protected by steep arches, and wide openings by flat arches with steeply sloped abutments.

Cogan's example of the Rainbow Bridge is reproduced in this thesis, as this natural bridge shows a symmetrical outline and represents an almost perfect arch (see Fig. 24). All of his work was performed in strata where the shear resistance of the rock was larger than the shear stress. In the present investigation, the conclusions in the case of failure by tension alone supports Cogan's analyses.

CLEAVAGE INDUCED BY MINING

Faulkner and Phillips (1935) examined failure of rocks in underground workings in coal mines in England. They observed that the general direction of the cleavage induced in longwall work is parallel to the face, curving to the rib at the side and ahead of the workings. The direction of the face advance with reference to the cleats apparently does not affect the cleavage. They reasoned that shear stress is mainly responsible for the



General shape of Rainbow Natural Bridge (From a series of photographs by J.Cogan)

Fig. 24

cleavage observed.

In his contribution to the Symposium on Rock Pressure in Holland (1947), Phillips again discussed the cleavage induced by mining. He had made thin sections of rock samples taken adjacent to workings. When viewed through a petrographic microscope with crossed nicols, microscopic shear fractures and slip planes in the thin sections are noticeable. Phillips suspected these shear fractures to have been induced by the mining.

The findings of the experiments outlined in this report agree with the reasoning of Faulkner and Phillips that the cleavage observed resulted from extensive shear stresses.

SWELLING OF GROUND

In coal mines frequent observations have been made of heaving of the floor. Similar observations were made in metal mines.

Some authors (Joseph, 1938; Spalding, 1950) explain the swelling of the ground by the expansion of rock upon relief of pressure. From the pressure difference and Young's modulus, calculations easily show that expansion can not be the reason for the swelling.

Other authors have tried to explain the swelling and heaving of ground by the phenomenon of plastic flow of rock. Adams (1912) and Griggs (1936) investigated this phenomenon. Even though their final conclusions are not the same, for the purpose of mining at present depths it may be concluded that plastic flow will not occur.

Farmin (1945) remarks that in the Idaho Maryland Mine, Grass Valley, California, swelling ground crushes timber and closes drifts. The swelling action continues with diminishing intensity for three to ten years. During that period timber sets sometimes had to be replaced five to nine times and the rock had to be removed. After "the squeeze has been taken out of the ground," the swelling of the ground ceases.

Farmin considered that the surface action of liquid water on the ground directly caused the swelling. He performed tests soaking samples in water and observed the increases of the rock volume. He did not make any pressure determinations simultaneously with volume determinations. He noted that when a drift was closed for a certain period of time and kept dry, the swelling action ceased. As soon as the drift was opened and the moist air wetted the walls, the swelling action began.

The author of this thesis believes that the expansion due to the addition of water under the prevailing pressures could never cause the swelling observed. The author believes that because of microscopic shear and slip planes the rock is able to behave in a plastically flowing manner. He believes that the moisture in the Idaho Maryland Mine acted as a lubricant, thus decreasing the frictional resistance on the slip planes.

The phenomenon that "the squeeze was taken out of the ground" can be explained as follows. The rock that failed by shear will be moved out of place as long as its weight times the frictional resistance is smaller than the horizontal component of the forces tending to move the material. When the amount of failed rock has become large enough so that its weight times the frictional resistance has become equal to the moving forces, equilibrium has been reached. This equilibrium in the Idaho Maryland Mine seems to be dependent upon the moisture content of the rock.

IMPORTANCE OF HORIZONTAL SUPPORT

If the original opening is to be maintained, supports must usually be installed.

Case I - In failure due to tension alone, supports are necessary only to maintain the exact shape of the opening. However, in this case, the openings could be given an arch-shaped roof for natural support. No more failure will then result.

Cases II, III and IV - In all of these cases, the rock is assumed to fail because of both tension and shear. It may be considered that the supports have the function of keeping the rock from caving in the area of failure due to tension and in the area of failure due to shear. However, the supports also have the function of keeping the rock from further failure due to both tension and shear.

It is a natural tendency to support the rock vertically rather than horizontally. Horizontal support is frequently neglected completely. Horizontal support may prevent failure in the roof and aid the vertical supports.

In Fig. 25 the side supports are perfectly able to hold in place the rock failed by shear. The vertical supports now have to bear the weight of only the rock failed in the dome-like area of failure due to tension, as indicated.

In Figs. 26 and 27 the side supports were installed some time after the opening was made. The rock that failed by shear had partially caved (Fig. 26). Because of this, a new stress pattern was set up, causing more failure by both shear and tension. If the side supports are brought in, at this stage, further failure might be prevented, but the vertical supports would then have to bear the weight of the dome-like area of failure due to tension, as indicated in Fig. 27.

It may be concluded from the above that horizontal support is as important as, if not more important than, vertical support; that horizontal supports and vertical supports both should be installed as soon as conditions allow; and, those supports that have failed should be replaced as soon as possible.

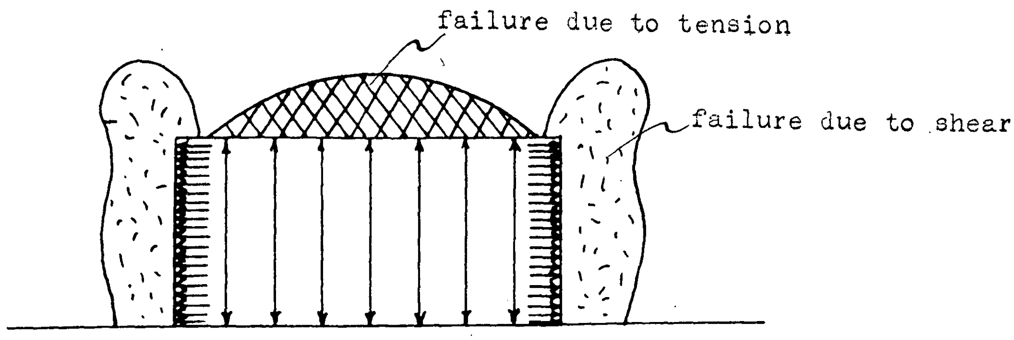


Fig. 25

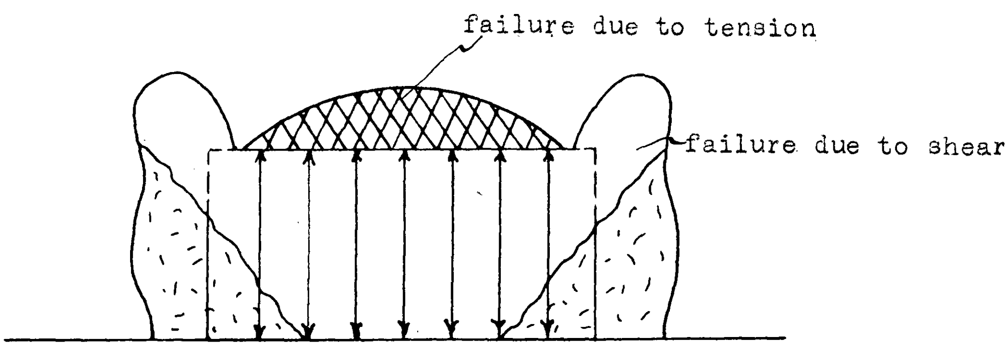


Fig. 26

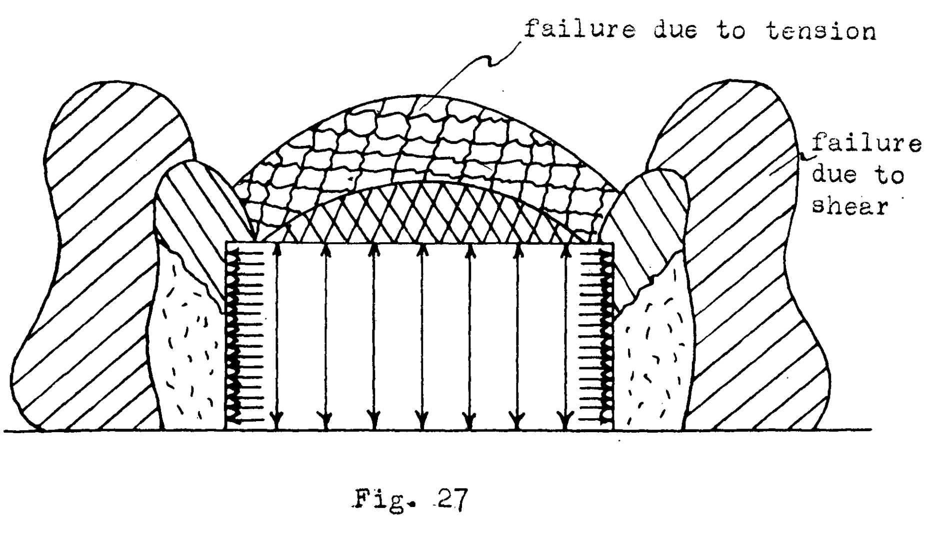


Fig. 27

SUMMARY OF CONCLUSIONS

In the following a summary of the conclusions is given, which includes both the findings directly obtained from the experiments and the conclusions indirectly obtained by further reasoning.

1. If rock fails because of tension alone, a natural arch will result, and the walls will not be affected.

2. If rock fails because of both shear and tension, shear failure will be of primary importance.

3. After failure once starts, it ultimately results in the same shape of failed area, independent of the original shape of the opening.

4. Closure of mine openings takes place because of the slippage along shear planes.

5. Horizontal support is as important as vertical support, if not more so.

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