MULTISCIA
JOURNALS PUBLISHERS

FRONTIERS IN MECHANICAL, MINING AND MATERIAL ENGINEERING

ISSN: (3065- 4025)

https://multisciajournals.com/journals/index.php/fmmme

editor.fmmme@gmail.com

Frontiers in Mechanical, Mining and Material Engineering
Volume 2 Issue 1 2026

An introduction to rock mechanics for practical engineers

Erick Eberhardt sahoo

Department of Mech

Article Info

Received: 20-01-2026 Revised:25-02-2026 Accepted:8-3-2026 Published:20-3-2026

Part 1. Theoretical Considerations

The author discusses theoretical issues in the first of three pieces that include excerpts from a set of six lectures given at the Royal School of Mines, Imperial College of Science and Technology, in November 1965. The other two will focus on laboratory methods before moving on to field rock mechanics.

This series of essays aims to introduce some of the fundamentals of rock mechanics to readers who want to learn more about the topic without becoming bogged down in technical minutiae. The articles' material is based almost exclusively on the author's personal experience; as a result, it does not provide a comprehensive overview of the field of rock mechanics. As a result, an overarching rock mechanics theory has been carefully avoided, leaving the reader to make his own judgments based on the facts as they are presented.

Definition and Scope

The American National Academy of Science's Rock Mechanics Committee established the following definition in 1963:

The theoretical and practical study of rock's mechanical behavior is known as rock mechanics. It is the area of mechanics that studies how rocks react to the force fields in their physical surroundings.

It is convenient to subdivide rock mechanics into the following branches:

a) Structural rock mechanics, which is concerned with the stability of engineering structures in which the material is predominantly rock.

b) Comminution, which is concerned with the reduction of rock to small fragments by the application of external forces as in drilling, blasting, cutting and grinding.

Rock deformation and fracture processes are controlled in each of these areas of rock mechanics. To maintain the stability of the structure in the first scenario, excessive rock failure (here defined as either excessive deformation or fracture) must be avoided; in the second scenario, rock fracture must be produced with the least amount of external energy. In terms of practical engineering, major tragedies like the Coalbrook mine accident and the failures of the Malpasset and Vajoint dams serve to highlight the significance of rock fracture.

Rock Fracture - Griffith Theory

About 15 years ago, rock mechanics research was started in South Africa in an attempt to better understand the rockburst hazard that exists in many deep-level gold mines.

Figure 1 shows the results of a typical rockburst. According to one definition, it is damage to subterranean workings brought on by uncontrollably shattered rock that releases energy violently in addition to that from falling rock fragments. Energy fluctuations are linked to the primary causes of rockbursts.
caused by mining in the rocks around sizable excavations; these factors have been discussed elsewhere2.

From the perspective of rock mechanics, a rockburst's primary feature is that it happens in hard, brittle, extremely competent rocks. As a result, it was deemed acceptable to examine the behavior of the rock material itself when examining the fracture behavior of these rocks, treating it as a homogenous, isotropic solid and disregarding the impact of significant geological discontinuities. The reader will quickly see the shortcomings of this method when it comes to the fractured and geologically discontinuous rocks that are found on or close to the earth's surface. However, it is thought that developing a logical behavior pattern for rock masses can be aided by comprehending the fundamental mechanism of rock material fracture.

It has been discovered that Griffith's theory of brittle fracture3, which McClintock and Walsh4 adapted to account for the primarily compressive stresses in rock mechanics, offers a solid theoretical foundation for the prediction of rock fracture phenomena5. This hypothesis is predicated on the idea that fracture starts at intrinsic fissures and discontinuities in the material, and that tensile stress created at the crack tip under load causes these cracks to propagate. According to Brace6, fracture in hard rock typically begins at grain boundaries, which can be thought of as the intrinsic discontinuities needed by the Griffith hypothesis.

Griffith's initial hypothesis focused on brittle fracture under applied tensile stress, and his calculations were predicated on the idea that an elliptical aperture might be used to represent the intrinsic crack that causes fracture. This simplifying assumption is no longer true when applied to rock mechanics, where the applied stresses are primarily compressive. The theory must be adjusted to take into consideration the frictional forces that arise when the crack faces are pressed into contact. McClintock and Walsh4 implemented this change by making additional simplifying assumptions about the mechanics of fracture closing. Berg7 has recently provided theoretical validation for these simplifying assumptions.

The extent to which the modified Griffith theory defines the fracture behaviour of rock is illustrated in Figure 2. Published triaxial strength test data for the fifty rock and concrete types listed in Table 1 are included in this graph. In order to render the test results comparable and to minimise differences caused by different testing techniques, specimen sizes and environmental conditions, the values are plotted on dimensionless scales which are obtained by dividing each test result by the uniaxial compressive strength of that particular material.

A further illustration of the usefulness of the Griffith theory in defining the fracture behaviour of hard rock is given in Figure 3. In this figure a theoretical Mohr envelope is fitted to Mohr fracture circles obtained from triaxial tests on specimens of a typical South African quartzite.

In spite of the encouraging agreement between theoretical and experimental results, illustrated in Figures 2 and 3, it would be incorrect to suggest that the Griffith's theory provides a complete description of the mechanism of rock fracture. It must be emphasised that its derivation is such that it is only strictly correct when applied to fracture initiation under static stress conditions[5]. It is largely fortuitous that it can be so successfully applied to the prediction of the fracture of rock specimens since, once fracture has initiated, propagation of this fracture and ultimate disintegration of the specimen is a relatively complex process[8, 9, 10]. Fortunately, it appears that the forces involved in fracture propagation are closely analogous to the friction forces assumed in the modified Griffith theory and hence the general form of the equations which define fracture propagation is very similar to that of the equations which define fracture initiation.

Figure 1. Effects of a rockburst in a deep-level South African gold mine.

The original and modified Griffith theories, when expressed in terms of the stresses at fracture[5], contribute little to the understanding of rock fracture under dynamic stress conditions, the energy changes associated with fracture or the deformation process of rock. However, since the theoretical concept of fracture initiation from inherent cracks has proved so useful in describing the observed fracture behaviour of rock, this concept is being extended to the theoretical study of energy changes and deformation processes in rock [11, 12].

The author is particularly fascinated by the belief that the processes which govern the failure of large, fissured and discontinuous rock masses are very similar to those which operate during the failure of a small rock specimen[10]. It is hoped that a rational theoretical description of the movement of interlocking blocks of rock in a large rock

mass will eventually be built up.

Additional Factors Governing Fracture

The Griffith theory was derived on the assumption that the material contains a random distribution of uniform cracks and that the inherent physical properties of the material remain constant. It is interesting to consider to what extent the theoretical concepts of the Griffith theory can be modified to cover cases in which the above assumptions do not apply.

Fracture of anisotropic rock

An extreme example of a rock in which inherent cracks are not randomly distributed is slate. If it is assumed that slate contains two crack systems, one preferentially oriented system of large bedding plane cracks and one randomly oriented set of small grain boundary cracks, it becomes possible to calculate the stress levels at which fracture would initiate under various conditions[13 14].

Figure 4 illustrates the remarkable agreement between the predicted and observed fracture behaviour of slate specimens subjected to uniaxial compression. It will be noted that the highest strength of slate can be as much as four times its lowest strength, depending upon the orientation of the bedding planes to the direction of applied load.

An important practical conclusion which can be drawn from Figure 4 is that a comparison of the results obtained from compression tests on core drilled normal to and parallel to the bedding planes does not necessarily determine whether the material is anisotropic - a procedure sometimes advocated by those concerned with "practical" tests. In the case of slate, the compressive strength of specimens drilled normal to and parallel to the bedding planes is almost the same, and if the strength of the specimen in which the bedding planes are oriented at 30 degrees to the direction of applied load is not taken into account, one may be tempted to conclude that slate is isotropic. This extreme example is included to demonstrate the dangers involved in drawing conclusions from inadequate test data.

Table 1. Summary of Triaxial Test Results on Rock and Concrete

Graph Point	Material	Uniaxial Compressive Strength in lb./sq. in	Tested by
1	Marble	13 700	Ros and Eichinger
2	Marble	18 000	Ros and Eichinger
3	Marble	20 000	Von Karman
4	Carthage Marble	10 000	Bredthauer
5	Carthage Marble	7500	Bredthauer
6	Wombeyan Marble	10 000	Jaeger
7	Concrete	2 380	McHenry and Kami
8	Concrete	3 200	Akroyd
9	Concrete	6 000	Jaeger
10	Concrete	5700	Fumagalli
11	Concrete (28 day)	3510	Balmer
12	Concrete (90 day)	4 000	Balmer
13	Granite Gneiss	25 500	Jaeger
14	Barre Granite	24200	Robertson
15	Granite (slightly alt)	10 000	Wreuker
16	Westerly Granite	33 800	Brace
17	Iwaki Sandstone	1 780	Horibe and Kobayashi
18	Rush Springs andstone	26 000	Bredthauer
19	Pennant Sandstone	22 500	Price
20	Darley Dale Sandstone	5780	Price
21	Sandstone	9 000	Jaeger
22	Oil Creek Sandstone	**	Handin
23	Dolomite	24000	Bredthauer
24	White Dolomite	12 000	Bredthauer
25	Clear Fork Dolomite	**	Handin
26	Blair Dolomite	**	Handin
27	Blair Dolomite	75 000	Brace
28	Webtuck Dolomite	22 000	Brace
29	Chico Limestone	10 000	Bredthauer
30	Virginia Limestone	48 000	Bredthauer
3i	Limestone	20 000	Jaeger
32	Anhydrite	6 000	Bredthauer
33	Knippa Basalt	38 000	Bredthauer
34	Sandy shale	8 000	Bredthauer
35	Shale	15 000	Bredthauer
36	Porphy	40 000	Jaeger
37	Siouxyr Quartzite	**	Handin
38	Frederick Diabase	71 000	Brace
39	Cheshire Quartzite	68 000	Brace
40	Chert dyke material	83 000	Hoek
4i	Quartzitic shale (Dry)	30 900	Colback and Wiid
42	Quartzitic shale (Wet)	17 100	Colback and Wiid
43	Quartzitic sandstone (dry	9070	Colback and Wiid
44	Quartzitic sandstone (wet)	4970	Colback and Wiid
45	Slate (primary cracks)	4300	Hoek
46	Slate (secondary cracks)	15 900	Hoek
47	Dolerite	37 000	CSIR
48	Quartzite (ERPM Footwall)	31 000	CSIR
49	Quartzite (ERPM Hanging wall)	43 200	CSIR
50	Glass	91 000	CSIR

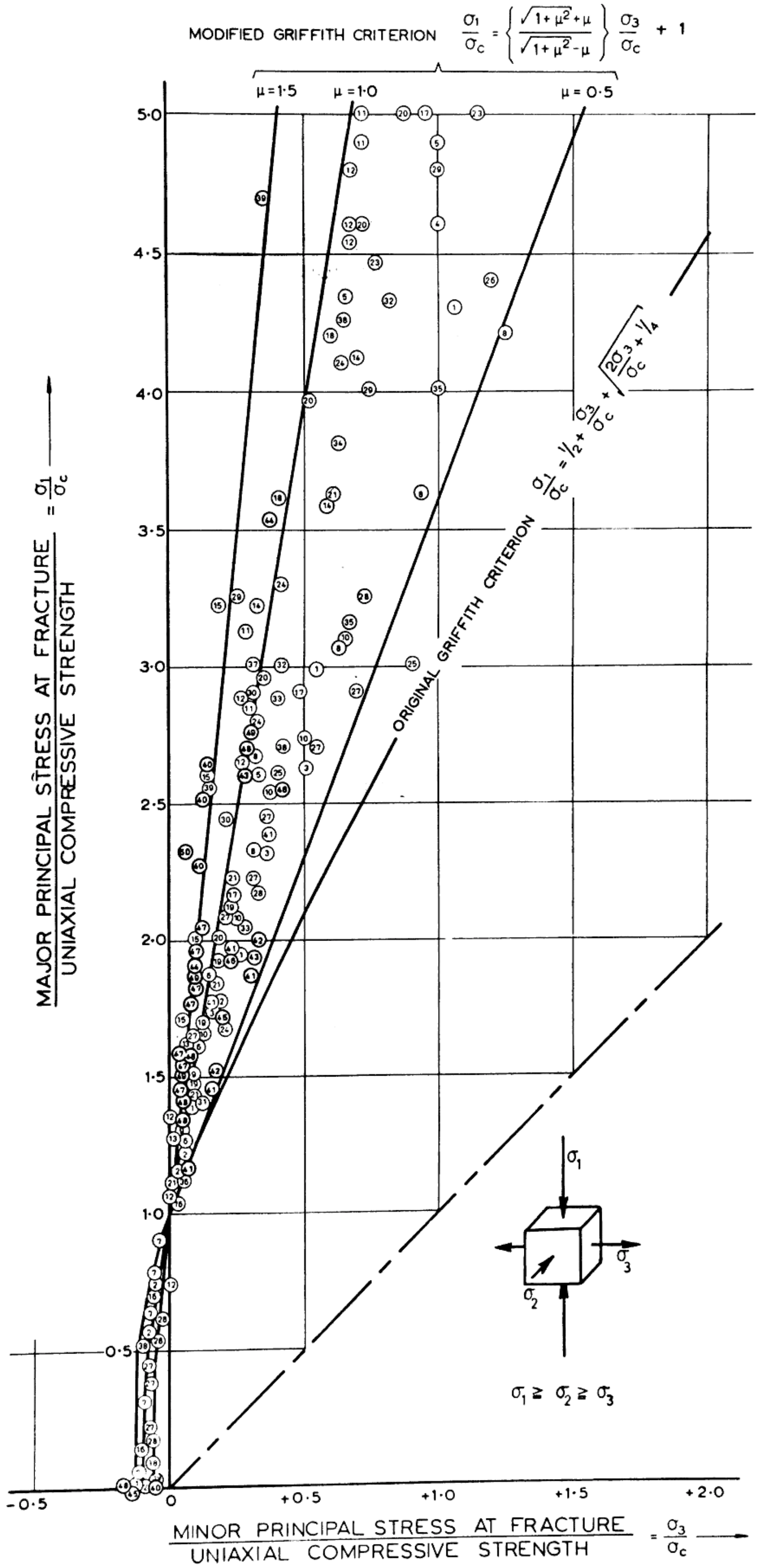

Figure 2. Triaxial fracture data for 50 rock and concrete materials listed in Table

Slate is a relatively simple example of an anisotropic material, having only one set of dominant planes of weakness. The same theoretical process as was applied in the case of slate could be extended to describe the behaviour of a material such as coal which may have two or more cleat systems in addition to its bedding planes.

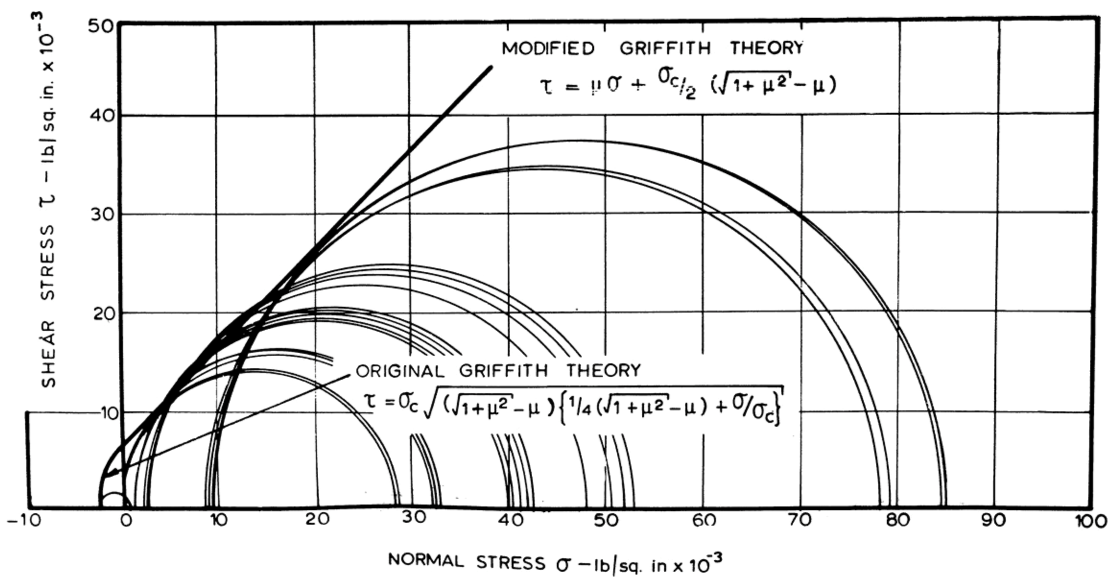

Figure 3. Mohr fracture diagram for typical Witwaterstrand quartzite with a uniaxial compressive strength j_c = 30,000 p.s.i. and a coefficient of internal friction $\mu = 1.00$

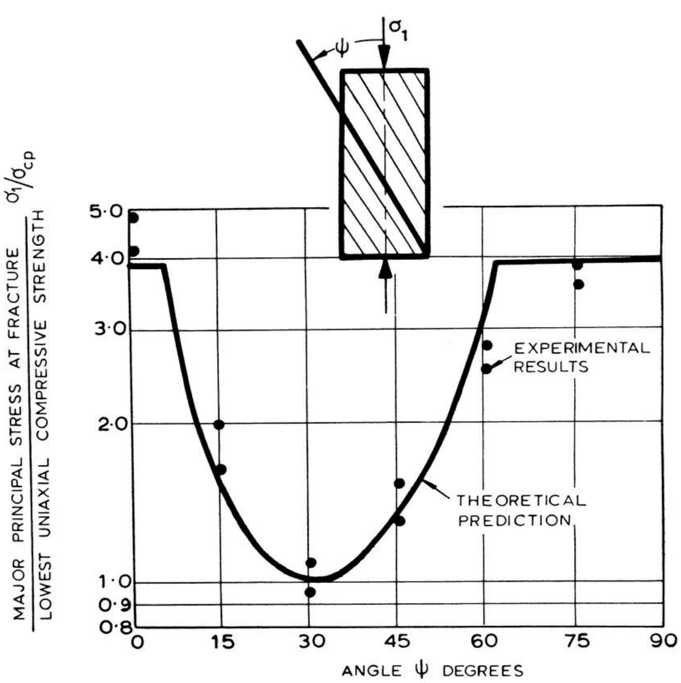

Figure 4. Relationship between bedding plane orientation and strength of slate

Influence of environment upon rock strength

It is frequently assumed that the strength of rock is not significantly influenced by the temperature or humidity of its surroundings. It has, however, been demonstrated that this assumption, particularly on the influence of humidity, can be seriously in error[15].

The influence of temperature upon the strength of rock is probably not significant within the normal range of temperatures encountered by the civil or mining engineer. However, at great depths where the temperatures approach the melting point of some of the rock constituents, the reduction in strength may be significant and could be of importance to those concerned with the overall behaviour of the earth's crust and with the origin of deep-level earthquakes.

The influence of moisture upon the strength of rock is so important that the author advocates that tests on coal and soft rocks should be carried out on site. In order to minimise changes in the moisture content of the specimen, the tests should be carried out as soon as the specimens have been removed from the parent rock. The practical details of this type of test will be discussed later.

The influence of moisture content upon the strength of quartzitic shale is illustrated in Figures 5 and 6 which are reproduced from a paper by Colback and Wiid[15]. It will be seen that a saturated specimen of this quartzitic shale is only half as strong as a dry specimen. Colback and Wiid have postulated that this reduction in strength is due to a reduction in the molecular cohesive strength of the rock material when moisture is present.

The practical importance of the influence of moisture upon the strength of rock in structural rock mechanics is the danger of a normally stable structure becoming unstable in wet conditions. In comminution, the strength reduction obtained under wet conditions results in more efficient cutting or drilling.

Influence of fluid pressure

In addition to the strength reduction associated with a high moisture content, a further threat to the stability of a rock structure occurs when water is present under pressure. This fluid pressure reduces the compressive stress acting across a fissure or fracture plane and hence the frictional resistance which causes interlocking of blocks of rocks can be reduced. In an extreme case, one block can be literally floated off another by the buoyance effect of water pressure. The role of fluid pressure in determining the strength of a rock mass is fairly well understood[16] and its influence can be allowed for in strength calculations.

The practical importance of the influence of moisture upon the strength of rock in structural rock mechanics is the danger of a normally stable structure becoming unstable in wet conditions. In comminution, the strength reduction obtained under wet conditions results in more efficient cutting or drilling.

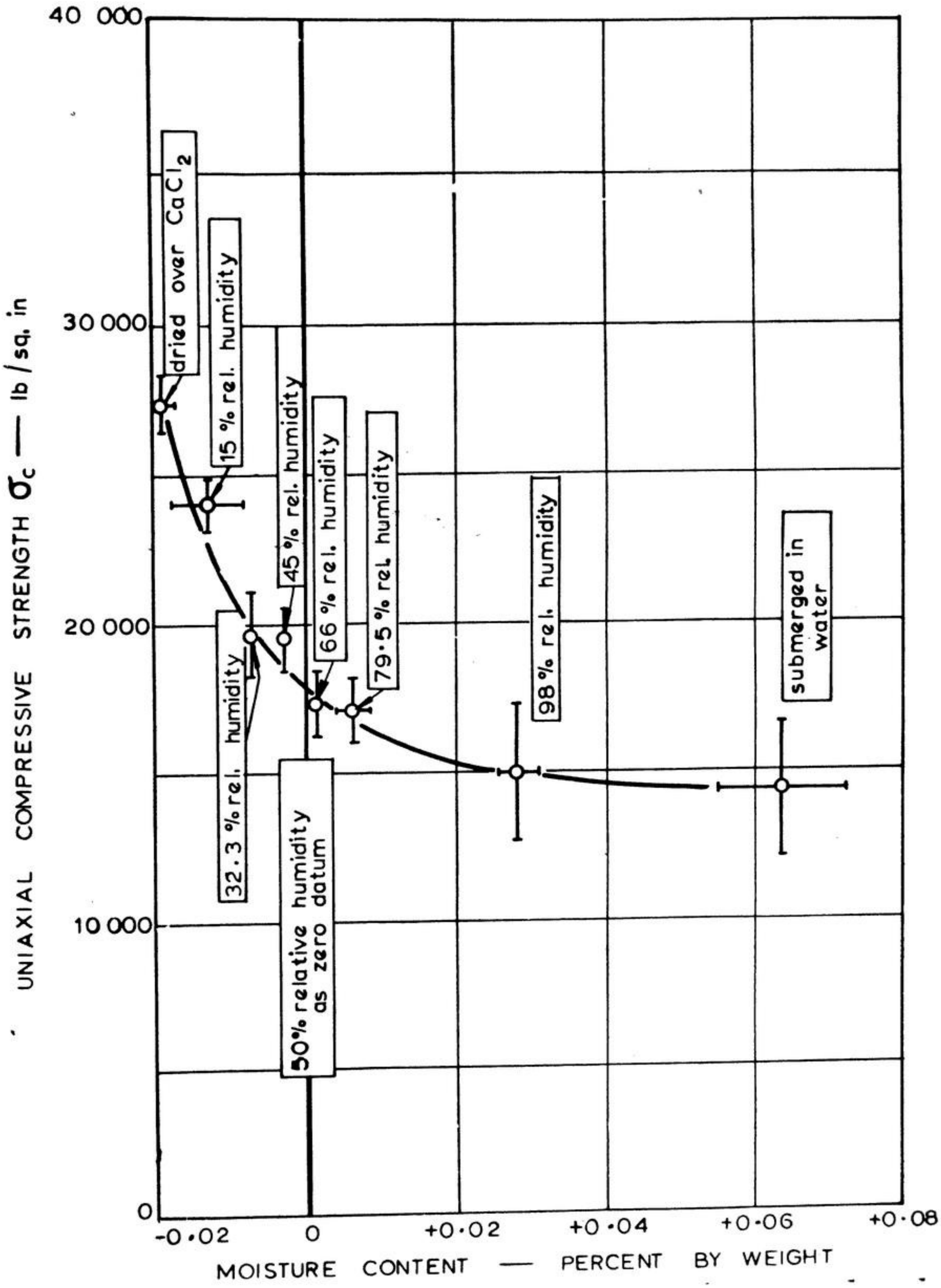
40 000
30 000
20 000
10 000
0
UNIAXIAL COMPRESSIVE STRENGTH σc — lb/sq. in
dried over CaCl2
15 % rel. humidity
45 % rel. humidity
66 % rel. humidity
79·5 % rel. humidity
98 % rel. humidity
submerged in water
32.3 % rel. humidity
50% relative humidity as zero datum
-0·02
0
+0·02
+0·04
+0·06
+0·08
MOISTURE CONTENT — PERCENT BY WEIGHT

Figure 5. Relationship between uniaxial compressive strength and moisture content for quartzitic shale specimens (Colback & Wiid).

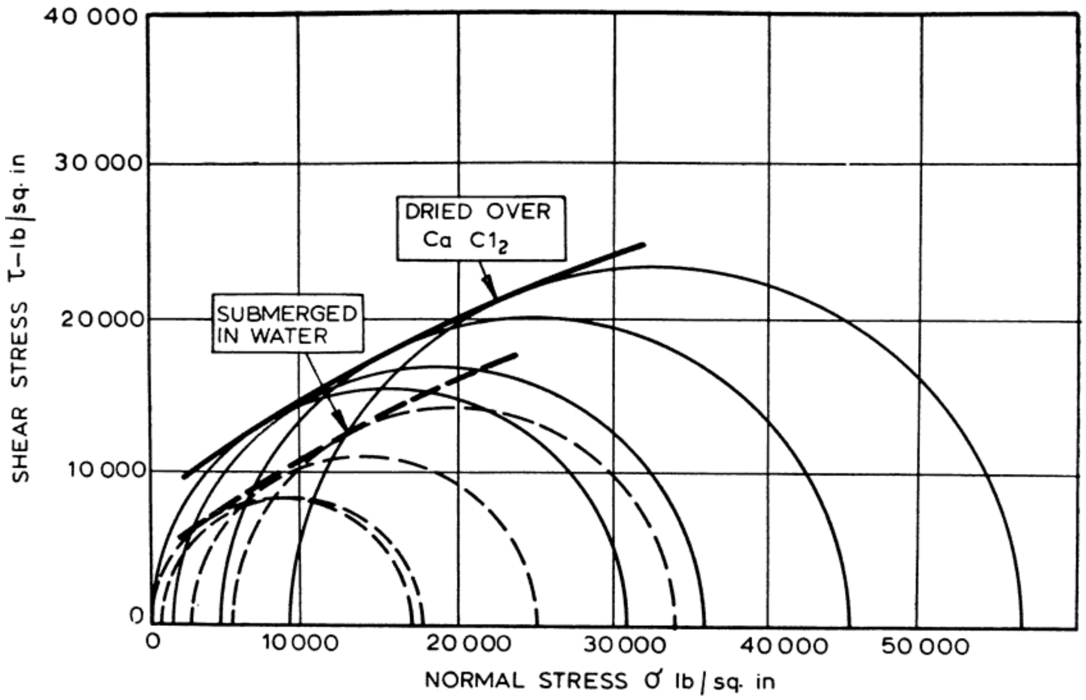

Figure 6. Mohr fracture envelopes showing the effect of moisture on the compressive strength of quarzitic shale (from Colback & Wiid)

Influence of fluid pressure

In addition to the strength reduction associated with a high moisture content, a further threat to the stability of a rock structure occurs when water is present under pressure. This fluid pressure reduces the compressive stress acting across a fissure or fracture plane and hence the frictional resistance which causes interlocking of blocks of rocks can be reduced. In an extreme case, one block can be literally floated off another by the buoyance effect of water pressure. The role of fluid pressure in determining the strength of a rock mass is fairly well understood[16] and its influence can be allowed for in strength calculations.

Time-dependent failure of rock

One of the least understood aspects of the mechanical behaviour of rock is the influence of time upon its deformation and fracture. It is convenient to consider this subject under two headings:

a. Weathering which is the gradual deterioration of an exposed rock surface. This deterioration may take place in the absence of applied stress and is due mainly to physical and chemical processes which are governed by the environment to which the rock is exposed;

b. time-dependent mechanical behaviour which involves the deformation or fracture of rock under conditions of constant applied stress (frequently referred to as “creep”, a term which the author avoids because of the possible confusion with the process of creep in metals which need not be the same as in rock).

In most practical cases, both of these phenomena play a part and it is usually impossible to establish which of the two is the main cause of failure.

Experience in South Africa shows that coal pillars usually fail a number of years after they have been mined. Examination of the pillars and measurements of time-dependent deformation suggests that both weathering and time-dependent failure are important.

Consequently, one must conclude that a full understanding of the time-dependent behaviour of rock must involve a knowledge of both weathering and time-dependent failure. To the best of the author's knowledge, no complete and systematic study of the process of rock weathering has ever been undertaken and this deficiency presents an interesting challenge to rock mechanics research workers.

Time-dependent mechanical behaviour of rock has been fairly intensively investigated, both theoretically[17] and experimentally[18], but a great deal more remains to be done before this knowledge can be effectively applied to practical problems. For the engineer faced with the problem of designing a rock structure in which weathering or time-dependent failure may be important, the most realistic approach appears to be to use the results of short time laboratory tests with a liberal allowance for the possible strength reduction with time.

The extent to which the strength will reduce with time depends upon so many unknown factors that no definite rules could be suggested but the author believes that under severe conditions, a reduction of 50 per cent over a ten year period is possible.

Influence of Specimen Size

It is accepted that the strength of a brittle material is dependent upon the size of the test specimen and yet very little reliable quantitative data on this effect is available. In the author's opinion, the most rational approach to this problem is that adopted by Protodiakonov[19] and his most important conclusions are presented in Figure 7.

The parameter 'm', as defined in Figure 7, depends upon the nature of the material and also upon the state of stress to which the specimen is subjected. From the few experimental data which are available the author has made the following estimates of the value of 'm':

Coal and soft rocks subjected to compression	$5 < m < 10$
Coal and soft rocks subjected to tension	$10 < m < 50$

Hard rock subjected to compression	2 < m < 5
Hard rock subjected to tension	5 < m < 10

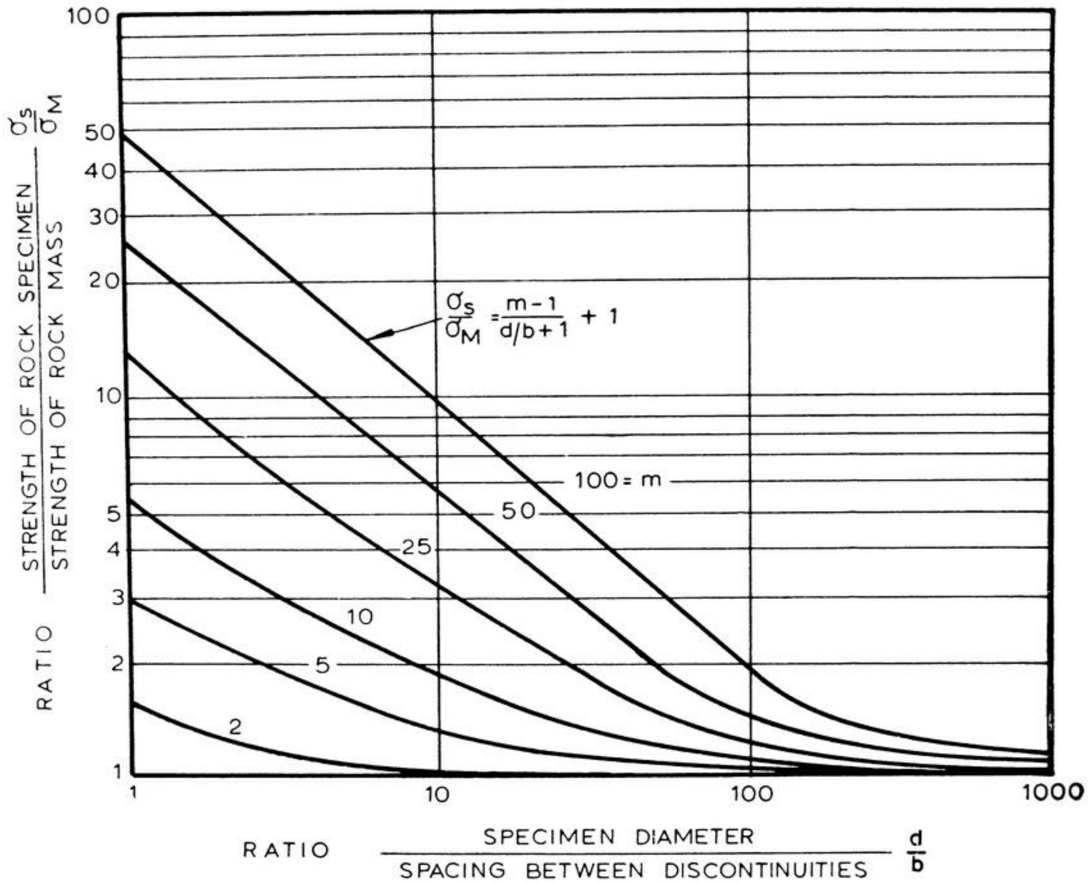

Figure 7. Relationship between specimen size and strength according to Protodiakonov

While these estimates must be treated with extreme caution, they do enable one to arrive at an order of magnitude for the acceptable specimen size.

Hence for a material such as coal subjected to compression, the specimen size should be approximately 50 to 100 times the spacing between discontinuities if the strength of the specimen is to be within 10 per cent of the rock mass. Since the spacing between discontinuities in may, as a first approximation, be equated to the cleat spacing which can be in the order of 2 inches, the specimen diameter required is estimated at between 100 and 200 inches.

This size of specimen would daunt even the most courageous exponent of large-scale testing and yet, if Protodiakonov's deductions and the author's estimates are reasonably accurate, it may have to be accepted at most tests on coal carried out on specimens which are far too small to give meaningful results.

In a series of large-scale tests planned by the Coal Mining Research Controlling Council of South Africa in conjunction with the Council for Scientific and Industrial Research. It is proposed to test coal specimens to 8 feet cube (the technique to be used will be discussed later). The results of these tests should contribute towards a more reliable evaluation of the problem of the size effect in strength testing.

In the case of hard rocks such as the Witwatersrand quartzites, the value of 'm' for compression is believed to lie between 2 and 5 and the spacing between discontinuities, in this case assumed to be the grain boundaries, may be of the order of 1/10 of an inch. Hence, from Figure 7, the diameter of specimen required to give a strength value within 10% of that for the massive rock (excluding the effect of major discontinuities) is between one and 2 inches.

Stability of Rock Structures

In designing a rock structures such as the damp foundation or underground excavation, the most important consideration is the stability of the entire structure. Local rock failure at the points of high stress or in zones of exceptionally low strength may only be significant if this failure forms part of a sequence of events, which leads to collapse of the structure.

The obvious question which the practical engineer, will ask is – what are the main factors which govern the stability of a rock structure and what information of these factors and their interaction does the science of rock mechanics provide?

Two main problem types

In discussing this question it is necessary to distinguish between two main types of structural rock mechanics problems:

a. Underground excavations in solid homogeneous rock, such as the massive quartzites which occur in the deep-level gold mines in South Africa, in which the stability of the rock surrounding the excavation is primarily dependent upon progressive failure of the rock material.

b. Rock structures such as dam foundations and surface excavations in which the material is so faulted and fissured that the stability of the structure depends upon the movement of interlocking blocks within the rock mass rather than upon failure of the rock material.

Progressive failure of the rock surrounding an excavation

When a rock specimen is loaded in a hydraulically actuated testing machine, the behaviour of the specimen at the point of fracture is obscured by the behaviour of the testing machine.

The release of the energy stored in the hydraulic system usually results in violent disintegration of the rock specimen with the consequent loss of all post-fracture data. On the other hand, an elevator of rock which forms part of a rock mass and which is subjected to an identical state of stress to that applied by the testing machine will behave in an entirely different man out, what is the point of failure has been reached.

In his case, disintegration of the rock element may be arrested by the transfer of load onto an adjacent development, which previously carried a lower load. In this way, local fracture induces a redistribution of stress, which may or may not result in further fracture.

In an attempt to understand this process of progressive failure, the author has studied the initiation and propagation of fracture in rock under non-uniform stress conditions such as those which occur around underground excavations[20].

An example of this type of analysis is presented in Figure 8 in which the theoretical fracture contours in the rock surrounding square and elliptical tunnels are compared. In deriving these contours it was assumed that fracture initiation in the material, assumed to be a homogeneous quartzite, is defined by the Mohr envelope illustrated in Figure 3. This fracture criterion is combined with the stress distribution in the material surrounding the excavations[21] to give the fracture contours and the critical crack trajectories[20].

The fracture contours are plotted in terms of the vertical pressure P which would have to be applied to the material surrounding the excavation in order to cause initiation of fracture at any point. Hence, if the fracture contours has a value P = 10,000 p.s.i., all the material enclosed by this contour, i.e. the material in which the fracture contours have lower values, would be prone to failure if the vertical applied pressure P was 10,000 p.s.i..

The vertical applied pressure P is due primarily to the weight of the overburden, increasing at a rate of approximately one p.s.i per foot of depth. The local pressure P, which acts upon a particular time or former may, however, be intensified by mining activities in the vicinity of this tunnel Hence, at a time situated in the pillar may be subjected to a vertical applied pressure P or 15,000 p.s.i., while the pillar itself may be only a subjected to a nominal pressure, due to its depth below the surface, of 7000 p.s.i..

The critical crack trajectories define the most dangerous crack orientation at any point in the material surrounding the tunnel. An inherent flaw or discontinuity oriented in this direction would fail at a lower stress level than that required to initiate failure from any other floor, of similar size, but at a different orientation, in the vicinity.

The significance of the fracture contours and critical crack to trajectories, illustrated in Figure 8, is interpreted in more practical terms in Figure 9. In both cases, the lowest fracture contour occurs in the roof (and floor) of the excavation; hence failure can be expected to initiate at these points. Since the critical crack trajectories in this region lie parallel to the vertical access of the excavation, a vertical crack will occur as shown in Figure 9a.

Experimental studies of the formation of vertical roof and floor cracks[22] have shown that, apart from a redistribution of the stress and the proximity of the crack from the stability of excavation is not endangered by the formation of these cracks. It must also be pointed out that the tendency for these cracks to form is markedly reduced by the presence of natural applied stress due to the restraint of the surrounding rock[23].

The initiation of sidewall fracture, illustrated in Figure 9b is dependent upon the geometrical shape of the excavation. In the case of the square tunnel, the high stress concentration in the sharp corner results in fracture initiation at a vertical applied pressure P > 7500 p.s.i.. The critical trajectories suggest that this fracture would propagate in such a way that "slabbing" of the site wall would occur.

The occurrence of this sidewall slabbing is illustrated in Figure 10, which shows a haulage in quartzite at a depth of 9500 feet below surface in a South African gold mine. Figure 10a shows the haulage in an area removed from mining activity and Figure 10b shows the sidewall fracture associated with an increase in stress level due to extraction of the reef immediately above the haulage.

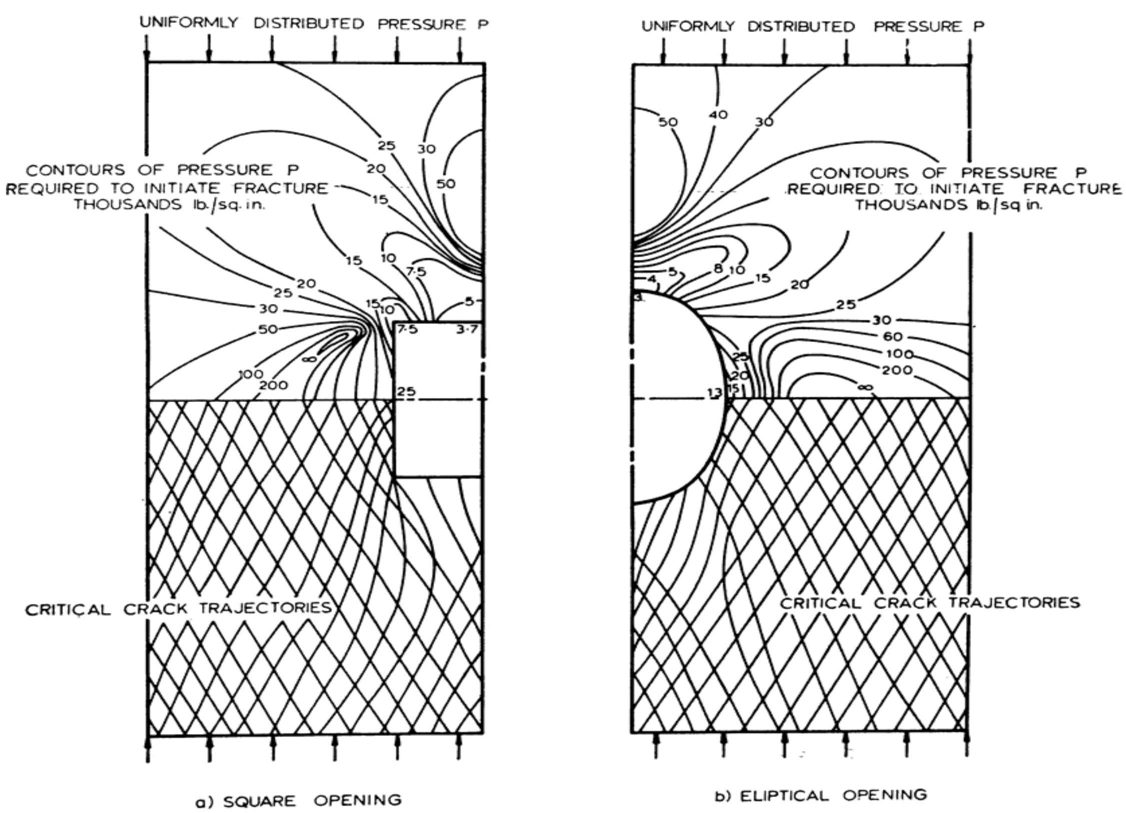

Figure 8. Fracture contours and critical crack trajectories in quartzite surrounding square and elliptical tunnels

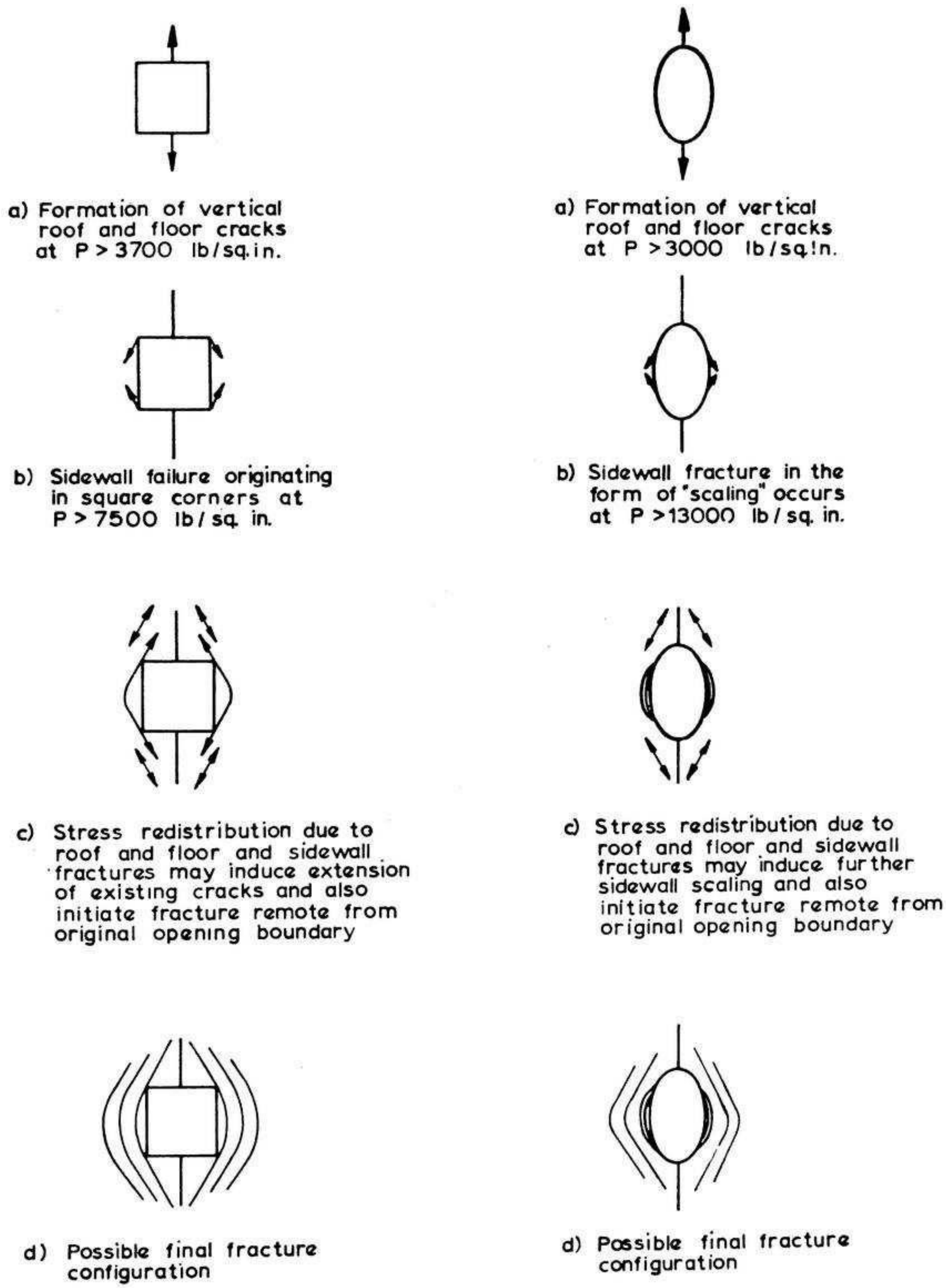

Figure 9. Possible fracture sequence for square and elliptical tunnels in hard quartzite subjected to vertical pressure P only

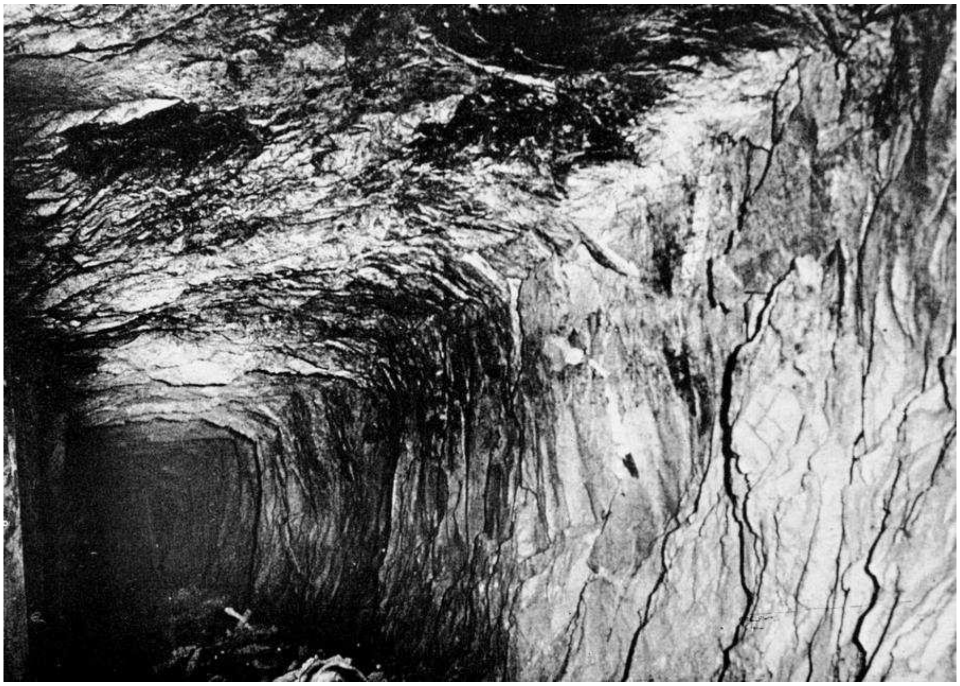

Figure 10a. Square tunnel in quartzite remote from mining activities. Depth of 9,500 ft. below surface in a South African gold mine

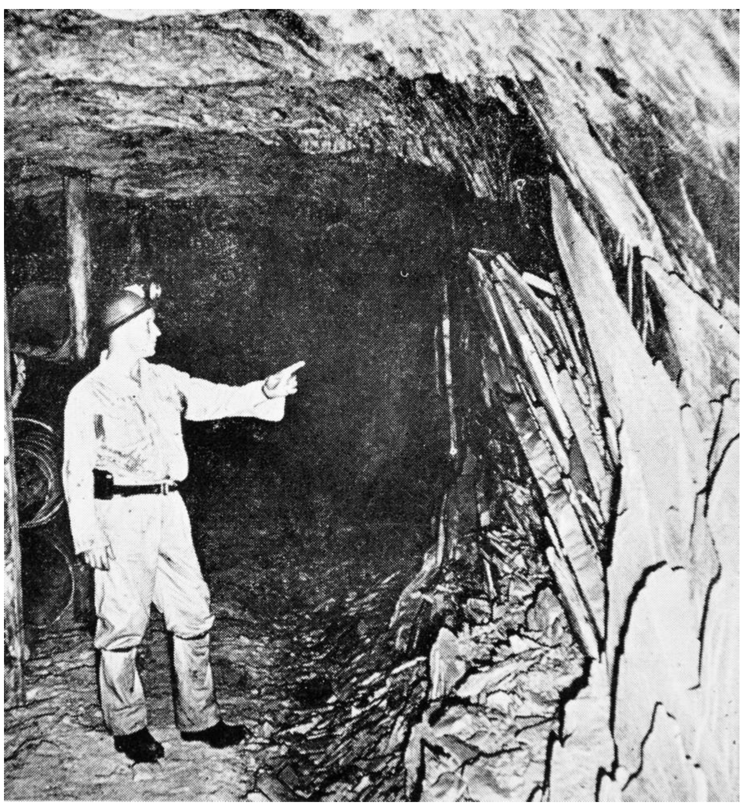

Figure 10b. Sidewall slabbing in a square tunnel in quartzite which had been subjected to a pressure increase due to over-mining. Depth below surface 9,500 ft.

In the case of the elliptical tunnel, sitewall failure can be anticipated at P > 13,000 p.s.i. And this it would probably take the form of a sidewall "scaling" as shown in Figure 9b. The fact that this scaling occurs at such high stress levels is important. Practical experience in deep level mines which use both square and elliptical tunnels confirms that the elliptical tunnel, at the same depth, has a lower tendency to sidewall failure and requires less maintenance and support in the form of rockbolting. Evidence from model studies[22] suggest that the next stage in failure might be associated with the stress redistribution in the roof and the floor and may follow the pattern illustrated in Figure 9., resulting in the final fracture configurations illustrated in figure 9d.

It must be emphasised that this analysis has been carried out on the assumption that the excavations are situated in solid, homogeneous quartzite, which is subject to vertical pressure only. The presence of a fault, Fissure or marked anistropic strength behaviour would have a significant influence upon the analysis and would invalidate the conclusions reached.

A further difficulty, which has emerged from fracture studies on models[20], is that the stress distribution in the rock around an excavation is drastically altered by failure of rock. In order to overcome this difficulty, it is necessary to re-analyse the stress distributions at each new boundary condition created by propagation of the fracture. While this form of analysis has been done[20], it is, at present, far too complex and tedious to the practical value. This example has been quoted to illustrate an approach which the author believes may lead to a better understanding of the problems of stability of rock structures.

Another approach, still in its early stages, has been adopted by Deist[24] and Cook[27]. This approach is based upon the assumption that, beyond a certain level of stress (failure), the load carrying capacity of an element of rock decreases linearly with increasing strain. This assumption has been experimentally verified by Cook using a specifically designed to "stiff" testing machine[26]. Diest has carried out theoretical studies into the stability of the fracture-solid rock boundary around excavations and, while, as yet only simple cases have been analysed, promising results have been obtained.

A recent paper by Fairhurst and Cook[27] suggests that a factor which may play an important part in determining the stability of the rock around a mining excavation is the buckling of sidewall slabs such as that those illustrated in Figure 10 b.

It will have become obvious to the reader that, as yet, the engineer who is faced with the problem of designing a stable structure is very little practical information at his disposal. At the same time, it will be appreciated at a start has been made in determining the most important factors governing the stability of rock structures and that the problem, having been defined, will be subjected to concentrated study until an unacceptable practical solution has been achieved.

Stability of a rock mass

To the best of the author's knowledge there are, as yet, no general rules analysing the stability of a rock mass. In any case, rock structures in which the rock is fissured and faulted can take such a wide variety of forms that the only logical approaches to treat each individual case on its merits.

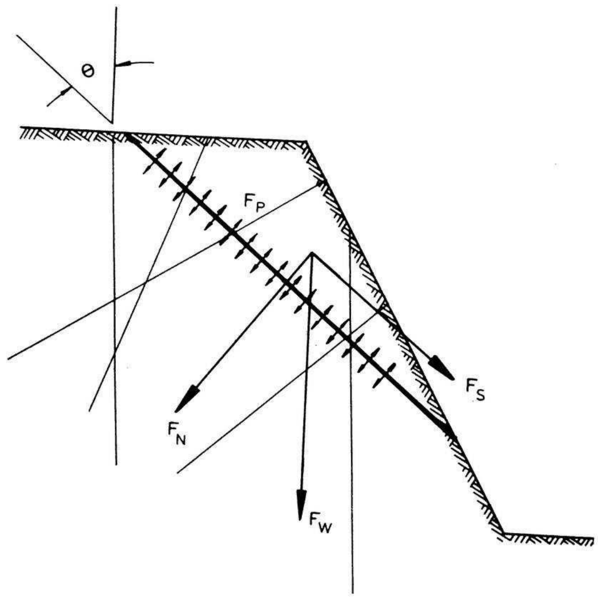

Figure 11. Forces acting on a rock slope containing a geological discontinuity

A practical example of such treatment, derived from actual experience, is presented in Figure 11, which illustrates a rock slope in which several geological discontinuities are known to exist. A cursory examination of this figure shows that only one of these discontinuities (marked as a heavy line) represents a threat to the stability of the system. The forces acting on this potential failure plane are as follows:

F_W – the total weight of the rock wedge
F_N – the normal force acting across the plane F_S
– the shear force acting along the plane
F_P – the total force due to fluid [pressure (if present) within the rock mass. The

condition for unstable equilibrium is

$$F_S = \mu \left(F_N - F_P\right)$$

where is the coefficient of friction which is effective along the potential failure plane. The requirement for stability of the slope can be expressed as follows:

$$\mu > \frac{\cos\theta}{\sin\theta - F_P / F_W}$$

From the limited amount of experimental data available, it is believed that the coefficient of friction, between two dry rock surfaces is in the region of 0.7. This value could be considered lower if the potential failure plane contains soft filling material or if the rock services had deteriorated.

Control of Fracture in a Rock Structure

Rock has several physical characteristics which distinguishes it from other materials, and, of these, the most important structural characteristics are:

(a) The large difference between the tensile and compressive strengths.

(b) The rapid increase in strength with increasing natural or confining pressure.

(c) The tendency for the structural properties of some rocks to deteriorate with time due to weathering.

Bearing these characteristics in mind, the engineer has the following techniques to add his disposal for making optimum use of its materials:

(i) The design of the structure to avoid, if possible concentrations of tensile stress.

(ii) The use of support means methods which increase the strength of the structure by increasing lateral pressure or restraint in zones of high compressive stress.

(iii) The protection of surfaces which are liable to weather.

Design of rock structures for optimal stress distribution.

In mining, the geometry of the orebody being mined dictates the layout of the major excavations within the mine. The only practical control which the engineer can exercise in this case is in support methods and in the sequence of mining[2]. However, in the case of secondary excavations such service haulages, airways, pumping chambers, etc, a mine designer usually has a reasonable amount of freedom, which he can use to position these excavations were the most favourable stress distribution.

The calculation of the stress distribution around excavations is a complex process[21] which would certainly not benefit the average engineer to attempt to learn. Fortunately, there are several relatively simple model techniques which can be used, either by the engineer himself or by a university or research organisation, which could undertake this work on a contract basis, to give solutions which are sufficiently accurate and practical design purposes. These techniques are discussed in Part 2.

In the case of civil engineering structures such as damp foundations for open-cast excavations, the engineering is usually in a position to design the structure of optimum stress conditions (in relation to the structural material at his disposal).

In this case, the low tensile strength of rock, particularly if it contains unfavourably oriented flaws and fissures, must be kept in mind and the structure designed to minimise tensile stress is as far as possible. Since this subject falls outside the scope of the author's experience, the interested reader is referred to the work of Muller[28] and Serafim[29] for further details on the application of rock mechanics to do to the design of civil engineering structures.

Use of support in rock structures

In this context, support denotes the use of materials other than rock to improve the local properties of the rock and thereby improve the stability of the structure as a whole. Hence, for example lining a shaft with concrete is frequently used to inhibit local deterioration and failure of the rock surface.

Generally structural support its used a) to support broken rock and to prevent it from breaking and b) to improve the strength of rock mass by increasing lateral stress in selected areas.

In the first case, the use of timber, steel or concrete structures or of rock bolts is familiar to any engineer who has been concerned with mining or tunnelling. The main purpose of this support is to minimise the danger of falling rock and to prevent a loose pieces of rock from choking a passage-way. The use of this type of support depends so much upon local conditions that it would be meaningless to attempt to formulate general rules. On the other hand, the use of support to increase the strength of rock is a less well-known concept and deserves close examination.

One of the principal features which distinguishes rock from other structural materials, is the stress required to cause failure. In the case of quartzite, the stress fracture increases by six units per every unit increased in confining pressure. Hence, for quartzite with an unconfined compressive strength of 30,000 p.s.i., the stress required to fracture of specimen, subjected to a confining pressure of 1000 p.s.i., would be 36,000 p.s.i..

If therefore, it is required to increase the strength of rock in a particular area, this can best be achieved by artificially increasing the triaxial compressive stresses acting on this area. This increase can be achieved by the use of such devices as rock bolts or steel arches.

High tensile steel rockbolts have been used successfully in South Africa to inhibit sidewall fracture of excavations. Rockbolting can be particularly effective when joints are present in the rock since bolts placed normal to the joint plane increase the resistance of the joint to shear movement by increasing the friction forces and also by the pinning action of the bolts themselves. The use of steel arches in tunnels provides effective support for the broken roof rock and also increases the strength of the sidewall. The volume increase associated with early sidewall fracture results in a packing up of broken between the steel arch and the solid rock surface. The restraint provided by these rock fragments is usually adequate to prevent further sidewall failure.

Protection of rock surfaces from weathering

Weathering is the term for the rapid deterioration of several types of rocks when they are exposed to the atmosphere. Therefore, a rock that seems to have good structural qualities when it is first exposed might not be acceptable after being left there for a few months. According to the author's experience, soft rock specimens that were exposed to the atmosphere in between testing and preparation have occasionally crumbled into piles of debris in a matter of days.

Most of the time, weathering can be effectively prevented or at least inhibited with fairly basic corrective actions. Enough protection can be obtained by regularly painting or misting the exposed area with a thin solution of cement and water. A reasonably thick layer of cement, concrete, or a similar sealant may be necessary for rocks that are especially vulnerable to weathering. Some coal mines in South Africa have found success with a commercially available device that sprays foam plastic onto exposed rock surfaces.

Summary and Practical Conclusions

The following characteristics distinguish rock from other commonly used structural materials, apart from concrete, which behaves in a similar manner.

a) Rock failure is generally of a brittle type, ie, it occurs with little prior warning and is not accompanied by a large non-elastic (plastic) deformations.

b) The tensile strength of rock is usually of the order of 1/10 of its unconfined compressive strength. The strength of rock increases rapidly with increasing confining pressure.

c) Some rocks exhibit marked anisotropic strength behaviour due to the existence of preferentially oriented weakness planes such as the bedding planes in a sedimentary rock.

d) Most rocks exhibit some degree of time-dependent deformation or fracture behaviour.

e) Some rocks suffer from a serious strength reduction when wet.

h) Fresh rock surfaces exposed to the atmosphere are liable to deteriorate in time (weather).

The most effective use of the structural characteristics of rock can be achieved if these physical properties are kept in mind and if necessary precautions are taken in designing a rock structure. Some of these precautions are:

1) The structure should be designed to avoid or, at least, minimise zones of high tensile stress.

2) Support techniques such as the use of rockbolts can be used to improve the stability of rock structure by increasing the confining pressure and hence to compressive strength of the material.

3) When a rock structure contains geological weaknesses in the form of bedding planes, fissures, faults, joints or dykes, due allowance must be made for the directional strength variation associated with these features.

4) In calculating the strength of rock structure, allowance should be made for the possible reduction in strength with time.

5) Exposed surfaces of rocks which are particularly liable to weathering should be protected, as far as possible, from exposure to the atmosphere.

Current activity in rock mechanics, throughout the world, is such that significant advances in knowledge can be anticipated during the next decade. The interested reader is therefore advised to keep in contact with rock mechanics literature, and with the proceedings of the conferences which take place from time to time.

List of References

1) Judd, W. R. (1963). Rock stress, rock mechanics and research. Proc. Intnl. Conf. on the State of Stress in the Earth's Crust. Santa Monica, ed. W. R. Judd. Elsevier Publishing Co., New York, 1964, PP- 5-53.

2) Cook, N.G.W. et al. (1966). Rock mechanics applied to rockbursts— a synthesis of the results of rockburst research in South Africa up to 1965. Journal South African Institution of Mining and Metallurgy, Vol. 66, No. 10, pp. 435-528.

3) Griffith, A. A. (1925). Theory of rupture. First Intnl. Cong. Appl. Mech. Delft, 1924. Technische Boekhandel en Drukkerij, ed. J. Waltman Jr., pp. 55-63.

4) McClintock, F. A. and Walsh, J. B. (1962). Friction on Griffith cracks in rock under pressure. Intnl. Cong. Appl. Mech. Berkeley, American Soc. Mech. Engrs., New York, 1963, pp. 1015-1021.

5) Hoek, E. (1965). Rock fracture under static stress conditions. Ph.D. Thesis, University of Cape Town, 1965, also South African Council for Scientific and

Industrial Research Report MEG 383.

6) Brace, W. F. (1961). Dependence of fracture strength of rocks on grain size. Penn. State Univ. Mineral Expt. Sta. Bull., No. 176, 1961, pp. 99-103.

7) Berg, C. A. (1965). Deformation of fine cracks under high pressure and shear. J. Geophysical Res., Vol. 70, No. 14, pp. 3447-3452.

8) Brace, W. F. and Bombolakis, E. G. (1963). A note on brittle crack growth in compression. J. Geophysical Res., Vol. 68, No. 12, pp. 3709-3713.

9) Hoek, E. and Bieniawski, Z. T. (1965). Brittle fracture propagation in rock under compression. Intnl. J. Fracture Mechanics, Vol. 1, No. 3, pp. 137-155.

10) Hoek, E. and Bieniawski, Z. T. (1966). Fracture propagation mechanism in hard rock. Proc. First Intnl. Congress on Rock Mechanics, Lisbon.

11) Walsh, J. B. (1965). The effect of cracks on the uniaxial compression of rocks. J. Geophysical Res., Vol. 70, No. 2, pp. 399-411.

12) Walsh, J. B. (1965). The effects of cracks on the compressibility of rocks. J. Geophysical Res., Vol. 70, No. 2, 1965, pp. 381-389.

13) Hoek, E. (1964). Fracture of anisotropic rock. J. S. Afr. Inst. Min. Metall., Vol. 64, No. 10, pp. 510-518.

14) Walsh, J. B. and Brace, W. F. (1964). A fracture criterion for anisotropic rock. J. Geophysical Res., Vol. 69, No. 16, 1964, pp. 3449-3456.

15) Colback, P. S. B. and Wiid, B. L. (1965). The influence of moisture content on the compressive strength of rock. Proc. Third Canadian Symposium on Rock Mechanics, Toronto.

16) Nicolyasen, O. (1965). The role of fluid pressure in the failure of rock and other porous media. Unpublished report Bernard Price Institute of Geophysical Research, July.

17) Robertson, E. C. (1964). Viscoelasticity of rocks. Proc. Intnl. Conf. State of Stress in Earth's Crust. Santa Monica, 1963, ed. W. R. Judd. Elsevier Publishing Co., New York, pp. 181-224.

18) Hardy, H. R. (Jnr.). (1965). Study of the inelastic behaviour of geologic materials. Ph.D. Thesis. Virginia Polytechnic Institute, June.

19) Protodiakonov, M. M. (1964). Methods of evaluation of cracks and strength of rock systems at depth. Proc. 4th Intnl. Conf. Strata Control and Rock Mechanics. New

York, (Presented in Russian). South African Council for Scientific and Industrial Research Translation No. 449.

Hoek, E. (1966). Rock fracture around mining excavations. Proc. 4th Intnl. Conf. on Strata Control and Rock Mechanics., New York.

20) Savin, G. N. (1961). Stress concentration around holes. Translated from Russian in Intnl. Series of monographs in aeronautics and astronautics, Pergamon Press, New York.
21) Hoek, E. (1963). Experimental study of rock stress problems in deep level mining. Experimental Mechanics, ed. Rossi, Pergamon Press, London, pp. 177-193.
22) Denkhaus, H. G. (1948). The application of the mathematical theory of elasticity to problems of stress in hard rock at great depth. Ass. Mine Mngrs. S. Africa, Vol. 1958/59, pp. 271-310.
23) Deist, F. H. (1965). A non-linear continuum approach to the problem of fracture zones and rockbursts. 5. Afr. Inst. Min. Metall., Vol. 65, No. 10, May, 1965.
24) Cook, N. G. W. (1965). A note on rockbursts considered as a problem of stability. 5. Afr. Inst. Min. Metall., Vol. 65, No. 10, May, 1965.
25) Cook, N. G. W. (1965). Failure of rock, Intnl. J. Rock Mech. and Min. Sci., Vol. 2, No. 4..
26) Fairhurst, C. and Cook, N. G. W. (1966). The phenomenon of rock splitting parallel to a surface under a compressive stress. Proc. 1st Intnl. Congress on Rock Mechanics, Lisbon.
27) Muller, L. (1963). Application of rock mechanics in the design of rock slopes. Proc. Intnl. Conf. on State of Stress in the Earth's crust, Santa Monica.. Ed. W. R. Judd, Elsevier publishers, New York, pp. 575-598.
28) Serafim, J. L. (1963). Rock mechanics considerations in the design of concrete dams. Proc. Intnl. Conf. State of Stress in the Earth's crust. Santa Monica. Ed. W. R. Judd, Elsevier publishers, New York, pp. 611-645.