Proceedings of the 7th CANADIAN ROCK MECHANICS SYMPOSIUM

EDMONTON, MARCH 1971

Sponsored by:
The Canadian Advisory Committee on Rock Mechanics,
and the University of Alberta.

PUBLISHED BY
MINES BRANCH
DEPARTMENT OF ENERGY, MINES AND RESOURCES, OTTAWA, 1972

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Foreword

Problems involving rock mechanics have attracted the interest of scientists and engineers with diverse backgrounds, and previous rock mechanics symposia in Canada have involved geologists, physicists, mining and civil engineers and others. Meanwhile other meetings and conferences have been held on related subjects, such as structural geology, foundations, pit designs, etc., and these have been attended by people with similar or complementary interests. After it appeared possible that a meeting in Edmonton in early 1971 of 4th Canadian Symposium on Research in Tectonics would be devoted to 'Applications of Structural Geology in Rock Mechanics' it was then proposed to broaden the format of this meeting by combining with it the next (7th) Canadian Symposium on Rock Mechanics, the Conference to be under the joint sponsorship of the National Advisory Committee on Geological Research and the Canadian Advisory Committee on Rock Mechanics.

Accordingly a committee was organized from these groups and a program developed to combine the informal format of previous tectonics research meetings, where free discussion was encouraged without concern for recording and editing, with the more formal presentation of papers that is traditional at rock mechanics symposia. The organizing committee therefore decided that the first three sessions should take the form of panel discussions around the central themes of 'In Situ Stresses', 'Role of Effective Stresses in Earth Behaviour', and 'Structural Mapping and Prediction of Engineering Properties of Earth Masses', with each theme to be surveyed first by an authority in the subject, followed by contributions from other experts in that subject and then with provision for free discussion from the floor. In order to allow for spontaneous comment it was agreed that presentations at these sessions would not be received as formal papers. However it was also agreed that short summaries of the themes and of consequent ideas and comments of particular interest would be prepared for the record.

The fourth and fifth sessions were planned for the presentation of formal papers on rock mechanics, with the fourth being devoted to papers on case histories in bedded deposits and the fifth dealing with studies in other deposits and on aspects of mine design. The papers of these two sessions (with the exception of one lengthy paper which is included only in abstract) are presented in this volume, following reviews of the themes of the first three sessions. Together the reviews and papers constitute the Proceedings of the 7th Canadian Symposium on Rock Mechanics. The detailed program with names of contributors is given in the following section.

The organizing committee wishes to express its thanks to all who contributed by presenting papers, reviews, comments and discussion. The exchange of information and ideas that developed during the course of the conference was stimulating and gratifying.

Generous financial support from the University of Alberta and the National Advisory Committee on Geological Research made the Conference possible, and the Mines Branch has undertaken the publishing of this volume of Proceedings. The assistance of these organizations is acknowledged with thanks.

T. Patching - for the Committee

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Program of Conference

"APPLICATIONS OF STRUCTURAL GEOLOGY TO ROCK MECHANICS PROBLEMS"
(4th Canadian Symposium on Research in Tectonics, and
7th Canadian Symposium on Rock Mechanics)

The University of Alberta, March 25-27, 1971

Thurday, March 25, 1971

Welcoming Address

H.A.K. Charlesworth, Dept. of Geology, University of Alberta, Edmonton.

Theme 1 "In Situ Stresses"

Chairman: R.A. Price, Dept. of Geology, Queen's University, Kingston.
Survey of Theme: H.U. Bielenstein, Mining Research Centre, Calgary.
Contributors: R.P. Benson, H.G. Acres Ltd., Niagara Falls; G. Herget, Mining Research Centre, Elliot Lake, Ont.; D.J. Varnes and F.T. Lee, U.S. Geological Survey, Denver; A. Brown, Dept. of Geology, University of Georgia, Athens; R.E. Goodman, Dept. of Civil Engineering, University of California, Berkeley.

Theme 2 "Role of Effective Stresses in Earth Behaviour"

Chairman: D.F. Coates, Mining Research Centre, Mines Branch, Ottawa.
Survey of Theme: N. Morgenstern, Dept. of Civil Engineering, University of Alberta, Edmonton.
Contributors: W.F. Brace, Dept. of Earth and Planetary Sciences, Massachusetts Inst. of Technology, Cambridge; P.E. Gretener, Dept. of Geology and Geophysics, University of Calgary; C.B. Raleigh, National Center for Earth-quake, Research, Menlo Park; W. Wittke, Tech. University Karlsruhe; J. Sharp, Golder, Brawner Associates Ltd., Vancouver.

Friday, March 26, 1971

Theme 3 "Structural Mapping and Prediction of Engineering Properties in Rock Masses"

Chairman: R.M. Hardy, Dean, Faculty of Engineering, University of Alberta, Edmonton.
ırvey of Theme: J.S. Scott, Geological Survey of Canada, Ottawa.
ntributors: G.H. Eisbacher, Geological Survey of Canada, Vancouver R.D. Call, Dept. of Mining and Geological Engineering, University of Arizona, Tucson; F.D. Patton, Dept. of Civil Engineering, University of Illinois, Urbana; D.K. Murphy, H.G. Acres Ltd., Niagara Falls.

Theme 4 "Case Histories in Bedded Deposits"

Chairman: W.M. Gray, Mining Research Centre, Mines Branch, Ottawa.
S. Serata, Serata Geomechanics, Berkeley
"The Serata stress control method of stabilizing underground openings."
C.A. Baar, Saskatchewan Research Council, Saskatoon
"Creep measured in deep potash mines versus theoretical predictions."
R. Parsons, Continental Oil Co., Morgantown, W. Virginia
"A study of the causes of roof instability in the Pittsburg coal seam."
I. Weir-Jones, University of British Columbia, Vancouver
"The modification of operating dimensions within an existing gypsum-anhydrite mine."
Film: "Stressing the Point in Rock Mechanics" - S.A.C.S.I.R.

Saturday, March 27, 1971

Theme 5 "Studies in Other Rocks and Mine Design"

Chairman: T.H. Patching, Dept. of Mining and Metallurgy, University of Alberta, Edmonton.
S.S.M. Chan, Dept. of Mining Engineering and Metallurgy, University of Idaho, and
T.J. Crocker, U.S. Bureau of Mines
"A case study of in situ deformation in the Silver Summit Mine, Coeur d'Alene mining district."
W. Pariseau, Montana College of Mineral Science and Technology, Butte
"Influence of surface topography on the pre-mining state of stress."
B. Ladanyi and A. Roy, Ecole Polytechnique, Montreal
"Some aspects of bearing capacity of rock mass."
K. Wardell, Wardell and Partners, Newcastle
"Mine design - A systems approach."

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Organizing Committee for the Conference

H.A.K. Charlesworth, Dept. of Geology, University of Alberta, Edmonton
N.R. Morgenstern, Dept. of Civil Engineering, Univ. of Alberta, Edmonton
T.H. Patching, Dept. of Mining and Metallurgy, Univ. of Alberta, Edmonton
K. Barron, Mining Research Centre, Mines Branch, Calgary
H.U. Bielenstein, Mining Research Centre, Mines Branch, Calgary

In Situ Stresses

H.U. Bielenstein* and K. Barron**

INTRODUCTION

This paper presents a summary of the presentations made during the session on in situ stresses. The objective of this session was to discuss, with examples, some of the problems and approaches to the interpretation and significance of in situ stress determinations from both the engineering and geologic viewpoints. The session coordinators (BIELENSTEIN and BARRON) posed a series of questions aimed at identifying problem areas; individual contributors (HERGET, BENSON, VARNES, BROWN and GOODMAN) reported on specific studies aimed at trying to answer some of these questions and, in turn, raised a number of more detailed problems. This summary is not comprehensive but merely attempts to report some of the highlights of the session in a cohesive framework. Many of the questions posed remain unanswered, some ideas presented may be controversial, but nonetheless it is thought that this session achieved a frank discussion between engineers and geologists and some mutual appreciation of how each discipline might aid the other in the search for answers.

TERMINOLOGY

The terminology currently used to describe in situ stresses shows great diversity; to avoid lengthy argument on terminology the contributors were asked to follow a specific terminology, provided that it did not offend their sensibilities, or alternatively to indicate clearly where they deviated from it. This suggested terminology is illustrated in Figure 1 and defined below:

Induced stresses are man-made stress components due to removal or addition of material. They are superimposed on natural stresses which exist prior to excavation. The natural stress field can be composed of gravitational stresses (due to mass of overburden); tectonic stresses and residual stresses (a much used and abused term, taken to mean "stress components that remain in the structure if external forces and moments are removed"(1)). Tectonic stresses may be active tectonic stresses (due to active present day straining of the earth's crust) and/or remanent tectonic stresses (due to past tectonic events which have only been partially relieved by natural processes).

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OBJECTIVES OF IN SITU STRESS DETERMINATIONS

Natural stresses in rock are the cumulative product of events in its geologic history; as, for example, the strength or the mineralogy of a rock are also determined by geologic processes. The geologist is concerned with the understanding of tectonic events and processes that took place in geologic time. These must be deciphered from the evidence currently available. A knowledge of the natural stresses, in conjunction with the geometry of the geologic structures and the kinematics of their formation, play an important part in tectonic analysis and should aid in answering such questions as:

(i) Is the current state of stress tectonically active or passive?
(ii) What tectonic events in space and time have affected this rock?
(iii) Do remanent tectonic stresses affect the style of deformation during subsequent tectonic events?

The engineer is primarily interested in designing and excavating safe structures in rock. He therefore needs to know the natural stresses, active in the rock, together with a knowledge of the mechanical behaviour of the rock when subjected to natural and induced stresses. His design must also take into account the useful working life of the structure, which may extend from milliseconds in the case of blasting to, say, 100 years for dam foundations. (In geologic terms this time spectrum would be regarded as "instantaneous".)

With these objectives in mind the following questions were posed: What is actually measured using current techniques? ; how are these measurements interpreted and what assumptions are involved? ; what is the influence of geometric scale on the measurements and their interpretation?

INFORMATION OBTAINED FROM CURRENT MEASURING TECHNIQUES

The majority of current "stress measuring" techniques in fact sense or measure strain; these strains must be transformed, using some assumptions of material behaviour, to give "stresses". Most of the techniques measure the strain produced by cutting out or overcoring instrumented rock specimens. These strain relief measurements can be split into the categories of short term (within two hours of overcoring the specimen) or long term strain relief (relief occurring after two hours).

If the strain relief measurements are confined to the domain beyond the influence of induced stresses then both short and long term relief may in turn be divided into two categories: that attributable to the relief of residual stresses and that attributable to relief of the other natural stresses (tectonic and/or gravitational), as illustrated in Figure 2.

Short Term Strain Relief

Short term strain relief measurements obtained by overcoring techniques are interpreted in terms of stresses assuming that the rock behaves as an elastic material, and that the elastic constants are obtained from tests on the overcored specimens.

(a) Overcoring of Rock In Situ. If in situ rock is overcored and the stresses determined in a number of localities are consistent then these stresses may be interpreted as tectonic and/or gravitational. The tectonic component would have to be determined as the difference between the calculated gravitational component at any one locality and the determined stresses.

The engineer would regard these stresses as the "active stress" on his structure without differentiating the various components. The geologist is more interested in the tectonic component; particularly in its spatial relationship to the principal axes derived from structural analysis. Stress determinations themselves cannot be used to differentiate between active tectonic and remanent tectonic stresses. This type of interpretation must be based on analysis of the kinematics of deformation (2).

It is in this category of stress determination, short term in situ relief assuming elastic behaviour, that most work has been done as was well illustrated by the presentations of both HERGET and BENSON.

HERGET reported on a well documented study aimed at relating stress determinations to geologic structure. A tectonic analysis and stress determinations were carried out to obtain the pre-mining stress field at an iron mine in the Lake Superior region. The siderite orebody is part of the Archean rocks which have been folded about regional ENE trending fold axes. Slaty cleavage, slickensides and extension fractures indicated, for this phase of deformation, a compression in a horizontal NNW direction. This picture was modified due to the development of NW to N trending folds which preceded the development of prominent faults of similar strike. Finally the youngest recognizable deformation phase was indicated by shearing along late Precambrian diabase dykes, development of kink bands and quartz filled fractures suggesting a compression for the mine area in an E to NE direction. On the assumption that no haphazard changes occurred and that tectonic stresses existed in the area, HERGET speculated that the maximum stress should be associated with the youngest deformation phase and thus should be oriented horizontally NE to E.

Short term elastic strain recovery measurements were carried out at six sites at three elevations in the mine. In each case the calculated stress tensor was the value of the best fit data from 30-60 strain-recovery measurements in three or four holes. The results showed clearly that there was no uniform direction for the maximum principal compressive stress. At three sites the measured directions were in agreement with those predicted from the kinematic analysis of the most recent deformation phase (NE to E). However, at another two sites, the measured directions were very close to the maximum principal compressive stress direction of the, earlier, major deformation phase (i.e. NNW). HERGET thus concluded that, in this case, it is not possible to predict the principal stress directions from the tectonic fabric in these basement rocks. The stresses must be determined; although the sites for stress determinations should be selected on the basis of tectonic analysis. Finally he concluded that, since there is no prevailing principal stress direction, the area investigated was truly an area of remanent tectonic stress. For a current build up of tectonic stresses he would have anticipated a more uniform stress field.

This study by HERGET clearly illustrates some of the difficulties of interpretation that can occur despite carefully taken and well documented measurements. His conclusion that the stress directions were probably attributable to remanent rather than active tectonic stresses is interesting, although it might well be the subject of some debate.

BENSON reported what could be regarded as a typical engineering application of in situ stress determinations aimed at assisting the design and layout of the underground chambers for the power house at Churchill Falls. Initially finite-element stress analysis studies were carried out for model chambers of various shapes assuming boundary conditions given by a vertical overburden stress, $\sigma_v = \gamma_z$, with a horizontal stress, σ_h, ranging from 0.3 σ_v to 2.0 σ_v. For each ratio of the boundary conditions, $K = \sigma_h/\sigma_v$, the chamber shape was changed to minimize the tensile zones around the openings. On completing this analysis a knowledge of the in situ stress conditions was required in order to select the optimum design for the actual boundary conditions of the site. (Note: in this case, the determination of the ratio K is more important than the actual magnitudes of the individual principal stresses.) Stress determinations were carried out at four locations with three to five holes at each location. Each hole was 20-25 ft long and fifteen tests were made in each hole using three axis deformation meters. It was estimated that 80% of the data for any given hole were within 15% of a median value, and this was considered to be confirmation of a general and predictable natural stress field in the rock mass. No preferred direction of lateral stress components was found. The results showed vertical gravity loading and an average K = 1.7 (ranging from 1.1 to 1.9). On the basis of these results the design was checked, and alterations were made as necessary to provide compliance with the results of the finite element model.

BENSON raised two very important points pertaining to this data analysis. Firstly he emphasized that measurements of this nature yield an excess of data over the minimum required for calculating the stress tensor. Consequently it is essential to average this data by means of suitable computer programs yielding best fit solutions to all the gathered data. The second major point concerned the value and determination of the Young's modulus of the rock. Usually the uniaxial modulus from laboratory tests is used in the stress calculations. In this case this yielded suspiciously high values for the stress components. Since this overcoring technique is essentially a biaxial measurement method they devised and carried out a test for Young's modulus determination under biaxial loading. This yielding a significantly lower value of Young's modulus ($E_{biaxial}/E_{uniaxial} \simeq 4x10^6/8x10^6$). This, obviously, significantly changes the stress magnitudes calculated (although not affecting the ratio or direction).

The importance of this latter point raised by BENSON cannot be overemphasized. In attempting to interpret stress magnitudes, rather than directions, total reliance is placed on the value of the modulus used in the calculations. Considerable effort must therefore be made in checking modulus values to ensure that correct values are used.

(b) Overcoring of Relieved Specimens. When an overcored specimen is itself overcored any resulting short term strain relief is attributed to relief, or more probably, partial relief of residual stresses. The degree of partial relief experienced must be dependent on the scale of the specimen compared to the scale on which the stresses are locked in. Whilst it is possible to interpret this strain relief in terms of stresses, assuming elastic properties, it is difficult to know what significance such magnitudes would have.

VARNES and LEE expressed the concept of residual stresses well in their presentation. In summary this concept may be expressed as follows: Most rocks have at some time in their past been under higher pressure than they are today. Any tendency of rock to relax under lessened or removed load is restrained by the interlocking fabric of anisotropic mineral grains, cement between grains, or by shear stresses along fractures. Consequently, equilibrium of rather large internal forces may be attained within completely free rock bodies.

From this concept VARNES and LEE stated the hypothesis that if a new free surface is created within a body that contains a system of balanced forces in static or dynamic equilibrium, the geometry of the body will change and the direction and amounts of the balanced forces will be altered. The effects may be instantaneous (short term) or time dependent (long term), or both. The free surface may be created by the slow process of natural erosion or by the rapid processes of tunnelling or drilling. The process of readjustment starts at the free surface and works inward. The initial rate of adjustment is more rapid near the free surface and depends upon the composition, fabric, and the structure of the rock and upon exterior loads, if any.

Some of the consequences of this hypothesis have considerable, practical and theoretical importance, amongst them are:

(1) Forces balanced on a microscopic scale can perhaps be mobilized to act on volumes of significance to engineering structures, and to add their effects to any active exterior tectonic loads. Real unbalanced stresses may be mobilized which are much larger than accounted for by overburden; they may amount to an appreciable fraction of those under which the rock consolidated or was last annealed.

(2) If the rate of relaxation varies with distance from a newly created face then there will be a size effect. From this it may be inferred that a rock of a certain size or shape having particular physical properties can contain a balanced system of forces of only a certain maximum intensity without application of exterior loads, and that corresponding to a particular level and distribution of internal stresses there is a volume or shape of "locking domain" in the rock that can contain this system of equilibrium. If the hypothesis is true then not only is there an equilibrium volume or "locking domain" for residual stress but this volume can be altered, generally enlarged, by the creation of a new surface.

These ideas proposed by VARNES and LEE have numerous implications. One very important one concerns the interpretation of short term, in situ strain-relief measurements of the type (a) described above. The engineer generally ignores residual stresses since, if they remain locked in the rock on the scale of the excavation, they cannot be regarded as stresses acting on the structure. In this case the effect of residual stresses will be apparent only insofar as they must affect the strength of the rock and its isotropy. Provided that the engineer can obtain a satisfactory measure of strength and its directional variation he may safely ignore residual stresses on the scale of these measurements. However, as VARNES and LEE pointed out, if the stresses are locked in the rock on a scale larger than that of the excavation, then excavation and in situ overcoring will relieve or partially relieve the residual stresses. In which case the result of sampling could be to infer, incorrectly, that the exterior boundaries of the body are under active tectonic load. It would also follow in such a case that stress relief techniques may be dependent on the shape and size of the instrumented body of rock that is freed from its surroundings.

Whilst the engineer may or may not be justified in ignoring residual stresses the geologist certainly cannot afford to do so. Residual stresses must reflect some aspect of the geologic history of the rocks. The magnitudes may be unreliable, but the principal stress axes obtained from measurements should again be compared with those derived from kinematic analyses. Unfortunately to date there is insufficient data to establish the usefulness of this approach; further research work is obviously warranted.

Long Term Strain Relief

The long term strain relief measurements are measures of time dependent strains experienced by overcored specimens. [Care must, of course, be taken to ensure that strains caused by temperature changes or by drying out of the specimen are eliminated or corrected for in the test procedures.]

Assuming that a true measure of time dependent relief is obtained this can be attributed to two possible sources.

(c) Overcoring of Rock In Situ. Time dependent relief of tectonic and/or gravitational stresses; i.e. this could be regarded as the viscous relaxation of engineering active stresses.

(d) Overcoring of Relieved Specimens. Time dependent relief, or partial relief, of residual stresses through similar viscous mechanisms. It is immediately apparent that, currently, a distinction cannot be made between these two sources of time dependent strain relief.

Relaxation of type (c) has engineering significance since it involves the active deformation of the structure. Thus, although the engineer currently ignores these components, it would be desirable to separate the two components. Perhaps the results of type (c) relaxation might then be interpreted in terms of stresses assuming some rheologic model of material behaviour. Whether or not the engineer could then safely ignore the effects of residual stresses would then depend, as before, on the scale of measurement as compared to the "locking domain". If the overcoring size were greater than the equilibrium volume the effects of residual stresses could be ignored since in this case they would be reflected as time dependent strength variations.

A distinction between the two sources of time dependent relaxation could be important to the geologist. No interpretation in terms of stresses would be attempted but the geologist would seek to compare his kinematic analysis with the axes of principal strain relaxation. Little work, even without distinction between the causes, has been done in this area. For this reason the presentation by BROWN was particularly interesting.

BROWN reported on both short term and long term strain relief measurements obtained, using photoelastic gauges, from in situ overcoring in a number of rock outcrops at several locations from Texas to Wyoming. The results were divided into initial (short term) and final (stable condition reached after several days) measurements. The initial and final principal strain axes were not coincident, but each could be related to distinct geologic features or stress environments.

Measurements in the Llano uplift, Texas, were made on the periphery of a "mini-mesa". The initial major extension paralleled the strike of NE trending normal faults and was perpendicular to the regional fold axes. The final major extension was radial to the "mini-mesa" (SSE, at the point of measurement) and was interpreted as relief of radial strain; the tangential strain having been relieved by a penetrative set of radial fractures.

Near Cody, Wyoming the strain relief measurements were made in the crystalline core of a tilted block uplift and the sedimentary envelope forming a drape fold over this uplift. Initial major extension axes were compatible with current incumbent loads at the measuring sites: vertical within the crystalline core and parallel to bedding dip at the base of the nearly vertical limb in the drape fold. The final major strain axes were related to the formation of the uplift. Within the core, major extension was perpendicular to bedding overlying the uplifted block; but at the base of the nearly vertical limb major contraction paralleled the bedding dip reflecting the extension of this limb during uplift.

Measurements at the Rangely anticline in Colorado showed a consistent rotation of initial to final major extension axes, from N70°E to N85°E. BROWN related this rotation to the changing stress environment of the developing anticline.

Combination short and long term strain relief measurements of the type described by BROWN open up a potentially valuable research approach for the geologist. Short term strain relief approximates elastic behaviour whereas long term relief may be regarded as creep; the measurements thus represent distinctly different mechanical behaviour in the rock. If the orientation of major strain axes is different between the short and long term strain relief, then the long term strain must reflect an older stress environment, with a younger, purely elastic, component superimposed on it. Long term strain relief, when measureable, could be a significant aid in the structural interpretation of some areas.

THE INFLUENCE OF GEOMETRIC SCALE ON MEASUREMENTS AND THEIR INTERPRETATION

Some aspects of geometric scale have already entered the discussions given above; they are sufficiently important to warrant specific consideration. Firstly, let us define three geometric scales; the microscopic, the mesoscopic and the macroscopic. Following Turner and Weiss (3) they may be defined as follows: The <u>microscopic</u> scale is considered to cover bodies up to a size that may be conveniently examined under a microscope (i.e. for most rocks, containing a large but finite number of grains). Consider the <u>mesoscopic</u> scale to cover rock bodies from hand sample size up to exposures that may be directly observed as in normal engineering excavations. The <u>macroscopic</u> scale covers bodies and structures too large or too poorly exposed to be examined directly in their entirety.

Currently all strain relief measurements by overcoring fit into the mesoscopic scale, as do most engineering excavations. Much care must be exercised in extrapolating results from one scale to the next. Consider first some of these problems in relation to standard overcoring techniques which relieve tectonic and/or gravitational stresses.

Whilst measurements on a mesoscopic scale might be expected to average out variations occurring on a microscopic scale they will not do so for variations on the mesoscopic scale. Thus a measurement at any one site should be regarded purely as a "sample" stress determined at that site. To extrapolate these results to cover a much larger rock volume (even though still in the mesoscopic scale), as the engineer often wishes to do, it is necessary to sample at a number of sites and only if individual site variances are comparable with the total variance is the extrapolation justified. This situation is akin to the problems facing the mining engineer when he wishes to assess overall ore grades from drill holes: - how many samples does he need and how reliable are his estimates ? Sophisticated statistical techniques for assessing the adequacy of sampling procedures must therefore be adapted for use in these problems.

For the geologist wishing to extrapolate results in the mesoscopic scale to applications in the macroscopic scale sampling problems become even more difficult. Sampling procedure must ensure that measurements from representative lithologic units are obtained. This type of problem is not new to the structural geologists; they have been confronted by it in the analysis of mesoscopic fabric elements.

For residual stresses the following complexities, already partially discussed, are introduced when geometric scale is considered. Do residual stresses exist in the macroscopic scale ? If so, then by overcoring on a mesoscopic scale they will be relieved completely, or nearly completely, and would thus be interpreted as tectonic stresses. To the engineer this misinterpretation as to cause would not necessarily be important since they would be active stresses on his structure. To the geologist however there is a significant difference in meaning which would affect his interpretation of tectonic events. It might be argued that the existence of residual stresses on a macroscopic scale would seem unlikely since this would imply that extensive volumes of rock would be in tension. The little evidence available to date suggests that such extensive volumes of rock in tension do not exist; thus this argument may be valid.

Likewise there is reason to doubt that residual stresses exist in the mesoscopic scale since this would require that discontinuities on the mesoscopic scale (such as joints, etc.) would be required to transmit tensile stress across them.

It would seem probable therefore that residual stresses only exist on the microscopic scale. The volume of rock containing this balanced system of forces must surely be dependent on the continuity on the rock mass. However, when these forces are mobilized by excavation of a free surface, on what scale are the effects of mobilization observed? VARNES and LEE postulate that these effects might be observed on a scale large enough to be of engineering significance. The authors are inclined to think that forces mobilized from residual stresses at the microscopic scale would affect a volume of rock close to that scale. At present there is little evidence to support either viewpoint; obviously this is an area for future research needs particularly with regard to effects on engineering structures.

GENERAL GEOLOGIC CONSIDERATIONS IN RESPECT TO NATURAL STRESSES

As pointed out previously, both the natural stresses and the physical parameters of a rock mass can be regarded as the cumulative product of all geologic processes active in the formation of the rock mass. The influence of some geologic parameters on natural stresses was discussed by GOODMAN. He considered briefly the heterogeneity of natural stresses linked to "hard" and "soft" regions or formations, and suggested that higher stresses might well be expected in the stiffer layers. He also pointed out that faults divide a rock mass into mechanically distinct blocks and disrupt the continuity of the stress field. Finally, GOODMAN presented some interesting ideas on the possible influence of erosion on the stress state:

Assuming that the rock originates from a molten state at depth Z_o, it inherits an initial natural stress state that is lithostatic. Thus the stress state at depth Z_o can be represented as follows:

$$\sigma_{horizontal} = \sigma_{vertical} = \gamma Z_o$$

where γ is the rock density.

If the earth's surface approaches the granite through erosion ΔZ, the vertical and horizontal stresses are reduced by amounts proportional to the depth of erosion. Corresponding to removal of ΔZ feet of rock, assuming the rock to be isotropic and linearly elastic, the stress changes are as follows:

$$\Delta\,\sigma_{vertical} = -\ \gamma \Delta Z$$

$$\Delta\,\sigma_{horizontal} = \frac{\nu}{(1-\nu)}\,\Delta\,\sigma_{vertical} = -\,\frac{\nu}{(1-\nu)}\ \gamma \Delta Z$$

where ν is the Poisson's ratio of the rock.

The vertical stress reduction is such that the vertical stress tends to zero as the depth approaches zero. However, the horizontal stress is reduced at a slower rate and is therefore always finite and greater than the vertical stress.

The stresses at depth $Z - \Delta Z$ are:

$$\sigma_1 = \sigma_{horizontal} = \gamma Z_o - \frac{\nu}{(1-\nu)} \gamma \Delta Z$$

and

$$\sigma_2 = \sigma_{vertical} = \gamma (Z_o - \Delta Z)$$

As erosion increases, the stress difference becomes increasingly severe and the rock may fail. Any failure criterion expressible in terms of σ_1 and σ_2 can be transformed through these equations to have the depth (Z_o - ΔZ) as the independent variable. The resulting stress state ($\sigma_1 = \sigma_{horizontal}$; $\sigma_3 = \sigma_{vertical}$) is thus postulated to follow the loci shown in Figure 3). Comparison with published stress measurement results indicates such a theory is not an unreasonable explanation for measured stresses. Further it shows that most measurements of horizontal stress fall along the failure portion of the locus (curve 3, Figure 3), i.e. that the natural stress field is almost never measured as it lies too deep.

The above ideas presented by GOODMAN give a satisfying explanation of the existence of high lateral stresses close to the surface and this may be particularly relevent for intrusive rock. Indeed it is this theory that Cadman (4) invoked to account for sheeting in granitic rocks; to do this Cadman chose to use a maximum extension strain theory of failure rather than the more general theory $\sigma_{1f} = f(\sigma_3/\sigma_1)$ given by GOODMAN.

ACKNOWLEDGEMENTS

The following contributors made presentations of their ideas and work during this session: R.P. BENSON, H.G. Acres Ltd., Niagara Falls, Ont.; A. BROWN, Dept. of Geology, University of Georgia, Athens, U.S.A.; R.E. GOODMAN, Dept. of Civil Engineering, University of California, Berkeley, U.S.A.; G. HERGET, Mining Research Centre, Elliot Lake, Ont.; D.J. VARNES and F.T. LEE, U.S. Geological Survey, Denver, U.S.A.

R.A. Price, Dept. of Geological Sciences, Queen's University, Kingston, Ont. chaired the session. The authors, who acted as session coordinators, would like to express their appreciation to all the above for making this a lively conference session. In particular, all credit is due to the contributors who made their notes available for compilation of this summary. Authors of a summary of this kind must always hope that not too many errors, accidental or personal, have been made.

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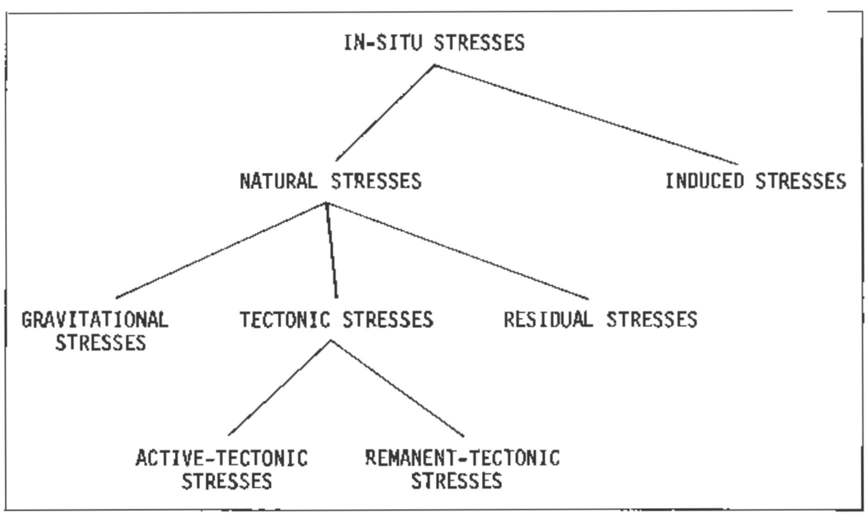

Figure 1

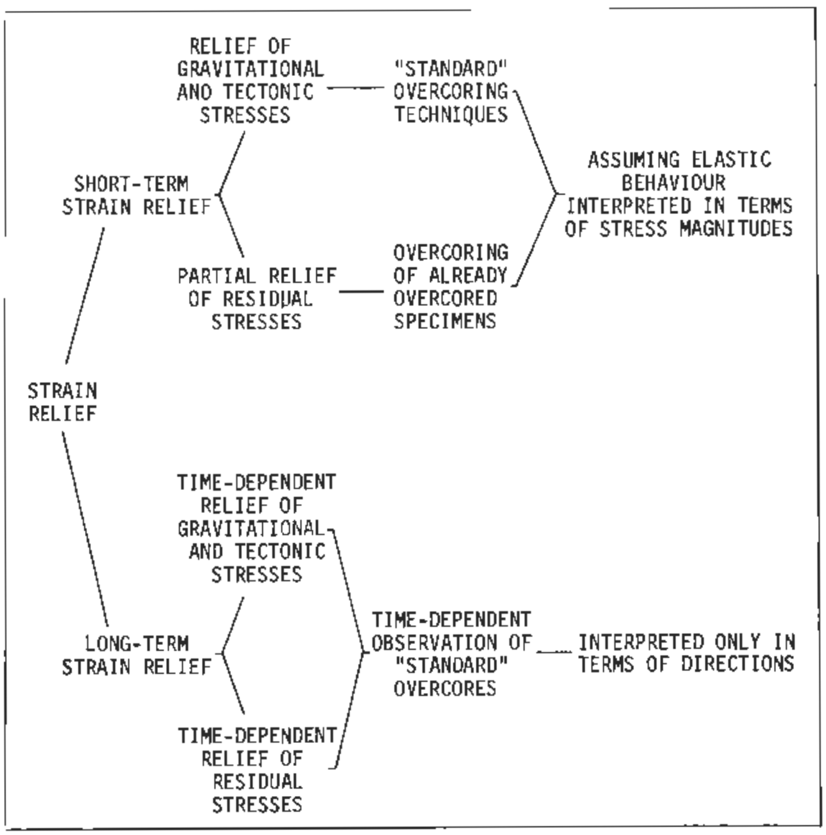
RELIEF OF GRAVITATIONAL AND TECTONIC STRESSES
"STANDARD" OVERCORING TECHNIQUES
SHORT-TERM STRAIN RELIEF
ASSUMING ELASTIC BEHAVIOUR INTERPRETED IN TERMS OF STRESS MAGNITUDES
OVERCORING OF ALREADY OVERCORED SPECIMENS
PARTIAL RELIEF OF RESIDUAL STRESSES
STRAIN RELIEF
TIME-DEPENDENT RELIEF OF GRAVITATIONAL AND TECTONIC STRESSES
TIME-DEPENDENT OBSERVATION OF "STANDARD" OVERCORES
LONG-TERM STRAIN RELIEF
INTERPRETED ONLY IN TERMS OF DIRECTIONS
TIME-DEPENDENT RELIEF OF RESIDUAL STRESSES

Figure 2

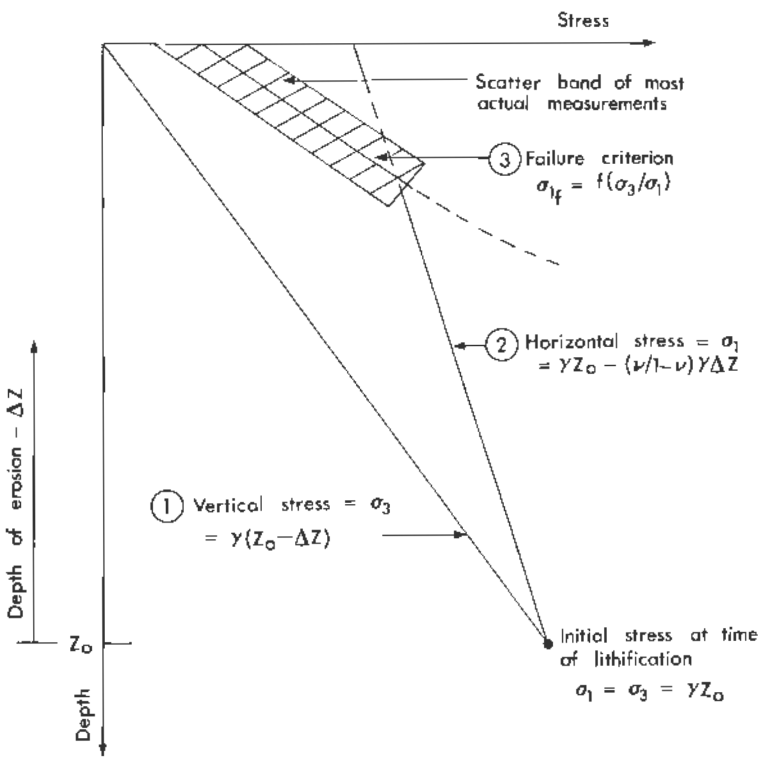

Figure 3. Loci of vertical and horizontal stresses as a function of erosion.

Role of Effective Stresses in Earth Behaviour

(A Summary of Discussion)

Norbert R. Morgenstern*

INTRODUCTION

The concept of effective stress if fundamental to an understanding of many diverse phenomena associated with the behaviour of earth materials. The Session Organizer drew attention to the role of the concept in explaining geological features such as low angle overthrusts, many sedimentary structures, and earthquakes. As is well known, the use of the concept of effective stress also provides a basis for rational design in rock engineering where the presence of water is recognized. High water pressures in rock masses reduce the available shearing resistance and therefore increase the possibility of such hazards as slides and collapse of underground excavations. Knowledge of laws governing the flow of water through rocks is essential if the pressure distribution is to be estimated with any accuracy or, in the case of excess flows, if the design of pumping systems is to be undertaken. The interaction of rock stress and water pressure is also of concern in reservoir engineering.

The aim of this Session was to bring together a panel representative of the broad spectrum of those concerned with exploring the range of validity of the concept of effective stress and applying it to both natural occurrences and engineering problems. The panel consisted of Professor W. Brace (MIT, Department of Geology and Geophysics), Professor P. Gretener (University of Calgary, Department of Geology), Dr. C.B. Raleigh (U.S. Geological Survey, Menlo Park, California), Professor W. Wittke (Technical University, Karlsruhe, Germany), Dr. J. Sharp (Golder Brawner Ltd., Consulting Engineers, Vancouver) and the Session Organizer (Professor N.R. Morgenstern, University of Alberta, Department of Civil Engineering).

ROCK PROPERTIES

Professor Brace discussed the dependence of rock properties on the effective stress. He reviewed data collected mainly at MIT that illustrated how both frictional resistance and wave velocities changed with changes in effective stress. Since friction is one mechanism for attenuation in porous media, it follows that attenuation also depends upon the effective stress. Experimental evidence was presented to illustrate that the sliding characteristics of rock interfaces also vary

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ACKNOWLEDGEMENTS

The Session Organizer is grateful to the members of the panel for bringing together a comprehensive view of the role of the effective stress concept in understanding rock behaviour. He apologizes for any error and omissions in this brief summary.

Structural Mapping and the Prediction of Engineering Properties in Rock Masses

J. S. Scott*

INTRODUCTION

The objective of the session on the relation of structural mapping to the prediction of the engineering properties in rock masses was to examine those aspects of structural geology of importance to engineering in rock masses and, through examples of recent work, to illustrate both problems and solutions relevant to the theme.

Presentations and discussions during the two preceding themes, "In Situ Stresses" and Role of Effective Stresses in Earth Behaviour", had clearly identified the role of structural geology in rock engineering. Thus an ideal background was provided for the more detailed examination of the problems and contributions of structural geology.

A brief presentaion of the scope of structural geology in terms of its three principal elements - recognition, representation and genetic interpretation - by the session coordinator (Scott) served to identify some of the problem areas. These problem areas, contained primarily within the recognition and representation aspects of structural geology, were further highlighted by the panelists (EISBACHER, CALL, PATTON and MURPHY) in their presentation of structural geological approaches to engineering problems.

Both the presentations and discussions were principally centred about questions of scale, structural features, methods of field measurement, analysis and application of structural geological data to specific engineering activities. These topics, therefore, are used as a framework for the summary of the session. However, without the aid of the many excellent illustrations used by the panelists, it is inevitable that some information of significance will be omitted.

SCALE OF STRUCTURAL GEOLOGICAL FEATURES

The scope of structural geology ranging from individual crystal structures to composite rock masses such as mountain ranges spans about 15 orders of linear dimension as pointed out by Friedman (1964). In a general way, therefore, the number of fabric elements for a given volume of the earth's crust increases exponentially with a decrease in the linear dimension of a specific fabric element as shown in Figure 1.

Scale of mapping of structural features is of importance if major individual structural elements are to portrayed separately as opposed to their being included as a part of a statistical repre-

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sentation. All the panelists stressed the importance of mapping individually the major through-going structural features that contribute to weakness in rock mass behaviour. In order to map such features individually consideration must be given to both the magnitude of the feature and the scale of representation.

STRUCTURAL FEATURES

From the standpoint of structural mapping, the principal features of concern are those within the visual rather than microscopic range of identification and therefore pertain to individual or composite rock masses. EISBACHER identified joints, through-going discontiuuities (master joints, slickensided faults), pervasive fractures or bedding, shatter zones, fault breccia and gouge along bedding or faults as the major structural features to be mapped. Identification of the structural features alone, however, is not sufficient.

CALL and PATTON drew attention to the need to recognize such attributes of structural features as their continuity, planarity, roughness as well as the relationship between shear stress direction and the orientation of discontinuities.

Although determination of the spatial geometry and physical characteristics of discoutinuities is the primary objective of structural mapping to assist in the evaluation of shear resistance of rock masses, the significance of the structural features to the localization of groundwater flow and to zonation of weak materials such as fault gouge must also be recognized.

METHODS OF FIELD MEASUREMENT

Field techniques for structural ampping are dependent upon the type of exposure, i.e., outcrop surface, pit footwall or drill core, and whether the approach to mapping is statistical or deterministic.

The basic measurement technique for structural mapping uses the Brunton Compass, which, CALL pointed out, can result in considerable variation in readings of strike resulting from operator error and local magnetic deviations. Experience can assist in reducing compass errors which fall into a normal error distribution pattern if enough readings are made. In drawing upon structural mapping experience in large scale open pits in banded iron deposits in Mauritania, West Africa, CALL reported on the use of "fracture set mapping". This technique involved stretching a tape along a segment of pit footwall and measuring the altitude and other aspects of joints as they occurred along the segment. Approximately 60 to 80 readings were found to produce an optimum joint sample size.

PATTON illustrated the value of the statistical approach to structural mapping for civil engineering applications but cautioned that a single through going discontinuity, while statistically insignificant in itself, can be highly significant to the behaviour of a rock mass.

MURPHY, in describing the structural geology evaluation of the site for the underground powerhouse at Churchill Falls, highlighted the relationship between surface structural mapping and structural evalustion of oriented cores. Core from 20,000 feet of drilling was examined in detail for joint frequency, orientation, roughness and planarity. Where possible surface structural data were projected to depth to assist in core orientation evaluation. Core analyses were supplemented by NX borehole photography which proved successful in water-filled holes to depths of 1,000 - 1,500 feet and in winter temperatures of -40°F.

It was evident from the panelists' presentations and from the subsequent discussion that no single method of structural mapping is universally applicable. However, the value of integrated surface and borehole techniques utilizing both statistical and deterministic approaches as warranted by local conditions were clearly demonstrated by the panelists.

ANALYSIS AND APPLICATION OF STRUCTURAL GEOLOGICAL DATA

Data from structural mapping surveys may be plotted graphically for direct visual analysis or provide basic input for computational analysis using the finite element or other computer-based techniques. Detailed descriptions and comparisons of methods of data plotting and techniques for utilizing structural geological data in computer programs were not included in the presentations and discussion.

CALL demonstrated the applicabiliby of statistical structural geological data to large scale open pits with particular reference to the selection of a pit slope angle in relation to the dip of planarity in an attempt to minimize the daylighting of discontinuities. He further pointed out the necessity of interpreting the structural geological data with reference to the mode of failure that might be expected such as a circular arc, sliding wedge or ravelling type.

The principal contribution of structural geological surveys at Churchill Falls (Murphy) was the location of an optimum block for siting the underground powerhouse. The presence of a major fault in the original location necessitated relocation of the site. As the excavation progressed underground mapping confirmed the absence of major discontinuities that had been predicted from the analysis of structural data obtained during the integrated structural mapping, core drilling and borehole camera programs. The structural geological data were further applied to a review of the orientation of engineering structures, evaluation of the design of various underground openings and for the determination of the spacing and length of rock bolts.

ACKNOWLEDGEMENTS

The contributions of R.D. Call, Dept. of Mining and Geological Engineering, University of Arizona, G.H. Eisbacher, Geological Survey of Canada, Vancouver, D.K. Murphy, H.G. Acres Ltd., Niagara Falls, and F.D. Patton, Department of Geology, University of Illinois toward a highly iuformative session, both through their pre-

sentations and the discussion which followed, is greatly appreciated. Chairmanship of the session was ably provided by Dean R.M. Hardy to whom the thanks of the conference organizers and session coordinator are gratefully extended.

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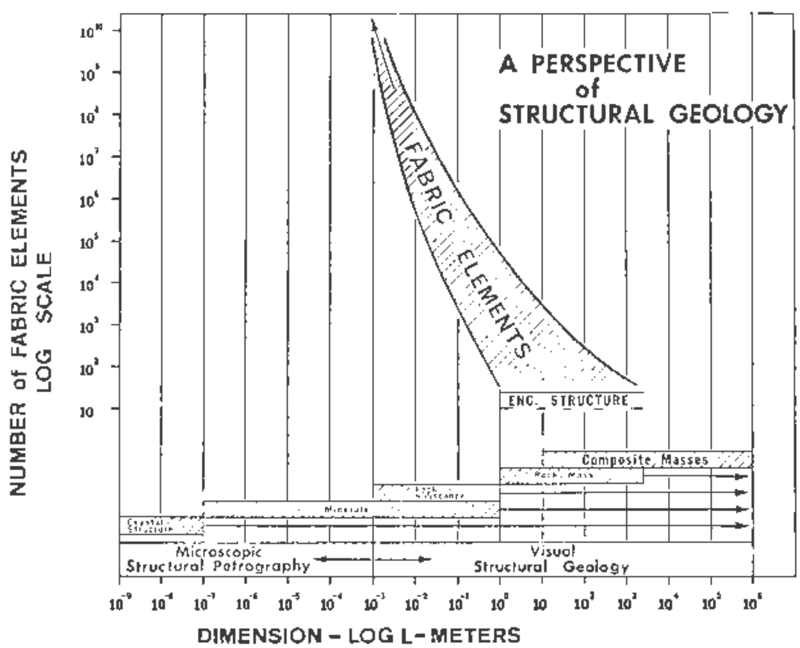

Figure 1

Creep Measured in Deep Potash Mines vs. Theoretical Predictions

C.A. Baar*

ABSTRACT

In recent years, many rock mechanics engineers, when facing actual underground conditions, recognized that laboratory testing is limited in its practical application to mining. This holds true in particular in salt and potash mines for two main reasons:

1. Strain hardening under usual testing conditions inevitably alters the creep
2. Rapid laboratory-time testing disregards the time dependence of creep under mining conditions.

Results of measurements in a deep potash mine (IMC) are presented to demonstrate

1) constant creep rates under constant loads, even in single openings in virgin ground,
2) the extent of creep around openings under various mining conditions,
3) the interrelationship between horizontal and vertical opening closure iu flat potash beds,
4) the interrelationship between creep rates and development of mined-out areas.

The measurement results clearly demonstrate that some theories developed from laboratory testing are not applicable to mining conditions. Applications of such theories in deep potash mining have proven dangerous with regard to rock bursts and flooding. Some basic principles to be applied in deep potash mine design are outlined.

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INTRODUCTION

Mining potash deposits at depths below 3000 feet requires the understanding of the behaviour of salt rocks under conditions which cause inelastic deformations (creep). In Germany, where potash mining began over a century ago, lack of knowledge of the physical properties of salt rocks has caused the total loss of more than one hundred potash mines by water inflow. In 1958, the largest rock burst ever experienced in world mining was caused by potash mining. Gases released by excess deformations in potash mines have claimed the lives of hundreds of workers.

Several disasters in potash mines during the early 1950's initiated extensive investigations into the creep in deep potash mines, in particular into the supporting capacity of mine pillars which are left in mined-out areas for overburden subsidence control.

However, the extensive laboratory testing of salt and potash samples has resulted in contradictory theories. Moreover, there is a tremendous gap between theoretical predictions and in situ measurements of creep in deep potash mines. During the past 15 years, the controversial aspects became obvious at International Rock Mechanics Conferences, e.g., in papers presented by Serata and Baar (Second Symposium on Salt, Cleveland, Ohio, 1965).

One purpose of this paper is to outline basic principles in deep potash mine design based on underground measurements, in particular to demonstrate for what reasons stress relief systems are capable of maintaining safe conditions in long-life underground openings in deep potash mines. First of all, however, some of the reasons which cause the obvious discrepancies between laboratory results and in situ measurements may be outlined.

WHAT CAUSES THE DISCREPANCIES BETWEEN CREEP UNDERGROUND, AND IN THE LABORATORY?

Low Limits of Elastic Behaviour of Salt and Potash In Situ

The scope of this paper does not allow a complete review of the creep properties of salt rocks under various conditions.

Reference is made to recent reviews by Odé (1968) and Shlichta (1968), who found that "there is reason to believe that slow creep might occur more extensively and with much less work-hardening than is observed in rapid laboratory-time deformation. Extensive steady-state creep might occur by simultaneous deformation and recrystallization."

Extensive steady-state creep indeed occurs in deep potash mines, as will be shown by numerous measuring results. Strain-hardening caused by rapid sample loading in the laboratory makes the determination of the true limits of elastic behaviour of salt rocks difficult, if not impossible. As a result, a wide variety of values has been published, and is claimed to represent these elastic limits ranging from less than 100 psi to several thousand psi.

For many decades already, the true elastic limits of virgin salt rocks have been known to be of the order of 100 psi, as emphasized by Baar (1961) with reference to a careful evaluation of the literature to 1936 prepared by Spackeler and Sieben (1944). Odé correctly stressed: "_If_ rock salt has a yield limit, this limit must be low."

Unfortunately, some writers (Hoefer, 1964) continue to postulate elastic limits of salt rocks far above 1000 psi which were determined in rapid laboratory tests. Serata's first determinations (1960–1964) of the octahedral shearing strengths (the limit of elastic behaviour under triaxial stress conditions) of common rock salt also yielded values of about 1500 psi. Changes in testing procedures lowered that value to 650 ± 50 psi in 1970. Obviously, different degrees of strain-hardening are responsible for such differences in the value of one of the basic parameters in mine pillar design.

Regarding the true limits of elastic behaviour of salt and potash, Serata (1968, p. 300) made a statement, the validity of which should never be questioned: "It is dangerous to use the commonly reported values in the design of potash mines as they are usually 2 – 10 times greater than the true value."

This danger has been demonstrated dramatically by disasters in German potash mines (Baar, 1966).

At this time, underground measurement results will be presented to demonstrate once more the consequences of using too high values of the elastic limits of potash in mine design. According to Coolbaugh (1967, Figure 3, which is identical with Serata's figures (1966, Figure 6, or 1970, Figure 3B) the value of 1500 psi was used in the original design of IMC's potash mine near Esterhazy, Saskatchewan.

Since most of the Canadian potash consists of NaCl and KCl crystals which possess nearly identical physical properties, the creep behaviour of potash (sylvinite) is resembled by that of common rock salt.

According to the above quotation, it would have been dangerous to use the value of 1500 psi if the 1970 value of 650 psi were the true value of the octahedral shearing strength of potash.

However, the true limits of elastic behaviour of salt and potash in situ are of the order of 100 psi, which means that the IMC design value was 15 times greater than the true value. Some consequences will be shown.

No Strain-Hardening Under Mining Conditions

The strain-hardening observed in laboratory tests has lead some investigators to a belief in "maximum possible strengthening" and "complete deformation" of mine pillars, which means that "no further deformation occurs" after an "end value" is reached. Strain-time curves (cumulative creep curves) are believed to develop into parallels to the time axis (Borchert and Mnir, 1964, pp. 277-279. Dreyer (1967), pp. 100-102).

Serata's (1968, p. 305) belief is similar: "The total cumulative closure of a mine opening approaches the ultimate value beyond which no further closure is possible", that is, the opening reaches a "stable condition".

However, the in situ creep of salt and potash is not affected by strain-hardening, as was demonstrated by in situ tests using sealed borehole sections to create differential stresses. Published records (Baar, 1959A, 1966) demonstrate continuing plastic reaction of the surrounding salt rock to any stress differences which were repeatedly created intentionally to determine the effects of strain-hardening. There was no indication of strain-hardening detectable.

Figure 1 shows the results of a test under triaxial stress conditions performed by Dreyer and Borchert (1955). In previous discussions regarding the true elastic limits of salt rocks and the missing of strain-hardening in situ, Baar (1961) referred to this figure as proof that only very little strain-hardeniug must be expected in situ if stress conditions are altered by mining.

A recent duplicatiou and re-evaluation of the same test (Dreyer, 1967, Figure 24) confirmed that the octahedral shearing strength remained less than 100 psi, no matter whether the axial load was increased or decreased, and finally removed completely.

In situ, the reaction of salt rock to removal of stresses in one direction by excavating an opening in highly stressed salt rock is similar to what Figure 1 demonstrates. The local stress differences are determined by the true elastic limits of about 100 psi, or even less.

Constant Creep Rates Under Constant Loads In Situ

"Classical" laboratory strain-time creep curves of salt or potash samples under constant uniaxial loads exhibit three stages;

1. primary or transient creep during which the creep rates decrease,
2. secondary or steady-state creep (also called pseudo-viscous or quasi-viscous creep) characterized by constant creep rates,
3. tertiary creep with increasing creep rates leading to failure of the test sample by rupture.

Due to strain-hardening, most of the numerous published strain-time curves found in laboratory tests of salt rocks resemble classical creep curves. If failure is not achieved under the applied loads, strain-hardening causes the creep rates to decrease, until no further creep occurs.

To the knowledge of this writer, only Obert (1964) found that "the creep rate in salt and potash model pillars becomes constant after a relatively short transient period".

Since no straiu-hardening occurs with in situ creep, continuously decreasing creep rates can not be expected under constant loads in situ. Numerous in situ measurements will be presented to prove this conclusion.

Does Steady-State Creep Indicate Impending Failure?

From "classical" laboratory creep curves, some writers postulate that steady-state creep inevitably leads to failure of mine pillars (Hoefer, 1964, McClain, 1967). The most recent application of that erroneous conslusion is seen in Figure 2 (Uhlenbecker, 1971), in which A shows cumulative pillar creep curves for loads $P_3 > P_2 > P_1$. Figure 2B is supposed to show the corresponding creep rates. However, it shows initially increasing creep rates in contrast to the decreasing creep rates in Figure 2a. The initial part of graph B appears to be derived from tests on samples which required re-consolidation of loosened grains. In Figure 2a, two possibilities are indicated for the further development of curve P_2. The creep rate either increases and leads to failure, or it decreases further to zero due to strain-hardening which is believed to make the pillars "stable". Steady-state creep is a priori excluded in these figures.

If statements such as these were correct, steady-state creep as demonstrated by numerous underground creep curves must cause considerable concern since it would indicate impending failure. Temporarily increasing creep rates would have to be regarded as indication of failure in the very near future.

It will be demonstrated that temporary increases in creep rates are caused by increases in loads on pillars due to overburden load re-distribution after mining.

As soon as constant loads are established, in situ creep rates become constant. Neither strain-hardening nor "weakening by structural damage" ie caused by creep in situ. Hence, "normal" creep under constant loads in situ is characterized by constant creep rates.

Stress Relief Causes the Initial Creep Into Underground Openings

The original stress field in all present-day mining areas in potash deposits is assumed to be hydrostatic, the stresses being determined by the dead weight of the overburden.

By excavation of an opening, the original stresses perpendicular to the opening surface are lowered to atmospheric pressure. The salt rocks around the new opening react to this change in stress conditions by creep into the opening.

Creep tends to re-establish local hydrostatic stress conditions. This is possible to the degree allowed by the limits of elasticity which are of the order of 100 psi for salt rocks.

While in the laboratory rock samples are usually loaded, to achieve creep, the initial creep around any new opening in a mine is caused by stress relief. This is a fundamental difference which is often overlooked. Unfortunately, many writers develop theories from rapid loading of samples which has no relationship to normal mining conditions.

In a mine, stress relief creep begins at the location where the stresses are removed in one direction, that is at the surface of any new opening. Creep immediately lowers the stresses in the two other principal directions, and results in partial relief of the stresses existing in the next layer of rock behind the surface. In a chain-reaction, rocks at greater distances are affected by stress relief creep into the opening.

Figures 3, 5, and 8 demonstrate that stress relief creep is a very uniform process all around any new opening. These figures also demonstrate that relief creep rates decrease with time until steady-state creep develops under constant loads.

Before dealing with the extent of stress relief creep around openings in deep potash mines, its significance in mine design should be outlined.

Principles of Mine Pillar Design

Theoretical principles of pillar design are often based on laboratory testing of elastic material, including photo-elastic experiments. Application of such theories to potash mine pillars postulates vertical stress peaks near the walls, and horizontal stress peaks near the roof and the floor of underground openings. Connecting these stress peaks results in "stress envelopes" or tangential stress peaks all around openings. Stable stress arches are postulated to exist in roof and floor of openings in deep potash mines.

Creep would be restricted to the rock inside such theoretical stress envelopes. Outside, the rock would remain in an elastic state and provide for "stable conditions". In order to achieve overall stability in a potash mine, mine pillars should be"designed so that there is not internal overlap of zones of plasticity within them " (Coolbaugh 1967, p. 73). This concept was applied in designing IMC's Canadian potash mine near Esterhazy, Saskatchewan.

This concept is not new, and it is suitable if applied in full knowledge of the horizontal extent of zones of plasticity around mine openings in salt and potash. The same concept has been applied for quite some time in German potash mines. However, due to theoretical assumptions which were essentially identical to those applied in the initial IMC mine design, the extent of creep was greatly underestimated, which caused some of the previously mentioned disasters in potash mines.

As a result of similar erroneous assumptions as to the extent of creep into pillars, the horizontal creep of the entire pillars in the first mining panels in the IMC mine causes continuous vertical room closure. The constant closure rates measured over several years allow the prediction that the first mined rooms will be completely closed within about 30 years after mining (see Figure 8).

Such closure rates may be acceptable in mined-out areas. However, main entries and other underground openings must be kept open at minimum dimensions for the life-time of a mine.

It appears to be obvious that the extent of creep zones around openings in deep potash formations and the rate of creep into openings are most important parameters in rational mine design. In addition, the possibility must be taken into account that a rock mass which eventually becomes a pillar may have been affected previously by creep into existing openings.

Unfortunately, laboratory research has failed in determining these parameters. For IMC's potash mines in Saskatchewan, these design parameters were established by re-evaluation of numerous underground measurements taken since the beginning of mining throughout the K 1 mine*. Inspections of all mined-out panels also have provided valuable information.

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* Permission to publish the figures on which this paper is based has been granted by the General Manager of IMCC(Canada) ltd., Esterhazy, Saskatchewan. This is gratefully acknowledged.

The application of these new parameters to entry and mining panel design will be outlined. It must be emphasized that the parameters only apply to the geological conditions under which they were determined. Under different geological conditions, in particular in bedded deposits with clay seams, application of the same parameters would be dangerous for reasons which will be outlined shortly.

MEASUREMENTS OF CREEP AND STRESS RE-DISTRIBUTION IN IMC'S POTASH MINES NEAR ESTERHAZY, SASKATCHEWAN

Dimensions of the Zone of Creep Around Single Openings

General Introduction

To this writer's knowledge, only a few attempts have been made to measure the exact dimensions of creep zones around openings in salt or potash in situ.

Knowledge of the extent of relief creep is in particular required for pillar design, since horizontal stress relief creep in pillars means further removal of overburden support, in addition to that already removed by excavation.

Vertical creep has no direct influence on overburden support. However, vertical stress relief creep above and below openings allows horizontal creep in corresponding areas above and below pillars. For this reason, the vertical dimensions of creep zones must also be known, in particular in mined-out areas in which overburden subsidence has not yet begun. Measurements in a German potash mine (Baar 1959A, 1966) demonstrate that the zone of stress relief creep around single openings in virgin ground at a depth of 2,700 feet extends much deeper into the surrounding salt rock than predicted from some theories. Two years after mining, the cross-section of the relief creep zone resembled an ellipse with a vertical axis of about 100 feet, and a horizontal axis of 80-85 feet.

Measurements in an isolated opening at IMC

Zahary (1965) showed the movement of points at distances up to 20 feet into an opening in virgin ground in the K 1 mine. Since no bedding planes exist in the potash bed and in the next 15 to 20 feet of salt rock in roof and floor, the rock around the opening may be classified as homogeneous.

In contrast to the predicted "self-stabilization" shown in Figure 3, the creep developed into steady-state creep as indicated by a dash line parallel to the time axis of the log-log graph. Curve 1 in Figure 8 shows the cumulative vertical creep measured over 8 years since excavation. The curve develops smoothly into a straight line which indicates the constant creep rate, and contradicts theoretical predictions as demonstrated in Figure 2A.

The dimensions of the zone of creep are listed in Table 1.

TABLE 1

Dimensions (ft.) of the Zone of Creep Around an Isolated Opening 21 ft. Wide by 7.5 High in Potash (Sylvinite) at 3,140 ft. Depth

Time since mining (days)	Vertical	Horizontal
450	94.5	73
1000	90.5	76
2000	88.5	81
2600	86.5	83

These dimensions are in good agreement with those in the foregoing sub-section .

With time, the vertical dimension of the zone of creep is reduced, while the horizontal dimension is increased. For this particular case, an almost circular cross-section was reached after 2,600 days. In order to learn more about the amount of initial vertical relief creep, and its time requirement, closure recorders were constructed which can be installed immediately after an opening is cut. Some recorded figures may demonstrate the rapidness of initial vertical stress relief creep.

Within the first few hours, the vertical relief creep into 14 feet wide openings 7.5-8 feet high amounts to 1 inch or more, the creep rates decreasing continuously.

Each step of widening an opening by cutting additional 7 ft. wide passes is reflected by an immediate increase in vertical creep into the original opening. The amount of immediate creep is less at each subsequent pass. Total initial vertical closures up to 3 inches were recorded in openings widened to 35 ft. After any increase in creep caused by widening, the creep rates decrease, the cumulative creep curves tending to develop into "normal" curves according to local conditions.

Near the faces and walls of openings, creep rates are smaller, indicating the restriction to creep caused by non-excavated ground.

These measurements demonstrate how rapidly the initial vertical stress relief creep extends into roof and floor.

The development of the horizontal creep into an opening differs greatly. Due to less opening height compared to width, and due to the circular cross-section of the opening at the walls, the initial horizontal creep is much more restricted. However, the rapid initial vertical creep partially removes the restrictions, lowering the horizontal stresses above and below the opening, and enabling horizontal creep above and below the opening walls into the stress relieved zone. Such horizontal creep, in turn, facilitates the horizontal creep in the walls.

After the initial rapid vertical relief creep, the further development is dominated by the tendency to establish more uniform stress conditions around the opening by horizontal creep. Horizontal creep is converted into vertical creep above and below the opening.

These interrelationships are indicated by the different courses of total closure curves. Vertical cumulative closure curves demonstrate the highest initial creep rates. After a few days, they flatten rather sharply, and the vertical creep rates become smaller than the horizontal ones. Horizontal cumulative creep curves, develop smoothly over a period of weeks into straight lines (see Baar 1970, Figure 2).

The interrelationships between horizontal and vertical creep are also indicated by the different times at which the creep rates become fairly constant at various locations around the opening. In Table 2, it can be seen that constant creep rates first develop at points along the horizontal axis of the zone of creep, beginning at 20 ft. and proceeding towards the opening. At points along the vertical axis, the creep rates develop correspondingly, but with a time delay of several weeks.

TABLE 2

Time (days) between mining and beginning of steady-state creep (constant creep rates) at various distances from a 21 ft. wide by 7.5 ft. high opening in potash at 3,140 ft. depth

Distance from opening (ft.)	Time (days) horizontally	vertically
20	90	150
10	100	175
5	150	200
.5	200	225

Measurements in Carnallitic Potash

In certain areas in IMC's potash mines, the sylvinite bed as well as roof and floor contain up to 10% carnallite ($KCl.MgCl_2.6H_2O$). This mineral is known to creep faster than halite and sylvite except for cases of sudden changes in stress conditions.

Carnallite as a component of salt rock facilitates creep, in particular if carnallite occurs as a bond between halite and sylvite crystals as is the case in the IMC mines.

The following measurement results were obtained in a 28 ft. wide room in carnallitic potash. Approximately one year after mining, the area received some additional load due to panel mining in the vicinity which resulted in increased creep. This may have influenced the development of the zone of creep around this opening.

The dimensions of the zone of creep at various times since mining are listed in Table 3.

TABLE 3

Dimensions (ft.) of the Zone of Creep Around a 28 ft. Wide Opening in Carnallitic Potash at 3140 ft. depth. Opening Height 7.5 feet

Time since mining (days)	Vertical (ft.)	Horizontal (ft.)
200	108	90
500	92	92
1000	86	94
1500	78	98
2000	62	102

Thirty ft. pins were installed at this measuring site. After 2000 days, the vertical creep was restricted to the rock between the 30 ft. points in roof and floor.

The exact amount of initial relief creep is unknown. During the first week of measurements, the vertical creep beyond 30 ft. was equal to the creep between 10 ft. and 30 ft., suggesting that the initial vertical extent of the relief creep zone may have been larger than listed at 200 days since mining.

During the same time period only very little horizontal creep beyond the 30 ft. points was measured.

The figures listed in Table 3 accentuate the points already emphasized in the evaluation of the creep around a 21 ft. wide opening.

Rapid vertical relief creep immediately after excavation extends much faster into roof and floor than initial horizontal creep extends into the walls. The initial cross-section of the zone of creep resembles an ellipse with a longer vertical axis.

In this particular case, the cross-section became a circle after approximately 500 days, and then developed into an ellipse with a longer horizontal axis. Horizontal creep reduced the vertical extent of creep to less than 30 ft. in roof and floor after 2000 days.

Development of Creep Zones Around Openings in Mining Panels

Interrelationships Between Creep and Overburden Subsidence

In theory, it might be possible to support the total weight of the overburden by stable pillars left in mined-out potash panels, the pillars being designed so that no horizontal creep occurs in the centres of pillars.

However, the large extent of the creep zones into pillars makes design principles aiming at stable pillars illusive unless the extraction ratios were reduced far below economic limits. For example, in a German carnallite mine at 2000 ft. depth, the extraction ratio was reduced to 15% after a rock burst had destroyed the mined-out panels in 1940. Even that drastically reduced extraction resulted in measurable overburden subsidence.

Therefore, another basic design principle in deep salt and potash mines must be to control the inevitable subsidence at any time by pillars or other means in such a way that no excess deformations occur, neither excess creep into mine openings which must be kept at minimum dimensions, nor excess deformations of roof and floor strata which could possibly result in gas or water discharge into the mine.

In order to apply these design principles in deep potash mining, it is imperative to know the exact dimensions of the zones of creep around openings at any time. Even more important is a knowledge of the relationship between creep and overburden load redistribution,

Here, the different time requirements for rapid initial relief creep, and for overburden subsidence come into the picture.

Surface subsidence measurements carried out over many decades above potash mines in Germany have shown that many similarities exist between overburden reactions to room and pillar mining of potash beds, and total extraction of other sedimentary beds such as coal seams.

The larger a mined-out area becomes, the more overburden formations become involved in subsidence, beginning in the immediate roof strata and procceding to the surface. As a rule of thumb, it can be said that the span of a mined-out area must exceed the depth below surface in order to allow unrestricted subsidence above the centre of the mined-out area.

Before this critical span is reached, the overburden weight is not fully supported by pillars in a mined-out potash bed.

The remainder of the overburden weight must be supported in an abutment zone around the mined-out area. The width of such abutment zones, and the amount of additional load are important parameters in potash mine design because of the additional relief creep which takes place immediately after excavating an opening in an abutment zone.

A mined-out area in which the pillars along an abutment zone are not yet fully loaded may be termed an "umbrella zone". The "umbrella" may be a temporary one, or it may remain effective over long periods of time.

Unfortunately, all these vital parameters - the span of a mined-out area at which overburden subsidence begins, the span needed for unrestricted subsidence, the time requirement for full subsidence, the width of abutment zones, the abutment loads at any time, the width of umbrella-zones, their life time, their effectiveness - have to be determined by measurements in the mines and at the surface. It is impossible to determine the behaviour of the various overburden formations by testing samples. This is mainly because of the frequent existence of cracks and fissure zones in most formations.

The values of some of the above parameters, as determined by measurements in the IMC mines, do not apply to other eonditiens. However, the measurement results provide a general picture of the interaction between creep into openings in potash and overburden load re-distribution after mining.

Horizontal Extent of Relief Creep in Pillars

Measuring and recording the horizontal extent of relief creep caused by excavation can be done conveniently by conventional instrumentation installed in a future pillar.

Measurements demonstrating the horizontal extent of relief creep around rooms 21 ft. wide by 7.5 ft. high were published by Zahary (1965, p. 8). The designed pillar width was 52 ft., and the belief was that there would be "no internal overlap of zones of plasticity" within the pillars (Coolbaugh 1967).

The measurements were taken in the first mining panel in the K 1 mine. Hence the original stress conditions resembled virgin ground conditions at a depth of 3140 ft. At measuring site 121-A, "ten days after mining the movement on the 20 ft. pin is slightly negative, that is, movement is toward the centre of the pillar".

The movement was toward the centre of the pillar. However, the centre of the pillar also moved toward a new opening excavated parallel to the one in which the measuring site was installed. One day after the measurements had begun in the existing opening, the parallel opening was excavated, the width of the pillar being 52 ft. Stress relief creep into this new opening caused the negative movement of the 20 ft. pin.

The anchor point of that pin at 32 ft. distance from the new opening moved slightly toward the new opening. The 10 ft. pin and the 5 ft. pin continued moving toward the older opening.

After one week, the negative movement of the 20 ft. pin reached its maximum value of .15 in. and remained unchanged during the following 50 days. Then panel retreat mining caused subsidence to begin, and to re-load the pillars, which in turn caused increasing horizontal pillar creep. The creep of the 20 ft. pin was no longer negative, the pillar centreline became the neutral line between movements in opposite directions toward both openings.

Similar measurement results were obtained at various measuring sites. Hence there can be no doubt as to the reality of "negative" movements in pillars which are not wide enough to cover the two zones of creep around two parallel openings.

Typical closure curves demonstrating the horizontal creep around two parallel openings excavated at different times are shown in Figure 4. Pillar and room dimensions were identical to those at site 121-A. The measuring site was installed in a room in the centre of the adjacent panel during panel entry development. (See Figure 7).

The development of stress relief creep, and the resulting vertical pillar stresses are schematically indicated in Figure 5, which is in excellent agreement with earlier measurements (Baar 1966, Figure 11).

Reduction of the Vertical Extent of the Relief Creep Zones by Horizontal Creep due to Pillar Re-Loading

In Figure 4, the creep rates begin to increase approximately 70 days after mining. Similar increasing creep has been measured at numerous measuring sites in the IMC mines. It begins as soon as a mined-out panel reaches a span of approximately 400 ft., regardless of the time elapsed since mining a particular opening. This means that under the geological conditions of that mine pure stress relief creep is restricted to mining areas which have a span less than 400 ft. If openings are excavated at the boundary of an area which has exceeded that critical span, the effects of relief creep and of re-loading creep are superimposed, resulting in more uniform cumulative creep curves which become straight lines as soon as no more changes in pillar loads occur (see Figure 8).

As pointed out before, horizontal creep due to pillar re-loading is the main factor iu reducing the initial vertical extent of the zone of creep. Since re-loading may begin immediately after the initial rapid stress relief creep, or at any later time, depending on the mining sequence around any particular measuring site, the reduction of the vertical dimension of the creep zone also depends largely on the load re-distribution around any measuring site.

Another factor of influence is the mineralogical composition of the involved salt rocks. Stress relief creep as well as re-loading creep take place at greater rates in carnallitic potash, reducing the time required for conversion of the initial vertical relief creep ellipse into an ellipse with longer horizontal axis.

Figure 6 shows vertical and horizontal cumulative creep curves measured in the first panel of a mining block in carnallitic potash in the IMC mines.

After 170-190 days, increasing horizontal creep was caused by pillar re-loading due to retreat mining of that panel. Constant creep rates were reached between 300 and 380 days, indicating constant loads on pillars. These loads, however, were not yet the final loads. The mined-out area was enlarged after 500 days to the dimensions required for full pillar loading.

The vertical creep development shows some remarkable differences. The 20 ft. pin did not move between 300 and 400 days, and showed "negative" movement between 400 and 500 days. This means that the stress relieved zone above the 20 ft. pin was re-compressed by .1 in. as a result of horizontal creep.

Some measured values of reduced vertical dimensions of zones of creep around openings in mined-out areas may be presented in Table 4 in order to indicate the order of magnitude of the reduction, and the time requirements for achieving such reductions. The initial vertical dimensions of the relief creep zones were over 100 ft.

Attempts to quantitatively relate any particular development of the shape of the zone of creep to the influencing factors would be merely speculative as long as the pillar loads at any time, and the mineralogical composition of the rocks involved in creep are not exactly known.

TABLE 4

Some Values of Vertical Dimensions of the Zones of Creep Around 7.5 ft. High Openings of Different Widths in Mined-out Areas at Different Times Since Mining. Values in Brackets are Extrapolated Values. Depth 3140 ft.

Room width (ft.) and type of ore		Vertical dimension of the creep zone (ft.) after 100 days	after 300 days	after 1000 days
21	sylvinite	(105)	(90)	(80)
	carnallitic sylv.	(70)	50	45
28	sylvinite	(95)	(90)	(85)
	carnallitic sylv.	50	47.5	45
35	sylvinite	(95)	(85)	(67.5)
	carnallitic sylv.	(105)	55	n.a.

The values listed in Table 4 are random values. The reduction of the initial vertical dimensions of relief creep zones depends on various conditions as mentioned above.

Apparently, less time is required in carnallitic potash than in sylvinite or rock salt. The final vertical dimensions of creep zones in mined-out areas under full overburden load appear to be of the order of 27-30' as shown in Figure 9.

It must be stressed that the effects of horizontal creep may be entirely different in bedded selt and potash, in particular if clay or anhydrite layers act as parting planes above and below openings. Under such conditions, horizontal pillar creep results in differential thrust on the individual separating beds above and below openings, causing the beds to bow into the openings.

Ratio of Vertical Opening Closure Rate to Horizontal Opening Closure Rate

The inter-relationships between vertical and horizontal creep into mine openings in homogeneous rock sālt and potash at depths of about 3,000 ft. may be summarized as follows:

Initial stage: Excavation of rooms causes rapid vertical stress relief creep. Only a few hours are required for partial stress relief in an elliptical zone, the vertical dimension of which may exceed 100 ft. shortly after a room has been widened to its final width.

Initial horizontal relief creep into an opening is more restricted.

Intermediate stage: Stress relief above and below openings initiates horizontal stress relief creep above and below pillars. Horizontal creep finally controls further vertical creep.

Horizontal opening closure rates become higher than vertical closure rates. The vertical dimension of the creep zone is reduced, the horizontal dimension is enlarged.

Final stage: The horizontal dimension of the creep zone is determined by a uniform stress gradient. Near-hydrostatic local stresses increase from near zero at the walls to high values outside the zone of creep where the vertical stresses depend on local overburden loads.

If creep zones overlap in pillars, the horizontal creep is a function of the loads on the pillars. Constant pillar loads result in constant creep rates.

The horizontal creep above and below pillars is converted into vertical creep above and below openings. The volume of rocks above and below openings which perform that conversion becomes a function of the room width. The wider the opening, the greater the volume of rocks within which a given amount of horizontal creep is converted into vertical creep. Consequently, the wider an opening, the smaller is the vertical creep resulting from a given amount of horizontal creep.

These conclusions are confirmed by measurement results from various sites in IMC's mines, some of which are listed in Table 5.

TABLE 5

Ratios of Total Vertical to Horizontal Opening Closure Rates After Beginning of Steady-State Creep in Openings of Different Widths. Opening Height 7.5 ft. Homogeneous Salt Rock Without Bedding Planes. Depth 3140 ft.

Site No.	Opening width (feet)	Ore type	Beginning of steady-state creep (days since mining)	Ratio of vertical/ horizontal closure rates
1	21	sylvinite	300	.75
2	21	sylvinite	550	.69
3	28	sylvinite	600	.65
4	35	sylvinite	400	.6
5	21	carnallitic	400	.6
6	35	carnallitic	200	.3

According to the figures listed in Table 5, vertical opening closure rates can be reduced by increasing the opening width, in particular in carnallitic potash. The initial vertical closure due to relief creep, however, increases with room width.

It must be re-emphasized that these principles do not apply to bedded salt deposits with clay seams which facilitate bed separation. With increasing room widths, the dead weight of sagging beds separated from the roof becomes more effective, increasing the probability that tensile and shear stresses develop in the sagging beds, and result in earlier roof failure.

In homogeneous salt rocks, the remaining horizontal stresses prevent intergranular loosening, and provide for self-support of the roof. Eventually, a homogeneous thick salt bed may sag as a unit if the opening width becomes large enough.

Similar conditions develop in mined-out panels if only small pillars are left for roof support. In such cases, a mined-out area represents a single large opening around which high abutment loads may develop, and result in horizontal creep into the stress relief zones above and below the mined-out area. The vertical extension of these zones is not sufficiently restricted in the centre of mined-out areas over which overburden subsidence has not yet begun. Some of the failures in German potash mines were caused in such a way (Baar 1966).

Relationships Between Creep and Development of Mined-Out Areas

Vertical closure measurement results obtained in the first mined-out block at IMC, K 1 mine, are shown in Figure 8. The locations to which the curves refer are identified by identical numbers in Figure 7, in which the mining sequence is indicated.

The usual mining sequence in a panel may be described as follows: A panel entry system of 3 entries is developed from the block entry system to the back of the panel, as can be seen, for example, near location 14. In that panel, the entry system was mined in 1963. Subsequently, panel retreat mining began around Site 14. Panel retreat mining then proceeded toward the block entry system. This means that, at the time of mining, the area around Site 15 was a remnant pillar area. It was surrounded by mined-out areas on all sides, 4 abutment zones being super-imposed at the time of mining. The development of creep curve 15 demonstrates the consequences.

The creep curves shown in Figure 8 were selected from a large number of similar curves to demonstrate typical trends in creep development as caused by mining and pillar re-loading after initial stress relief creep.

In the following, a short summary as to the causes of changes in creep rates at each particular site will be given. The measurements began at various times after mining. Hence, various amounts of initial relief creep are missing in each curve in Figure 8.

No. 1: Site 000B (Zahary 1965, Coolbaugh 1967 Location about 50 ft. from the main entry in virgin ground (shaft safety pillar). Constant creep rate 1964–1968, not affected by mining. The creep rate is constant at ≈0.3 in./yr.

No. 2: Site 121-A, located near the centre of the first mining panel (Zahary 1965). Relief creep to time R. Later in 1963, increasing creep was caused by retreat mining in that panel, and by mining the adjacent panel. In 1964, the creep rate became fairly constant. Measurements had to be discontinued because of roof slabbing, as was also the case at other sites.

No. 6: Site located in the first room at the panel boundary. Until 1965, the site was protected by an umbrella provided by Block 3. Hence, Curve 6 indicates relief creep to time R. Mining Block 3 in 1965 caused increased loads, which resulted in increased creep. In 1966, the creep rate became fairly constant, but measurements had to be discontinued.

No. 7: Site located in a panel retreat mining area. Initial relief creep and re-loading creep due to partial subsidence follow each other in such a way that the time of the beginning of re-loading cannot be identified. In 1964, the creep rate became constant. However, a slight reaction to 1965 mining by development of slightly steeper slopes can be seen.

No. 12: Duplication of Curve 7 due to similar conditions.

No. 13: Site located in a remnant pillar (at the time of mining), similar to Site 15. High initial creep rates were caused by the resulting combination of relief and re-loading creep. The site was protected by an umbrella provided by large pillars in the block entry system.

For this reason, the constant creep rate indicated through 1964 is lower than the creep rate at Site 12 for the same time period. In 1965, panel mining in Block 1 was discontinued, but some additional rooms were mined in some of the large block entry system pillars. Consequently creep at Site 13 increased.

No. 14: Until 1966, this site was located in an umbrella provided by Block 3. After mining the abutment, the creep developed into steady state creep which continued through 1968, and through 1969-70.

No. 15: Measurements for this curve began several months after mining. Hence, a large amount of initial creep is missing. The reaction to 1966 mining is indicated by development of the final slope of the curve which becomes a straight line, indicating constant creep rates.

No. 10: The creep development at this site has been dealt with under Measurements in Carnallitic Potash. To time R, the creep was relief creep. Mining adjacent areas in 1964 caused an increase in creep. Since 1965, the creep rate is constant,

It should be noted that all creep curves in mined-out areas develop into straight lines with a final slope which is very much the same for all sites, no matter how any particular curve had developed previously due to the particular loading history at that site. As soon as constant loads are imposed on the pillars, the creep becomes steady-stata creep. The final width of the mined-out area shown in Figure 7 exceeds the width required for non-restricted overburden subsidence. Hence, it may be assumed that the pillar loads represent the full overburden load according to depth and extraction ratio at all sites except 1 and 10. The resulting pillar load distributions, and the final dimensions of the zones of vertical creep, are sketched in Figure 9.

The final constant vertical closure rates, as shown by the respective curves in Figure 8 are approximately 3 in./yr., which indicates that the rooms will be completely closed within approximately 30 years after mining.

All sites for which data are presented in Figure 8 are located in sylvinite, except for site 10. Creep rates in carnallitic potash are greater, as demonstrated previously. The resulting final constant creep rates also are higher in mined-out carnallitic areas, shortening the time required for complete room closure. Some of the mined-out carnallitic panels at IMC have been inspected. Observations and measurements confirm the conclusion that less time is required for complete room closure in mined-out carnallitic areas.

CHANGES IN MINE DESIGN AT IMC

Underground inspections and tests

Prior to applying any of the parameters determined by creep measurements to mine design, their validity was checked by inspections of all mined-out areas, and by some additional underground tests which were performed at the request of an outside consultant. Observations in mined-out areas confirmed the conclusions drawn from creep measurement results. No conflicting evidence was encountered. A short description of the underground observations in mined-out areas may be of interest.

Observations in mined out panels

Roof Slabbing

In homogeneous salt rocks, roof slabbing may be initiated by shear stresses which result from the conflicting vertical and horizontal movements near the walls. Roof slabbing is a serious safety hazard in openings to be entered by men. Since horizontal creep is the major cause of slabbing, and since horizontal pillar creep near pillar corners occurs in two directions with reduced creep rates in each direction, it was concluded that opening intersections should be less affected by slabbing.

This conclusion proved correct. None of the many thousand opening intersections in mined-out areas at IMC has failed by roof slabbing.

This "intersection effect" extends to distances which are determined by the horizontal extent of creep zones around openings. Since these creep zones also intersect, they result in less horizontal creep near pillar corners where two creep zones overlap.

The intersection effect will overlap between two intersections at not too great a distance, making such a short room safer with regard to slabbing.

This conclusion also was confirmed by observations in all mined-out panels in which the original pillar design had been applied. In these panels the pillars were 52 ft. wide and $\approx$172 ft. long. No long room was found which had not slabbed (except in umbrella zones), but many of the short rooms had not slabbed even 8 years after mining.

"Umbrella" effect of virgin ground or barrier pillars

As stated before, no roof slabbing was found in umbrella zones along non-excavated areas, indicating the protection from full pillar loading by an umbrella which is provided by roof strata. It is not known which of the overlaying formations provides this protection.

In mined-out areas which exceed the dimensions required for unrestricted subsidence, the width of umbrella zones appears to decrease with time.

Umbrella effects were also found in mined-out panels of an extent less than the depth below surface, if such panels are surrounded at least at two opposite sides by virgin ground.

Underground tests

Roof loading tests

An outside consultant had suggested that intergranular loosening and/or bed separation along anticipated stress arches might contribute to roof failure, in particular in wide openings in carnallitic potash where the carnallite bond between halite and sylvite crystals might deteriorate.

Roof conditions were tested by applying loads up to 700 tons by means of two hydraulic rams seated on the floor, the loads being transferred to the roof by high strength steel pipes.

The vertical roof creep resulting from loading and unloading was measured by pins anchored in boreholes up to 30 ft. deep. Highly sensitive instrumentation was employed to record the differential movement of the measuring pins and the loads.

Testing was carried out under the worst possible conditions, that is, in 35 ft. wide rooms over 200 feet long in the central area of a mined-out panel in carnallitic potash. The adjacent panels had been mined previously. According to Figures 8 and 9, high initial relief creep, followed immediately by re-loading creep, occurs under such conditions.

Application of loads of 700 tons to the roof in the centre of the 35 ft. wide rooms resulted in creep between the roof surface and the 15 ft. pin anchor point. The recorded creep curves closely resembled relief creep curves, the creep rates decreasing with time over periods of several days. The creep was "negative", the distance bwtween reference points decreased due to re-compression of the salt rock in the stress relief zone above the room.

However, the amount of deformation was not of the order of a few inches for loads of 50 to 150 tons, as had been predicted from pre-assumed intergranular loosening or bed separation. The maximum negative roof creep between surface and 15 ft. distance was only .005 in., and that minimal creep was achieved only by maintaining loads of about 700 tons for several days.

No "negative" creep was found for the section between 15 ft. and 30 ft. To the contrary, for this section, continuing positive creep at similar rates was recorded. Apparently, normal stress relief creep was continuing in this section.

Upon roof unloading by releasing the ram pressures, the roof section between zero and 15 ft. showed positive creep, the recorded creep curves being similar to corresponding loading curves.

At the beginning of any loading cycle, the recorded loads decreased as a result of the negative creep that was caused by the loads. Several days were required before fairly constant loads could be maintained.

Upon partial or complete unloading, the positive roof creep caused load build-up in the rams if the hydraulic system was kept closed.

The following conclusions were drawn:

1. Creep does not cause any structural damage in the deforming salt rocks. No intergranular loosening occurs in roof strata even if complete stress relief to the limits of elastic behaviour takes place.
2. The wider a room, the safer it is with regard to shear slabbing.
3. Repeated loading and unloading in situ does not change the creep behaviour of salt rocks.

Pressure cell tests in pillars

U.S.B.M. copper cells were installed in 5 ft. and 10 ft. deep boreholes in pillars.

Over periods of weeks, equilibrium pressures built up which were in good agreement with those found in similar tests in another potash mine (Baar 1966).

Reactions to intentional changes in pressures also were similar: Equilibrium pressures once established were re-established by creep of the surrounding potash, no matter whether the cell pressures were increased or decreased from outside.

These tests also confirmed earlier conclusions (Baar 1966): In the creep zone around openings in salt rocks, local near-hydrostatic stresses develop which increase from the opening surface to the boundary of the creep zone.

Application to panel design

Since all data obtained from long-term creep measurements and from the observations and tests described above were in excellent agreement with data obtained in numerous other potash mines, a number of recommendations were made regarding the most economic and safe design of mine openings.

Depending on the purpose of any particular opening, various principles were applied. Regarding panel entry systems, it was recommended that advantage be taken of the intersection effect.

The mining method applied at IMC requires 3 panel entries, which are developed to the back boundary of any panel. Then the panel is mined in retreat to the block entry system.

The time period for which safe roof conditions must be maintained in panel entries may be up to one year.

Since the intersection effect had been shown to provide safe conditions, the pillar dimensions had to be kept to lengths which would provide overlapping intersection effects. "Safe" pillar lengths had been found to be of the order of 50 ft. - 60 ft., at least for the time period during which the pillars are not fully loaded by subsiding overburden.

Two pillars with the above dimensions, plus three entries say 30 ft. wide, result in an entry system which is approximately 200 ft. wide. Such a width is protected by an umbrella until panel retreat mining begins at the back end of the panel.

Even if roof slabbing begins in the mined-out area, the conveyor belt can be shortened correspondingly, so that there is no need for men to be in the mined-out area.

Using such an entry design for panels, the extraction ratio in the entry system might be higher than the planned overall panel extraction ratio. However, there is no difficulty in designing the room spacing in the panel mining area so that any desired overall extraction ratio well over 50% can be achieved.

Application to block entry and main entry design

At IMC, the original entry height is 7.5 or 8 ft., depending on the type of boring machines. The minimum height required for the lifetime of a travel entry is 6 ft. (to allow relocation of mining machines, etc.).

Hence, only 18 - 24 in. of vertical closure are admissable during the entire lifetime of a travel entry.

It was recommended that full advantage be taken of the stress relief achieved around two parallel openings to be cut prior to cutting the permanent entry between them, and to protect such an entry system by an umbrella to be maintained over the system.

The required parameters were determined by the above evaluation of creep measurements.

Pillar widths in a stress relief entry system should be kept to a minimum in order to achieve the maximum possible stress relief for the permanent entry between the stress relief entries.

The permanent entry also should be kept at minimum width, while the relief entries are more effective if they are wider. However, the overall width of a stress relief entry system must also be kept to a minimum in order to provide for an effective umbrella. The best combination of these two conflicting principles depends on the lifetime envisaged for any particular entry system.

Large abutment pillars must be provided along both sides of a stress relief entry system in order to keep the umbrella effective.

CONCLUSIONS

The parameters dealt with above only apply to the geological conditions for which they were determined.

Different parameters, but similar principles, will apply to different geological conditions, which may change in any single potash mine, or between mines in the same district, as appears to be the case in the Saskatoon area. The only way to determine the parameters for any particular case is by underground measurements. Simulating salt rock reaction to mining in the laboratory appears to be extremely difficult for many reasons, e.g. strain-hardening of laboratory samples, and time requirement for creep.

In stress relief system design, the extent of creep around openings of different shape is of particular importance. Its time requirement under different conditions can only be determined by in situ measurements.

The most important fact to be accounted for in underground creep data evaluation is that stress relief creep inevitably occurs around any new opening in salt and potash. The first deformations near new openings do not indicate high stresses, but stress relief creep caused by the stress differences which are created by excavations in salt rocks under high original stresses.

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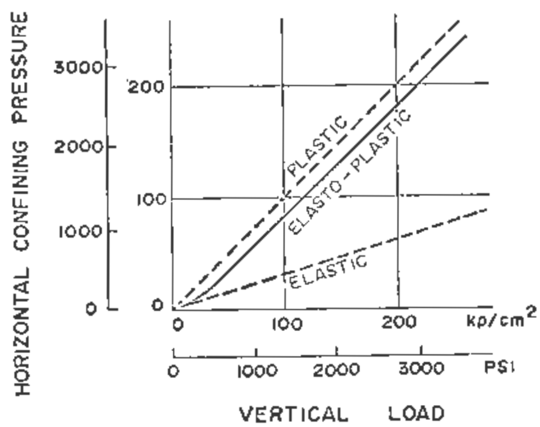


Figure 1. (Dreyer & Borchert 1955, Dreyer 1967) Elasto-plastic reaction of rock salt to vertical loading and unloading. Horizontal deformation is prevented by confinement. The calculated octahedral shearing strength increased from 50 psi under 285 psi to 75 psi under 3700 psi vertical load.

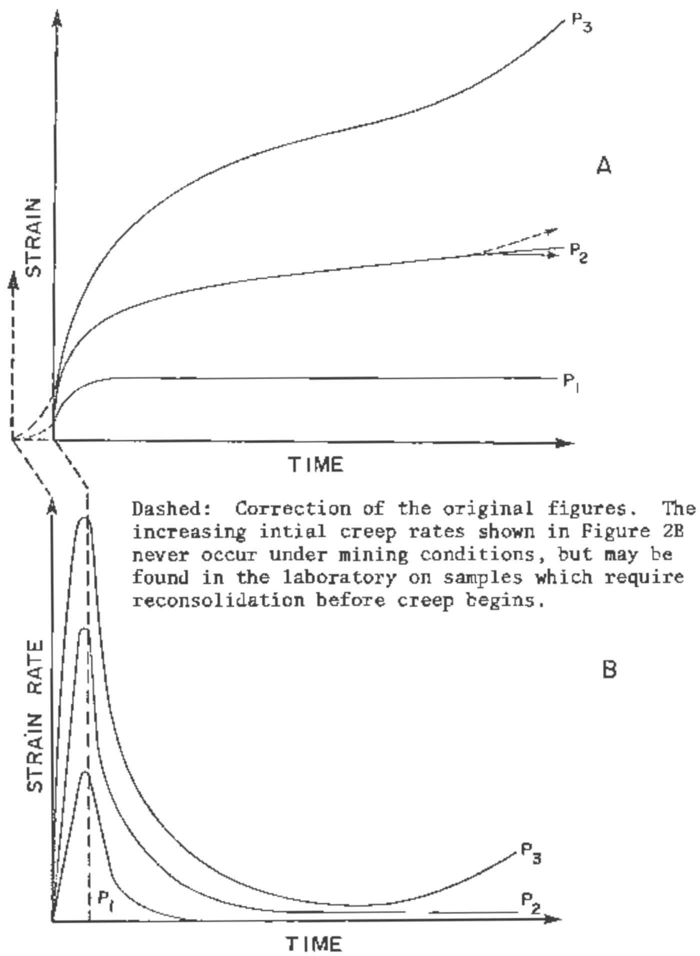


Figure 2. (Uhlenbecker 1971) Postulated development of
A Strain-time curves (after Hoefer 1956, 1964) and
B Strain rate-time curves for mine pillars under loads
$P_3 > P_2 > P_1$.

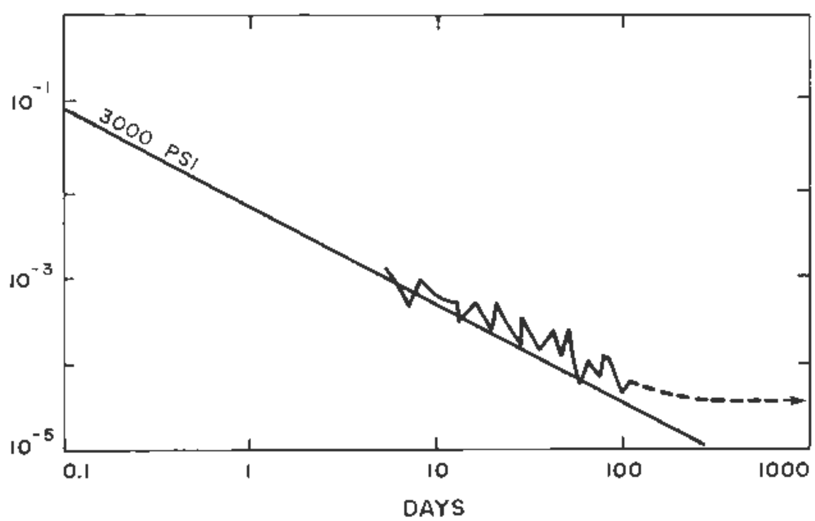


Figure 3. (Coolbaugh 1967) Predicted and measured opening closure rates at IMC.
3,000 psi line = predicted closure rates at 3,000 ft. depth.
Erratic line = measured closure rates of an opening 21 ft. wide by 7.5 ft. high in sylvinite (potash).
Dashed line = measured closure rates added to the original figure according to Figure 8, curve No. 1.

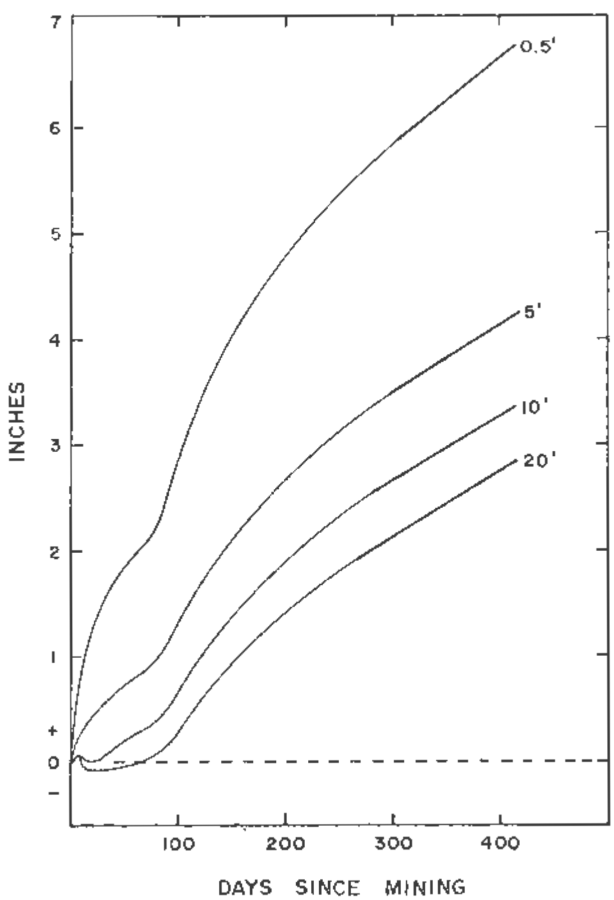
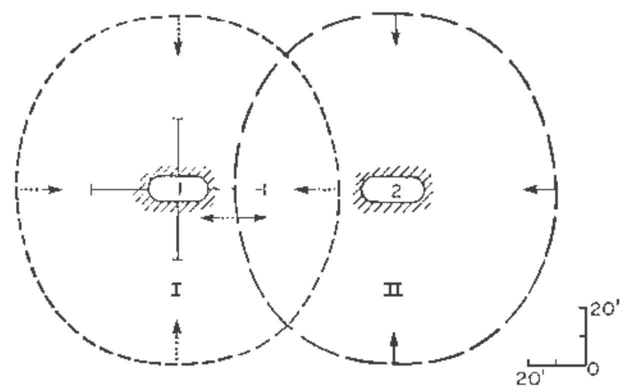


Figure 4. Horizontal movement of points at various depths in a 52 ft. wide pillar between two rooms in potash at 3,140 ft. depths. Openings cross-section 21 x 7.5 ft. For mining sequence see Figure 5.

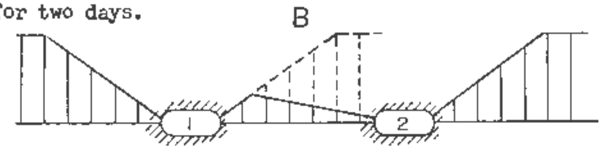
I = First stress relief creep zone around room No. 1
which was mined one week prior to room No. 2.
Measuring pins anchored at distances up to 20 ft.
from room No. 1 as indicated.

A



II = Second stress relief creep zone around room No. 2
shortly after mining room No. 2.
"Negative" movement of the 10 ft. pin (room 1)
for two days.

B



C

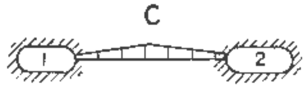


Figure 5. Scheme of stress relief creep zones and pillar load development around two parallel openings according to Figure 4.

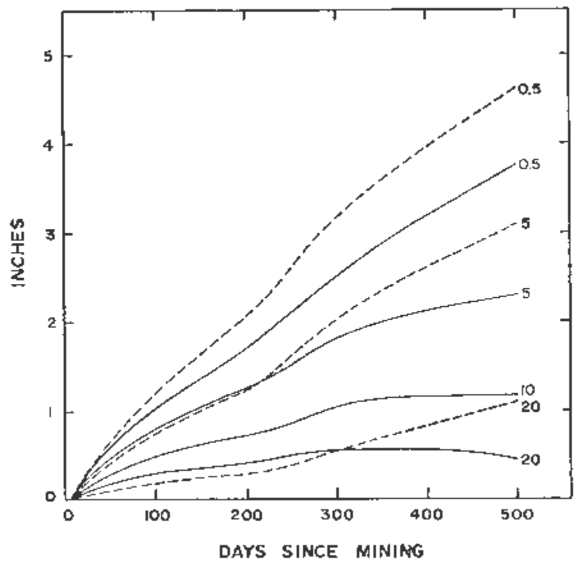


Figure 6. Development of vertical and horizontal creep around an opening 21 x 7.5 ft. in a mining panel in carnallitic potash.
Dashed curves: Horizontal movement of pins anchored at distances as indicated.
"Negative" movement of the vertical 20 ft. pin after 400 days indicates the recompression after initial stress relief creep due to pillar re-loading which resulted in increasing horizontal creep. For development of the vertical dimension of the creep zone as Figure 9.

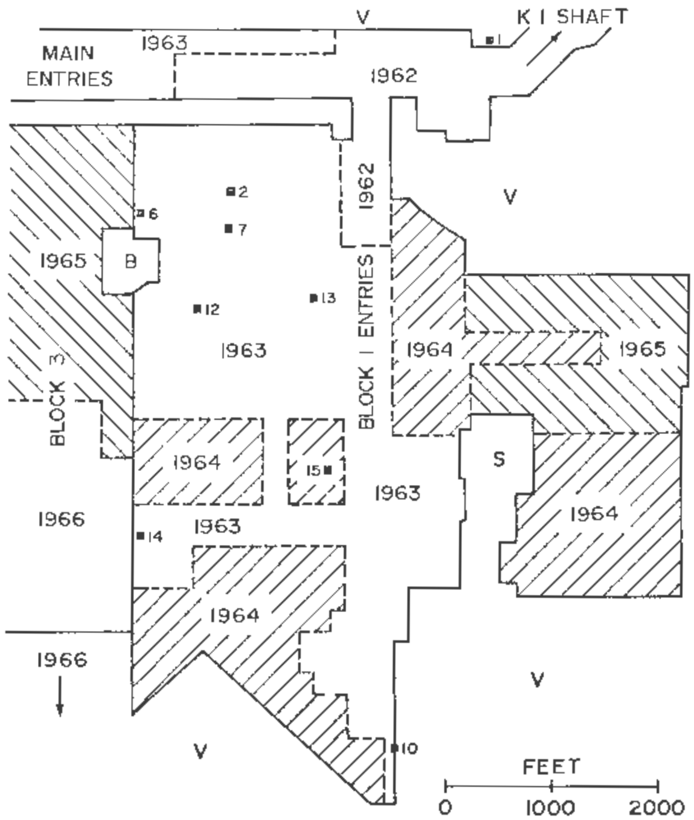


Figure 7. Mining sequence and location of closure measuring sites in blocks 1 and 3, K 1 mine, IMC.
Numbers of measuring sites correspond to creep curves in Figure 8.

V = Virgin ground
S = Salt horses
B = Bore Hole Safety Pillar

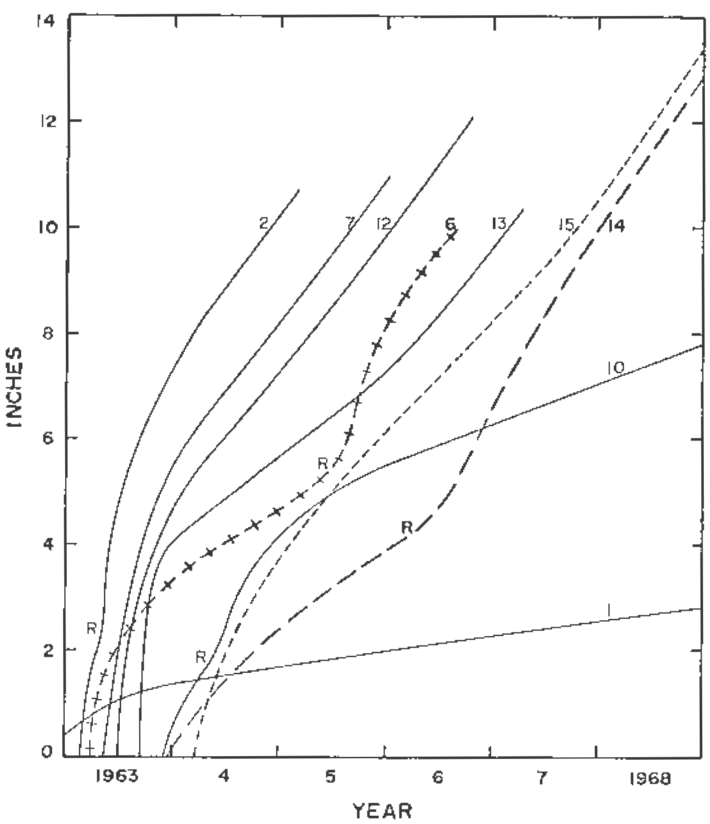


Figure 8. Vertical closure of rooms in relation to the development of the mined-out area shown in Figure 7.
Creep curves identified with the same numbers for the respective locations shown in Figure 7.

R = Relief creep terminated at the respective times. Begin of pillar re-loading indicated by increasing creep rates.

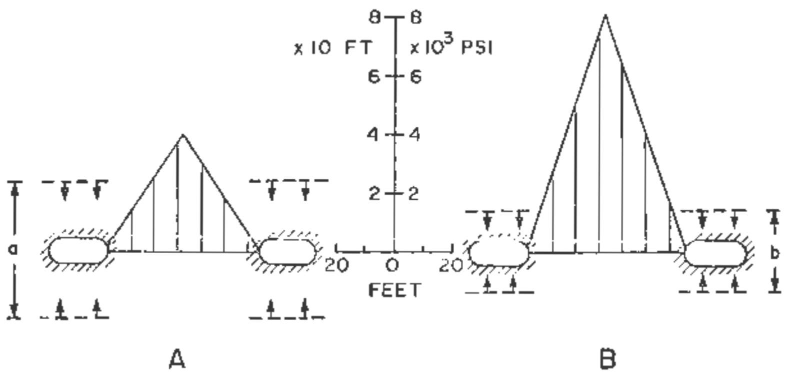


Figure 9. Development of pillar loads and vertical dimensions of creep zones around openings in carnallitic potash in mined-out areas at IMC.

9 A: Scheme of pillar load in an "umbrella" zone. Average pillar load 2,000 psi.
a = Vertical dimension of the zone of creep.

9 B: Scheme of pillar under full overburden load after 33% extraction. Average pillar load 4,000 psi.
b = Vertical dimension of the zone of creep.

A Study of the Causes of Roof Instability in the Pittsburgh Coal Seam

Roger C. Parsons* and H.D. Dahl**

INTRODUCTION

This paper summarizes two years of research into the causes of roof failure in the Pittsburgh coal seam. Although the data was collected from one group of mines in northern West Virginia, the results are believed valid for all of the Pittsburgh seam. The variety of conditions found in the study area cover very well all the geological conditions found in the rest of this large coal field.

The research was initiated in January, 1969, by the Central Research Division of Continental Oil Company and had two major objectives. The first was to define the geological and/or geometrical parameters that affect the severity of roof conditions. This study was concerned with roof fall intensity. The second was to define why roof failures are oriented so that they occur primarily in north-south rooms and entries. This study was concerned with roof fall orientation.

This study will be dealt with in this paper under four main headings:

1. Geological factors which have an observable affect on roof stability.
2. Geological and geometrical factors which are deemed significant statistically.
3. Physical measurements of stress and deformation and their implications.
4. A finite element study which verifies the deformations measured and predicts the mining geometry for increased roof stability.

GEOLOGICAL FACTORS WHICH HAVE AN OBSERVABLE EFFECT ON ROOF STABILITY

Clay veins present a discontinuity within the coal seam. Their shape varies with cross-sections being as in Figure 1. They are continuous in length, some traceable for thousands of feet, and represent ancient drainage systems in the sediments immediately overlying the coal. The largest channels observed were fifteen feet across at the top. They often do not completely penetrate the coal bed and when they do are only inches in width.

The main component of the filling material is silica, the clay minerals being kaolinite and illite. This leads to a hard dense rock with a relatively low compaction ratio on consolidation. Numerous highly polished slickensided surfaces within the filling show a preferred orientation parallel to the axis of the channel. The fractures mate so well that the material forms a barrier to methane flow, yet left exposed will fall loose, and by attrition, form an extensive fall.

The influence of the channel fill is felt for many feet

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from the axis, the sediments themselves showing slickensided surfaces formed due to differential compaction during the consolidation process (Figure 1).

The main effect of a channel filling, regardless whether it is observable in the coal or does not actually penetrate, is to greatly reduce the strength of the roof rock in the vicinity.

Large sand lenses create areas around their borders of high differential compaction. They create broad zones of low roof rock strength necessitating more elaborate roof support techniques.

Zones under existing valleys can sometimes produce highly fractured roof rock similar to that found around channel fillings. The reason for this is that present day drainage is influenced by the ease of erosion which it encounters in an area where fracturing exists. Through geologic time, drainage systems become the locus of fracturing due to differential compaction. This increases the probability that any drainage system in the future will run parallel to a pre-existing one. A highly fractured zone in the roof rock of the coal seam under a valley is usually not caused by the valley. More often the valley is caused because the fracture zone exists.

Cleat systems within the coal are well developed orthogonal vertical sets of two different frequencies. This affects the degree of surface spalling of a pillar; and, if a punching problem of pillar into roof and floor exists, can affect the magnitude of the shear stresses developed in the roof and floor on adjacent pillar sides. Higher roof shear stress would be expected on the most rigid side. In the Pittsburgh seam, the high shear would be expected in north-south rooms and crosscuts. It will be shown later that most of the differential dilation observed in pillars in this coalfield can be explained by a lateral east-west stress field.

GEOLOGICAL AND GEOMETRICAL FACTORS WHOSE EFFECT HAS TO BE DETERMINED STATISTICALLY

To determine those variables which have a significant effect on the areal distribution of roof falls and their intensity, analysis of variance was undertaken. Specific methods used were multiple curvilinear regression and component discriminent analysis.

The variables which can affect the intensity of roof fall are many; and some, such as span, extraction, and shape, are dependent on one another because of geometry of the opening and mining machine design. It can, therefore, be very misleading to conclude cause and effect relationships unless all the variables are considered together. For example, it may appear that large pillars down the centre of a multiple entry system have increased stability. In reality, it may have decreased it; but the decrease is offset by a combination of a change in roof span, cover load, and strata curvature. If one could move from one area underground to the next with only one variable changing, the cause-effect relationship could be determined. It is not possible to this underground; however, it is possible to collect information on all the variables at a large number of locations and construct a mathematical model. From this, one can cause one variable to change at a time and observe the effect. A multiple curvilinear regression analysis produces such a model.

Roof falls underground have been classified based on the depth of material fallen from the roof on the following basis:

Rank No.	Description
0	No visible deterioration.
1	Visible cracks in roof. No shale fallen.
2	Coal and shale fallen from the roof to a depth of 2 feet.
3	Material fallen from between 2 and 4 feet.
4	Material fallen from 4 to 8 feet.
5	Roof fallen to a depth greater than 8 feet.

Data was selected from sixty fall areas comprising ten of each rank. At each fall area, twelve independent variables were collected. Each of the variables listed below were assumed to affect roof fall intensity.

(1) Depth of coal seam - X_1.

(2) Curvature of strata - X_2. From an underground survey, elevation contours in the coal seam were drawn on a one foot interval. Cross sections were then drawn perpendicular to the strike of the coal bed passing through the bad roof area. The slope of the strata at two points 100 feet apart was measured from the sections. The second derivative was then approximated by

$$\frac{d^2y}{dx^2} = \frac{\frac{\Delta y_1}{\Delta x} - \frac{\Delta y_2}{\Delta x}}{\Delta x}$$

where y is vertical distance and x is horizontal distance.

(3) Percent extraction - X_3.

$$\text{Percent extraction} = \left(\frac{(a+d)(c+b) - ab}{(a+d)(c+b)}\right) \times 100$$

a = length of pillar
b = width of pillar
c = width of entry
d = width of crosscut

(4) Head coal thickness - X_4. This is the thickness of coal left in the roof by the mining machine.

(5,6) Number and cumulative thickness of rider seams - X_5 and X_6.

(7) Roof span - X_7. In the ripper openings, this was measured across the span between the roof rib lines. In the borer openings, the maximum width of opening was taken.

(8) Shape of opening - X_8. This was recorded as either flat (0) or arched (1).

(9) Bolting pattern - X_9. The pattern was ranked from 1 to 5, based on the number of bolts and type of bolt pad used.

(10) Entry height - X_{10}.

(11) Coal thickness - X_{11}. Where possible, it was measured directly. Where the roof and floor shales were not exposed, it was calculated as the sum of the entry height plus the head coal thickness.

(12) Cleat frequency - X_{12}. This was checked at three points per site over a gauge length of one foot and averaged.

MULTIPLE CURVILINEAR REGRESSION ANALYSIS

The computer program REGANL performs the computations necessary to determine the least-squares estimates of the parameters θ_i, in a response function:

$$y = f(x_i, \ldots x_q, \theta_i, \ldots \theta_p) + \varepsilon \qquad (1)$$

where y is the dependent variable, x_i are the independent variables, θ_i defines the functional relationship, and ε is an error term. The parameters are estimated from the form:

$$Y = \beta_0 + \beta_1 Z_1 + \beta_2 Z_2 + \ldots \beta_k Z_k + \varepsilon \qquad (2)$$

where Y is a known function of y, the z_i are known functions of the x_i's and ε is again an error term. For example, Z_2 could be X_1^3, $Z_1 = X_1X_3$, etc.

Given $N > k + 1$ observations of the x_i, the least-squares estimate of the β's are obtained by minimizing:

$$\sum_{i=1}^{N} (Y_i - \hat{Y}_i)^2$$

where the subscript $_i$ now refers to the i^{th} observation and $\hat{Y}$ is the least-squares estimate of ^{v} as determined by (2) for the i^{th} observation.

The computer program allows terms to be deleted (or added - in various functional forms) until a form is obtained which contributes significantly to the fit. An F test is used to evaluate the significance of the β's and to estimate the probability of error incurred by concluding that any $\beta_i = 0$. Of the twelve variables included in the analysis, four (span, overburden, percent extraction, and strata curvature) resulted in the best fit. Only two of these can be varied underground. Of course, not all possible forms of equation (1) were tried, and it is possible that a better equation could have been obtained. The final regression equation is given by

$$\text{Rank} = -1.97 \times 10^{-3}X_7^2 + 7.04 \times 10^{-4}X_1X_7 + 1.08 \times 10^{-2}X_7X_3 \qquad (3)$$
$$-3.75 \times 10^{-3}X_1X_3 - 3.12X_3X_2 + 7.40 \times 10^3X_2^2$$

The equation has a multiple correlation coefficient, $R = 0.85$ ($R = 0.92$). In other words, 92 percent of the variation can be explained by the regression model.

DISCRIMINANT ANALYSIS

Roof fall data was also analyzed by discriminant analysis. The method requires that the number of positions in each set be larger than the number of observations at each position. For the roof fall data, this means that the number of degrees of roof fall that can be analyzed must be reduced. For this reason, roof falls were divided into two groups: Group I included roof falls of degree 0, 1, and 2, while Group II contained roof falls of degree 3, 4, and 5. Both linear and non linear discriminant analyses were performed. Discrimination is measured by Mahalanobis distance--a large distance reflects a large difference between Groups I and II.

Results of the linear analysis showed that the maximum distance (3.40) between groups occurs when the nine quantitative variables--overburden, roof span, extraction, height, second derivative, thickness, weighted rider seam thickness, head coal thickness, and summation of rider seam thickness--were included. The F statistic has a value of 3.64. The prediction equation is given by

$$\text{Rank} = 0.0025X_1 - 124.3X_2 - 0.259X_3 + 0.1725X_4 + 0.0025X_5'$$
$$- 0.995X_6' + 0.0843X_7 + 0.053X_{10} + 0.00225X_{11} \qquad (4)$$

where X_5' = weighted rider seam average; weighted from top of entry.
X_6' = summation of rider seam thickness

Reducing to the five variables--roof span, extraction, second derivative, weighted rider average, and summation of rider thickness--the distance becomes 2.91, and the test statistic increases to 6.20. Thus, a reduction from 9 to 5 variables causes a 15 percent reduction in distance but results in a considerable improvement in the F statistic. The improvement in F statistic reflects a reduction in degrees of freedom. The prediction equation is now given by

$$\text{Rank} = 111.4X_2 - 0.241X_3 + 0.0021X_5' - 0.097X_6' + 0.807X_7 \quad (5)$$

Note the closeness of coefficients between equations (4) and (5).

The contribution of each variable can be illustrated by multiplying the mean values of each variable by the corresponding contribution in the prediction equations given above. Roof span is by far the most important variable, and per cent extraction appears to affect severity in an inverse fashion. It may be worthwhile to note here that the existance of a lateral stress field could help to explain this apparent discrepancy.

The nonlinear rank prediction equation is given by

$$\text{Rank} = 7834X_2^2 - 84.6X_2 - 1.33X_3 - 0.0003359X_7^2 + 0.5866X_7X_2 + 21.6 \quad (6)$$

The F statistic is 6.14; difference between the two sets of roof falls is highly significant.

The coefficient in the prediction equation should not be treated as "God-given". Neither should the gradients or intercepts indicated by the prediction equation. The importance of both techniques in analyzing the roof fall data is the reduction in number of variables and their relative importance. Roof span is the most important, followed by second derivative, per cent extraction, and the rider seam terms. The regression analysis, in the large, supports this result, with the exception of rider seams. The contribution of the latter is small enough to make the discripancy between the two analyses minimal. It is worthy of note that in the discriminant analysis, roof span explains 61 per cent of the "difference" between groups.

DISCUSSION OF REGRESSION ANALYSIS

Per cent Extraction (X_7 and X_9 Constant)

The model predicts an increase in intensity in the range of from 29 per cent to 41 per cent extraction with the rate of increase continually decreasing. The expected fall intensity maximum was 40 per cent extraction (for a span of 14 feet at 600-foot depth). For extraction ratios greater than 0.41, increased stability is predicted, becoming zero at 53 per cent extraction. This prediction, as with span, is beyond the input data range.

Strata Curvature (X_1, X_7, and X_8 Constant)

Greatest roof stability is obtained in areas where no flexure exists in the roof. A decrease in stability occurs as the curvature increases, approaching both anticlinal or synclinal axes. This is due to the increase in fracture intensity near the fold axis.

Span (X_7 and X_9 Constant)

As the span increases from 12 to 16 feet, the rank continually increases, but at a decreasing rate, maximizing at 16.5 feet. Spans in excess of 16 feet reduce the probable fall intensity. Since the large spans are generated as a result of cutting into the pillar, the increased stability predicted may be due to the increased pillar yield. It is believed that pillar stiffness is a function of pillar dimensions, and that an optimum pillar size exists which will allow a minimum roof deterioration.

Variation of Cleat Frequency

This was measured on the face cleat only and variations in frequency were not great. It is believed that the degree of development of the cleat systems could affect spalling of the rib if significantly different in other coal seams or in other areas of the Pittsburgh seam.

Opening Shape

The shape of the opening is related to span due to the geometry of the mining machines. Flat ripper openings have a range of from 13 to 17 feet with a mean value of 16 feet. This family of spans differs from the borer, which varies from 12 feet 6 inches to almost 14 feet, with a mean value of 13. Initially, when using data from both shapes of openings, shape remained in the equation as being a significant variable. However, when the data was split and only one shape used, the overall correlation coefficient of the regression equation increased. The better fit obtained suggests that decrease in fall severity obtained when going from a ripper to a borer machine is due to the span reduction and not to the shape.

Head Coal Thickness

The thickness of head coal, which varies from a few inches to over one foot, was not significant. However, when missing entirely, roof deterioration does start and often progresses to a fall. In this particular area, it is believed that its presence is significant, but its thickness is not important as long as the roof material is protected from weathering. The fact that thickness variation is not important suggests the head coal neither contributes to nor detracts from the strength of the roof.

Bolting Patterns

The bolting patterns used were ranked from zero to five, based on the number of bolts per unit area and the degree of planking. It is believed that since the bolt anchorage horizon is so poor, no real measure has been made of their effectiveness.

Coal Rider Seams

The number of coal rider seams is remarkably uniform at three, and its significance as a variable is not fairly tested. Coal riders laminated between shale and sandstone imply high facies changes, and the number should be an index of roof strength.

--Thus, the statistical study indicates that the significant variables, in order of decreasing effect, are roof span, per cent extraction, second derivative, depth of cover, and the number of rider seams, the the last two only marginally so.

It is interesting to note that the geometrical variables are the most important. This is fortunate since these are the only ones which we are able to manipulate, i.e., span and per cent extraction. The suggestion of increased stability with large spans and higher extraction ratios, and the low significance of the vertical overburden load suggests we may be dealing with a high lateral stress field.

Measurement of Pillar Dilation and the Regional Stress Field

It is readily observable underground that east-west rooms undergo heavy spalling along their ribs and that the roof frequently exhibits tensile fractures. In north-south rooms, ribs are rigid and shear failure frequently occurs in the roof adjacent to the rib (Figure 2).

The magnitude of this differential pillar dilation was measured in development sections by installing three-anchor dilation rods on the four pillar sides. Typical results in Figure 3 confirm that a differential pillar dilation does exist at least to a depth of 15 feet and that in the outer 3 feet the dilation into an east-west room is an order of magnitude greater than into a north-south room.

Similar measurements taken on pillars where a gob line was rapidly advancing fail to exhibit this differential phenomenon (Figure 4). In this area there is an increased vertical load due to increased extraction, and any horizontal tectonic stress would be deflected higher above the opening.

The conclusion is that for the differential dilation phenomenon to disappear, it could not have been a physical property of the pillar originally. It must, therefore, have been stress induced. That stress must also be in a lateral direction. Previously, a lateral principal stress was suggested by the lack of high correlation between roof fall intensity and overburden depth.

In-Situ Stress Measurements

The hydrofrac technique was chosen to measure the stress field at four locations several miles apart. This method, along with available alternatives, is discussed by Fairhurst (1968).

A hole is drilled parallel to the direction of one principal stress. The distribution of stress around a long cylindrical hole is the well-known problem of Kirsh and is available in most elasticity textbooks, e.g., Wang, 1953. Internal pressure applied to a packed-off section of the hole causes a redistribution of stress around the hole; and, at sufficient pressure, a fracture is initiated in the hole. Fracture initiates in the direction of the major compressive stress, perpendicular to the hole axis.

Hydrofrac equipment used in the study is shown in Figure 5, and consists of an airdriven high pressure pump (up to 15,000 psi), hand pump, fracture packer, impression packer, down-the-hole pressure transducer,transducer power supply, and recorder. The fracture packer consists of two 6-inch long rubber inflatable packers which seal off a zone for the high pressure oil. The hand pump is used to inflate the packers, following which the zone between packers is pressurized by the air-driven high pressure pump.

Figure 6 is an actual pressure curve obtained from a depth of 97 feet. This curve is typical of a good fracture obtained from competent strata. A rapid pressure buildup to the fracture initiation pressure is followed by an immediate pressure drop to the fracture propagation pressure. If the pump is shut off, a constant pressure slightly below fracture propagation pressure is obtained.

An impression packer is used to determine fracture shape and orientation. The impression packer is wound with neoprene rubber which flows under pressure and generates an impression of the hole surface at the desired location. General procedure was to fracture in the hole at several locations keeping the fracture packer in the hole, starting from the deepest point in the hole and working toward the surface. Those locations having good pressure curves were then impressioned for fractures. In particularly broken ground, it is necessary to impression the hole prior to hydrofracing to pick up any pre-existing fractures, bedding planes, or joints.

It was assumed that one principal stress direction is vertical. This assumption is based upon the relatively shallow depth of cover and the presence of horizontal bedding planes. Four vertical holes were drilled; three into the roof and one into the floor.

The histogram of Figure 7 presents the hydrofrac data. The major principal stress in the horizontal direction is at an azimuth of approximately 100° with the minor principal stress orthogonal to it and vertical.

σ_1 = 1500-2700 psi azimuth 96°
σ_2 = 800-1000 psi azimuth 186°
σ_3 = 720 psi vertical, overburden

These magnitudes are subject to some interpretation, as stresses were measured in the most competent beds. It may be expected that these beds are relatively "stiff" and, therefore, assume a larger proportion of the lateral load than adjacent softer beds.

FINITE ELEMENT ANALYSIS

For prediction and analysis of ground movement due to mining, the ground is considered to be elastic-elastoplastic. The finite element method was used for analysis; and the effects of layered media, initial gravity-induced stresses, and non-linear post-yield deformation were included.

The first analyses were performed to confirm the field measurements. The first finite element idealization of a rectangular opening in coal at depth is shown in Figure 8. A five-layered system is shown with geometry and properties as indicated. Only half the room need be analyzed due to symmetry. The ground is assumed to be initially stressed by gravity and the opening presumed not to exist. The opening is created by incrementally removing the loads supported by the material comprising the opening. This feature of the analysis is necessary due to the load dependency of a non-linear analysis.

First, two cases were analyzed; one having a horizontal to vertical stress ratio of two (σ_v = -720 psi, σ_h = -1440 psi), and one with normal lateral restraint assumed (Poisson's ratio = 0.25, σ_v = -720 psi, σ_h = -240 psi). Room geometry is 6 feet high by 14 feet wide. Figure 9 shows pillar strains predicted for the two cases. Strains were plotted at one-half room height as a function of depth into the pillar, identical to extensometer measurements. Total strains are computed by the analysis, whereas incremental strains are measured by the extensometers (Figure 3); therefore, strain magnitude cannot be compared. However, the curves are remarkably similar in shape to the field curves. Thus, high lateral stresses in an east-west direction could cause the pillar movement observed in the field.

Figure 10 shows the shear strain distribution around the roof of the opening. Shear strains are contoured and the maximum connected. The process is similar to following a ridge on a topographic map. With lateral restraint only (presumably an east-west room) the shear locus is vertically above the rib and does not grow over the opening. High lateral stresses generate a shear locus arching over the opening and connecting at the middle. Tension is indicated in the roof over the lateral restraint only room, and no tension appears in the other case. Tension cracks are observed underground in east-west rooms.

This analysis showed that the deformational characteristics of both stable and unstable rooms in both directions could be predicted mathematically and that it could be due to the existance of high lateral stresses in an east-west direction and the lack of such stresses in a north-south direction. Taken together with the in-situ stress and extensometer measurements, the evidence is conclusive: the cause of the north-south roof fall orientation is the presence of a biaxial lateral stress field with the major compressive component in an east-west direction.

Having identified the cause of the orientated roof falls, it remains to discuss what can be done about them. Outside of roof support, the items that can be changed underground are (a) orientation of mine workings, (b) room geometry, and (c) pillar geometry. In a new mine, the easiest to implement is orientation; in an old mine with well-established haulage routes, it is very difficult. Room and pillar geometry, however, can be changed within certain limits set by machinery size and law. It was, therefore, decided to analyze the effects of room and pillar geometry changes in the light of a finite element idealization, as this method has been able to predict the conditions existing underground. North-south rooms only are considered.

Wide rooms were considered because of the high horizontal to vertical stress ratio, implying that rooms having a height to width ratio of less than one are desirable. The question is how wide, and finite element analysis can provide an indication. Narrow pillars were analyzed because it was believed that sufficiently narrow pillars could not support significant horizontal loads, thereby subjecting the immediate roof and floor to a reduced horizontal load. In terms of room orientation, the best direction to drive rooms is 96° azimuth and the worst is 186° azimuth. Precluding an analysis, it is expected that a gradual shift between these extremes occurs, and that no optimum exists other than 96°.

Room Geometry

The finite element model for studying changes in room width (room height is fixed by coal thickness) is depicted in Figure 11. An initial stress state of $\sigma_y = -1440$ psi, $\sigma_x = -720$ psi was assumed. Room width was varied in 4-foot increments from 8 feet wide to 24 feet wide. Room width was varied by successive removal of elements comprising the pillar edge. Due to symmetry, only one-fourth of the model need be analyzed. Results are as follows:

Relatively wide rooms (20 to 24 feet) result in a generally reduced state of shear strain. In addition, yielding of the rock immediately above the opening occurs at somewhat higher load levels. It is true, however, that wider rooms, when they fall, will result in larger volumes and greater heights of fallen material. What is implied by these results is that wider rooms would have a smaller probability of failure. There will always be areas which are relatively weak and will be subject to falls. This analysis suggests that with wider rooms, failure may result at a later date or not at all. It should not be implied that all falls will be eliminated. There is no shear strain criterion available upon which to assess absolute stability; only comparative stabilities can be assessed by the methods presented herein.

Pillar Width

An improved roof condition is predicted with a narrowing of pillar width. Almost no change could be noted from 46-foot pillars down to 22-foot pillars. A slight improvement in shear strain is noted in the 22-foot pillar, but yield zone development is not noticeably changed. At 18 feet, however, a noticeable change in both maximum shear strain and yield zone development has occurred. This trend continues through the 10-foot pillars. At these pillar sizes, particularly the 10-foot pillar, pillar shears start to rise and pillar stability would be a problem if long term stability is required.

There is obviously some pillar width which is impractical due to problems of pillar deterioration and bottom heave. However, there is certainly some pillar width which optimizes pillar, roof, and bottom stability, taken as a unit.

CONCLUSIONS

The consistent orientation of roof falls in a north-south direction is due to an east-west tectonic stress. Roof fall intensity at any given opening was found to be affected by room width, strata curvature, overburden thickness, rider seams, and extraction ratio.

North-south failures were found to be caused by an east-west compressive lateral stress. This stress field accounts for shear zones (cutters) in the roof strata of north-south rooms, the rigid ribs along north-south rooms, high spalling of ribs in east-west rooms, and tensile cracks in the roof of east-west rooms. This result is supported by in-situ stress measurements, field deformation measurements, and by analysis of finite element methods.

The aerial distribution of roof falls throughout the mine areas would be related to variations in the strength of the roof rock. This strength in turn is a function of strata composition and facies change and of the degree of folding.

The phenomenon of roof fall orientation and the density of roof falls can be reduced by changes in room geometry and orientation. However, even when stress concentrations are minimized to their lowest value, areas will still exist where the roof strength is less than this value and a fall will occur.

Although one can do little about geologic changes in roof strata, one can anticipate areas of high fall density due to changes in geological variables and prevent many by various support techniques.

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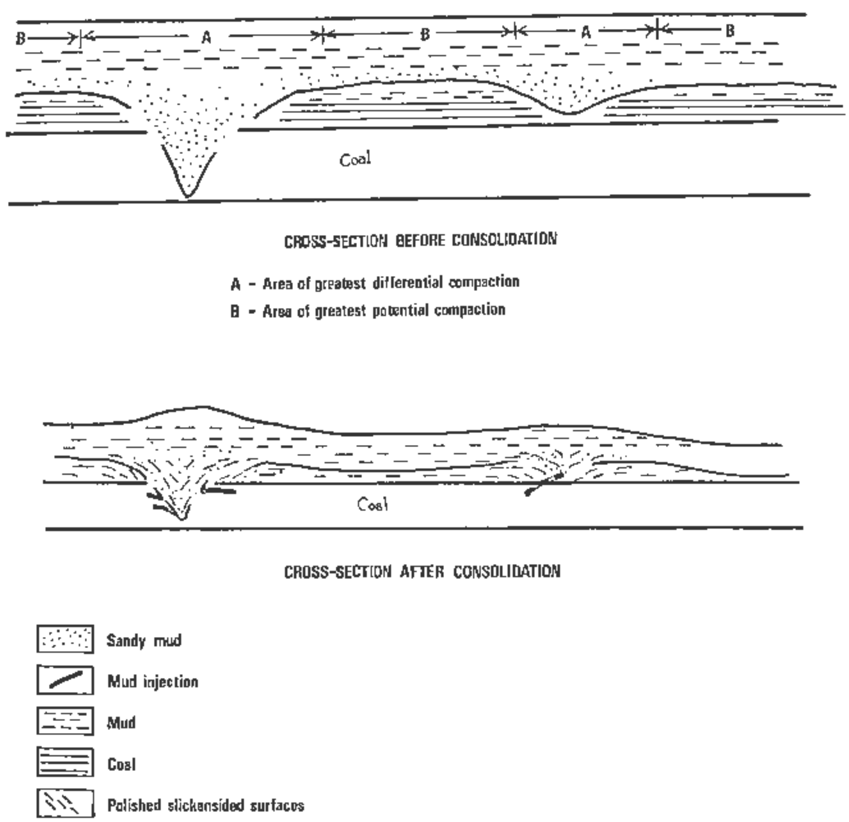

Figure 1. Consolidation prosess.

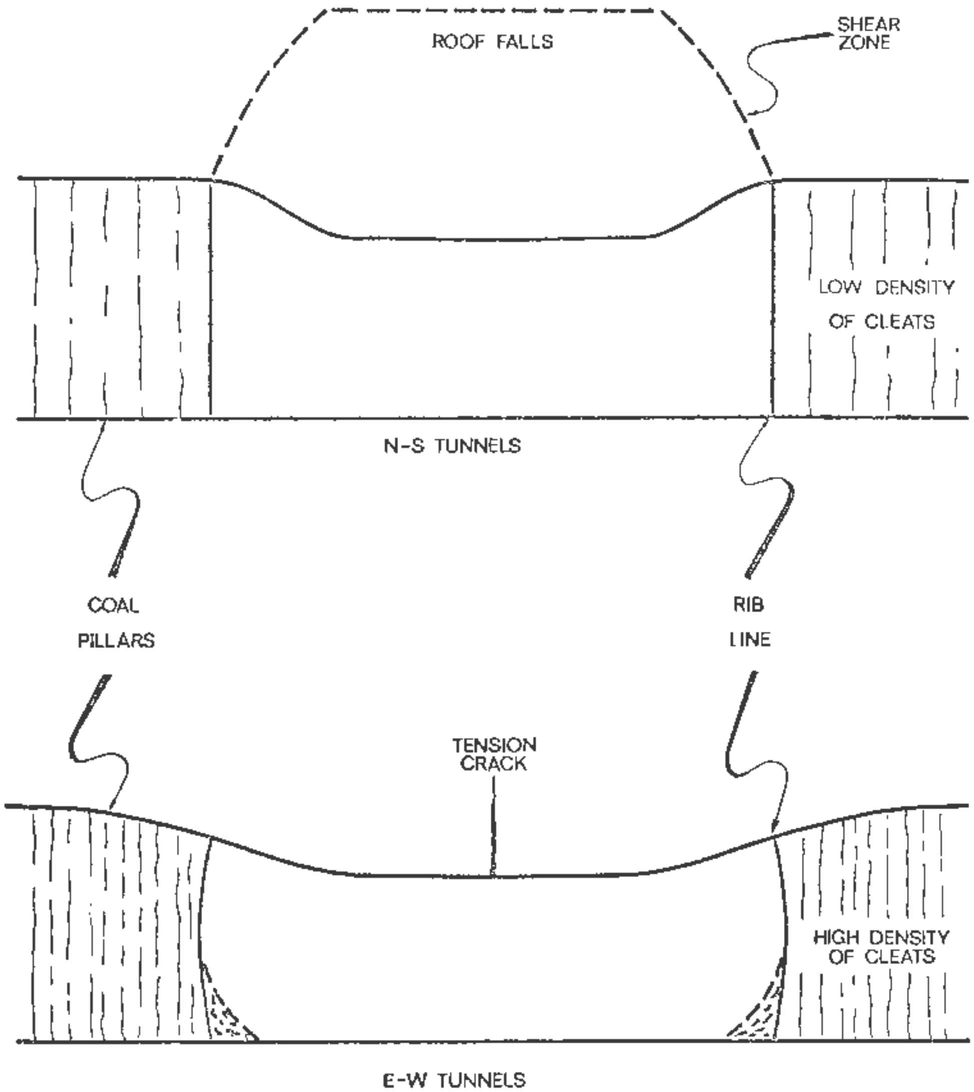

Figure 2. Typical conditions of north-south and east-west rooms.

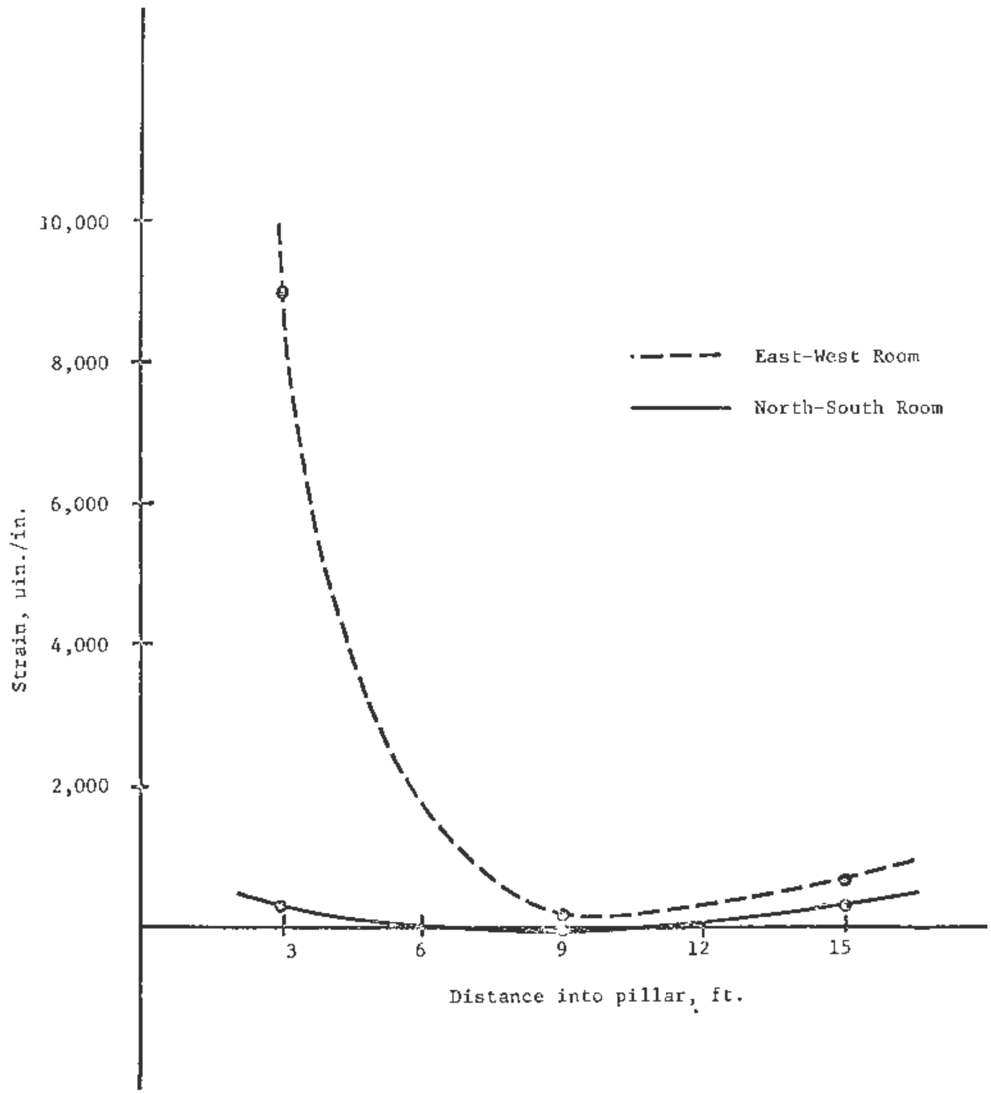

Figure 3. Pillar strains in a pillar of B-11 section in development.

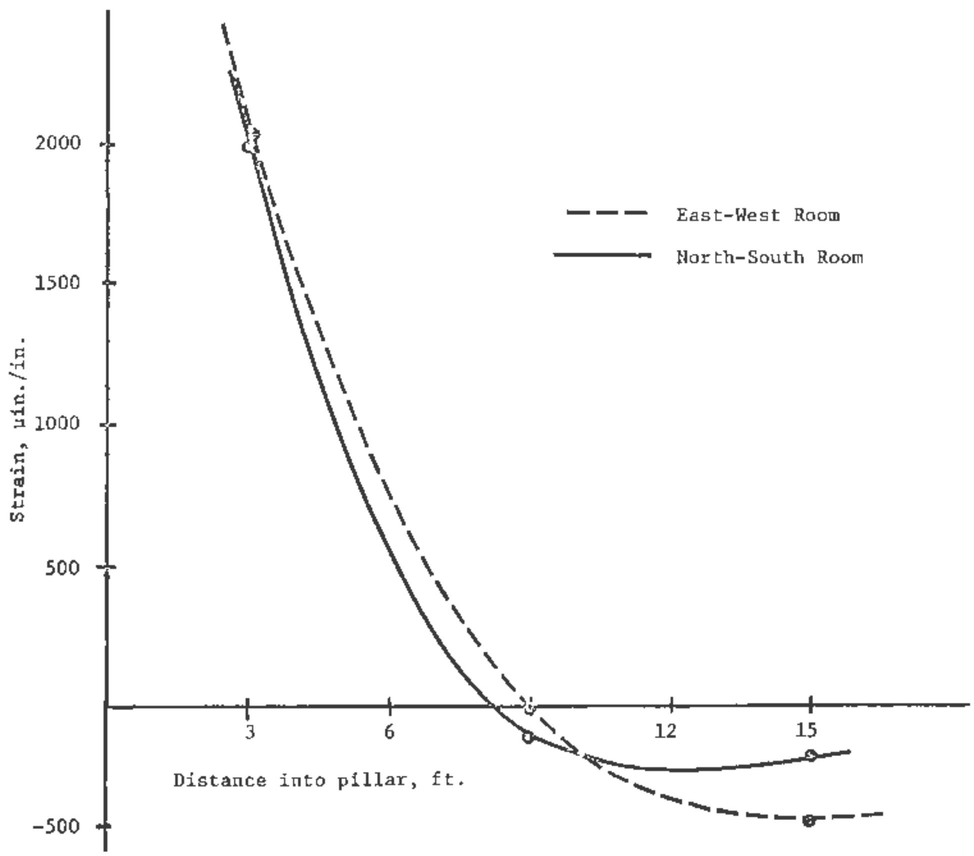

Figure 4. Pillar strains in a pillar of B-11 section in retreat.

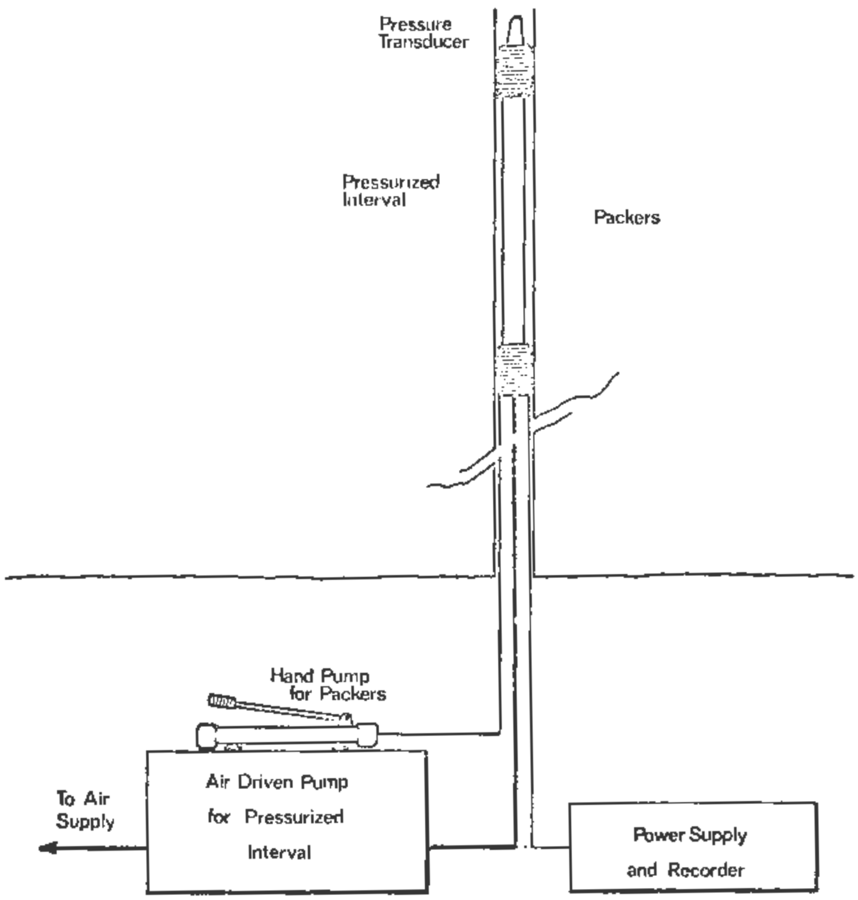

Figure 5. Hydrofac equipment and set-up.

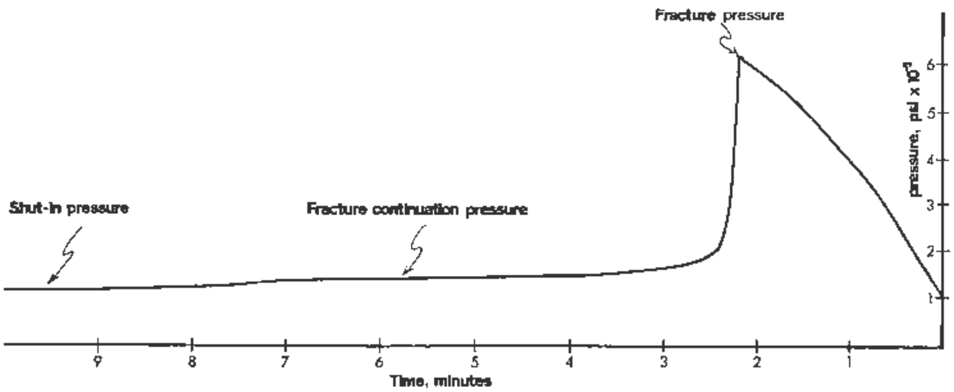

Figure 6. Pressure curve from hole No. 4 at 97 feet into the floor (sandstone formation).

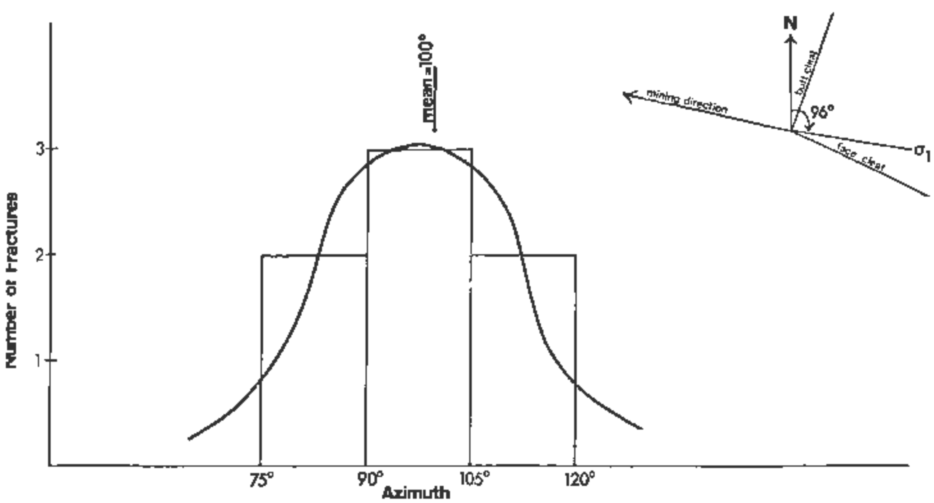

Figure 7. Stress measurements by the hydrofracture method. Histogram of fracture orientation.

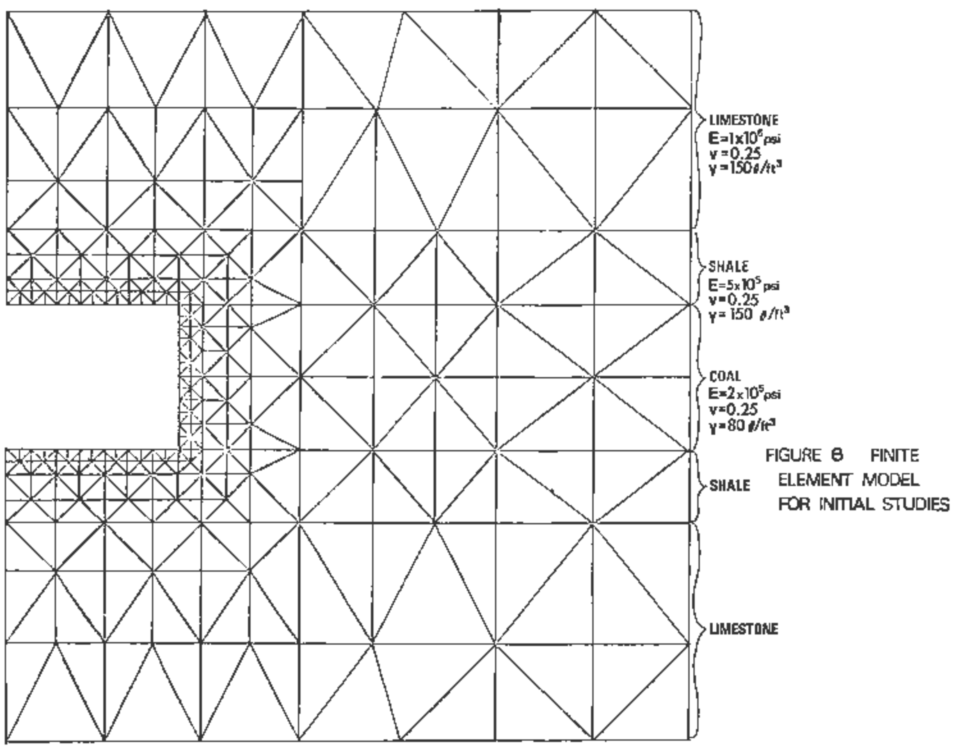

Figure 8. Finite element model for initial studies.

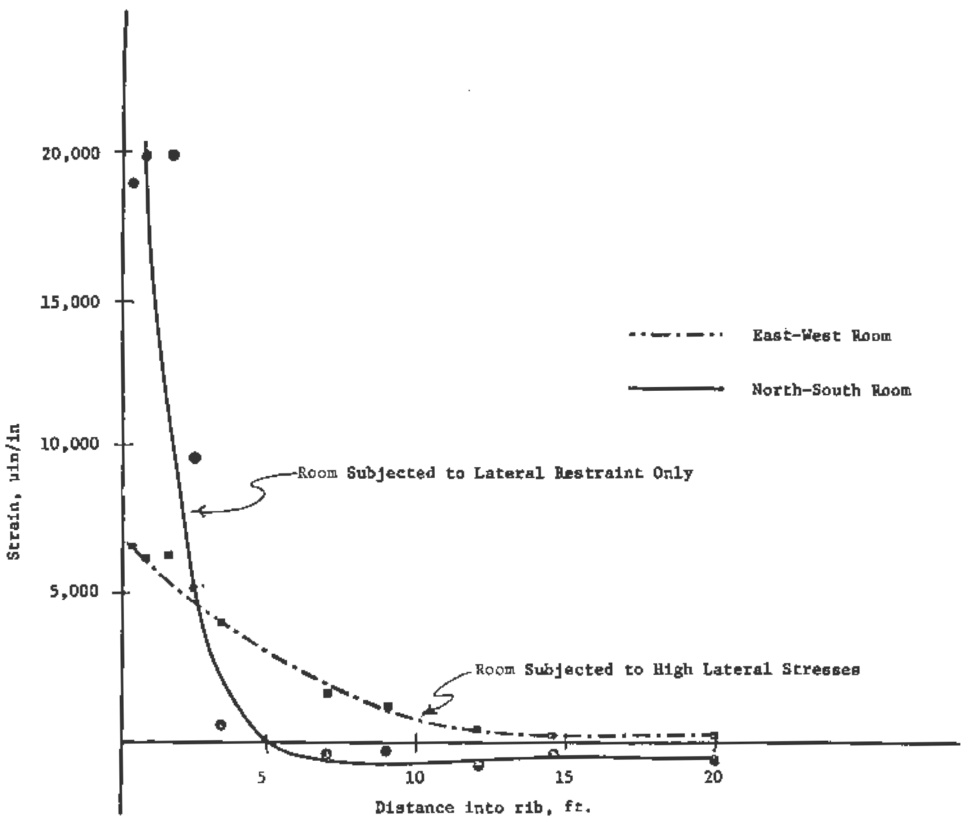

Figure 9. Pillar strains for rooms subjected to high and low lateral stresses.

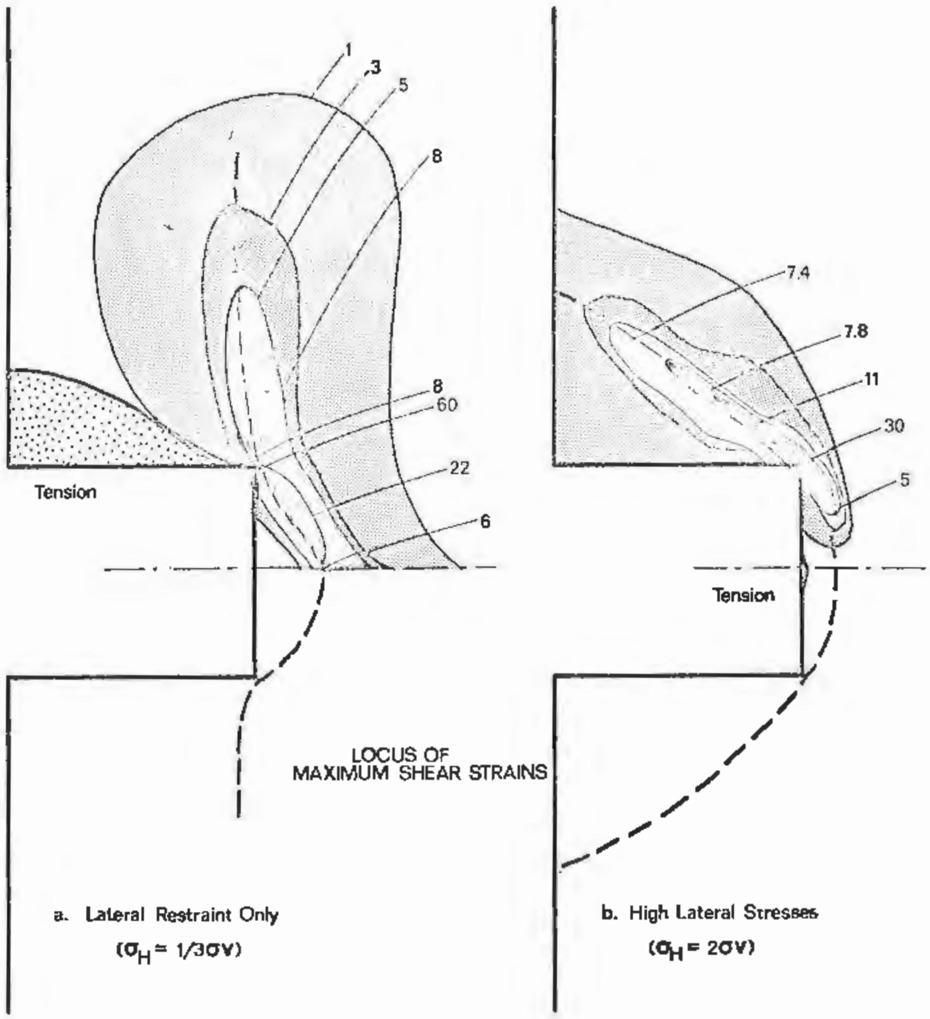

Figure 10. Shear strain contours around rectangular rooms.

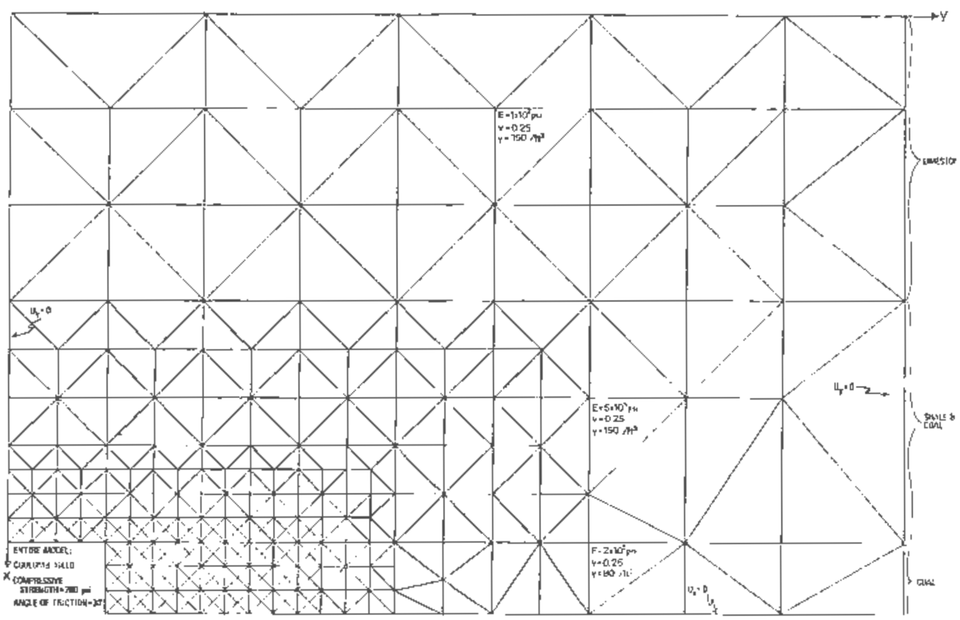

Figure 11. Room width variation finite element model.

The Serata Stress Control Method of Stabilizing Underground Openings

Shosei Serata*

INTRODUCTION

I have developed a practical and effective method for stabilizing underground mine openings based on in-situ stress analysis. This method eliminates the need for artificial ground support such as anchor bolting, wood work, and concrete lining in various types of mines. Instead, it utilizes the natural ground materials to do the job of withstanding earth pressure as well as providing a protective lining to the mine. Since this method is dependent upon the fundamental rock properties of elasticity and non-elasticity, including brittle fracture and clay seam creep, it can be applied to various types of underground excavations. The application with some modifications can be extended to hard rock and solution mining. This paper deals only with the conventional method of mining bedded deposits.

PROBLEMS

First, in order to see the nature of roof failure problems in a bedded deposit, some photographs from Saskatchewan potash mines are shown in Fig.1. The initial trouble-free appearance of a freshly cut opening is often misleading in determining the future state of the opening. It may remain stable for a long time, 5 to 10 years, or it may start failing within a week. The failure is a time-dependent process dictated by the specific ground conditions in each site. Once the roof starts to come down, no conventional artificial support system can prevent its eventual massive failure. The planes of the ground separation continue to rise deeper into the roof mass. The pattern of failure is determined mainly by the three basic rock mechanics factors; namely, stress field, rock properties and geometry of the opening. A typical example of massive roof failure is shown in Fig. 1a. No anchor bolt was found to be effective in preventing these massive roof failures. The best it can do is to hold the fractured roof mass a little longer before its eventual fall just as other types of support, as shown in Fig. 1b. The problems of massive roof failure in bedded formations may be summarized in Fig. 2a. This shows the five general stress zones usually found around a failing underground opening. They are brittle fracture, yielded, plastic, elastic and stress relieved zones. In harder rocks, the plastic and yielded zones are replaced by ground fractured with various degrees of intensity ranging from microscopic to macroscopic fractures. At auy rate, it is impossible to describe the behaviour of the opening by any numerical means such as the finite element method or a continous mechanics solution because the individual stress states require a number of constitutive equations, even with idealized conditions

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of isotropy and homogeneity. Yet in the actual ground, a large number of different stress states, each of which require a different set of constitutive equations, develop according to strain rate tensor states.

Students of the finite element technique may not be willing to accept this conclusion as many of them believe in the technique for solving any and all underground stability problems. In the past, numerous finite element solutions for underground openings have been worked out by various researchers including the author. Unfortunately all the available solutions for predicting roof failure are not any better than the judgment of an experienced miner who is familiar with the geological variations of the underground.

In order to illustrate this difficulty in a more quantitative manner, the same problem is described in the stress spaces shown in Fig. 2b. All probable stress states of rock type materials are identified in this diagram. In solving practical mine problems involving time-dependent deformations caused by fractures, yield deformations and subsequent stress state changes, the three-dimensional time-dependent tensor coefficients of rock materials must be determined as a function of stress and strain rate tensors. The minimum number of such tensor coefficients for a given type of rock is twelve and possibly many more for finer description of the material behaviours. This means that accurate determination of a large number of the tensor coefficient values is the prerequisite for any useful application of the finite element technique to the problems of mining in bedded deposits (1). Of course, thousands of finite element solutions for a mine opening behaviour can be easily obtained by assuming the tensor coefficients. Until recently, however, meaningful assumptions were not possible because there was no reliable method for determining the tensor coefficients. A new tensor testing technique is currently being developed by the author and his associate (2). Fig. 2b shows that there are many zones of stress and strain rate tensor states where the time-dependent tensor coefficients change as a function of time and strain rate. In short the major difficulty we face today in solving mine roof failure problems is not computing finite element networks but determining the true time-dependent tensor coefficients of rock type materials.

LABORATORY STUDIES

A more practical approach to the problem of non-elastic structures is the model study using the same ground materials. The rock material itself is used for the model rather than the commonly used materials of plastic sheet and model rock. By coating the surface of a large block specimen with birefringent film, the behaviors of an opening drilled in the material can be observed as shown in Fig. 3a. In this way, you can eliminate the difficult and time-consuming work of establishing the constitutive equations completely. Figure 3b shows clearly what would happen if you made a wide room in rock salt. Shown in Fig. 3c is the effect of multiple openings drilled close together but with a certain sequence of delay time under various loadings.

I found however that these model cavities produce up to 1,000 times greater creep closure of the openings compared to a comparable prototype in the underground. The reason for this discrepancy is the unrestricted creep flow in the exposed direction. To eliminate this undesirable effect, the model is partially confined as shown in Fig. 4a. This reduced the flow but did not stop it. Then, a whole specimen was enclosed as shown in Fig. 4b. Still it failed to simulate the underground condition even with the simplest form of cavity; a circular borehole. An excellent correlation was finally established when the specimen was loaded triaxially, then stabilized by prolonged creep loading under accurately regulated pressures in the three principal stress directions before drilling the test cavities.

The test hole is drilled through one of the thick loading plates in the back of the ATT(*) machine as shown in Fig. 4c. Although I succeeded in the simulation of a borehole behaviour in relation to three dimensional stress and strain conditions by this machine, I was still unable to see the overall behaviour patterns of the ground surrounding the opening. No information can be obtained by this method regarding the effects of discontinuities which may exist around an opening. (* Ref. No.2)

FIELD INSTRUMENTATION

This was the time for my studies to move from the laboratory to the field. There, I could measure the actual deformation behaviours of the rock media surrounding the underground openings by using a large number of anchor bolts. An actual underground test site is shown in Fig. 5a. Establishing a test site like this requires a considerable amount of time and effort and therefore is difficult to do with the desirable frequency and number. This problem was overcome by introducing a portable electronic instrument, the Serata Microcreep Meter, shown in Fig. 5b. It is capable of determining both the external and internal creep rate distribution patterns around mine openings in a matter of 15 minutes to a few hours. The creep patterns thus obtained are very informative as to how the ground deforms. But it fails to show the stress state which is the driving force of the deformation. The creep pattern is something like an image of the stress condition because there is no direct relation between stress state and creep state. In order to determine the stress state in the ground which does not conform to the ideal theory of elasticity, I developed a Borehole Stressmeter which can be used effectively in non-elastic ground. Actually most rocks were found to be uon-elastic when observed very carefully. The details of the theory and design of the Stressmeter have already been described (3). Contrary to all other stress determining methods, this non-elastic method is simple and fast without requiring anything except the drilling of a single test hole as shown in 5c. The latest model of the Stressmeter shown in this photograph is miniaturized in size and power requirement so that it no longer requires a carload of electronic equipment to operate.

STRESS CONTROL METHOD

With complete knowledge of both the stress and strain conditions in the underground obtained by the method described above, I developed the Stress Control Method for stabilizing underground mine openings as illustrated in Figs. 6a, b, and c. Two prestress openings are excavated, each of which forms an envelope of increased stress (primary stress envelope) as shown in Fig. 6a. These two openings are placed sufficiently close together so that the ground between these two openings is also subjected to additional compression for strain hardening. Then through the strain hardened ground, a third opening is excavated to form a protected room as shown in Fig. 6b. The two pillars thus created are made to yield quickly at a predetermined rate so that the three preliminary stress envelopes suddenly transform into one single envelope. This resultant stress envelope is formed in such a particular shape and intensity so that it can stand against not only the existing earth pressures but also the anticipated increase of the pressure due to future extraction. Inside of the protective envelope lies a large mass of stabilized ground which acts as a protective lining. This protective zone is designed to absorb the future increase of the ground stress.

One of the most critical parts of this design is the time-dependent change of stress in the immediate roof around the protected room. The horizontal stress here should be controlled so that excess tension development does not occur. Otherwise a deep tension crack may develop along the contreline of the roof over the protected room. There are four design parameters for the quantitative control of the final stress condition. They are the Prestress Ratio $(L_1 + L_2)/s$, Yield Pillar Ratio Y/H, Time Delay Δt, and Stress Transfer Rate $\Delta P/P_o$, as illustrated in Fig. 6c. P_o is the initial total overburden load acting across the entire entry and ΔP is the difference between the initial overburden load and the final vertical load remaining over the entire multi-opening entry.

The effectiveness of the Stress Control Method was proven by extensive in-situ stress measurements using the Stressmeter. The measurements reflect the change in the stress envelopes in relation to the excavation and time, as the theory predicts. More specifically, the speed of the stress distribution pattern is dictated by the tensor coefficients of the ground materials. In both yield and abutment pillars, the vertical stresses are always greater than the lateral stresses by an amount set by the octahedral shearing strength of the pillar material. Over the roofs of the openings, the critical lateral stresses decrease gradually with time, approaching a certain asymptote. In the Saskatchewan potash deposit, the natural lateral stresses were found to be quite often greater than the overburden pressure.

The effectiveness of the protective stress envelope is demonstrated in the substantial reduction of the creep closure rate in the protected room as shown in Fig. 7a. Without stress control, the closure rate taken at the room centre was over 2,000 μ-in/hr. The closure rate taken across the stress controlled room is substantially smaller as compared in the creep distribution curves of the figure. The stability of the stress controlled room is noted in the following observations: (1) the closure rate is significantly reduced, (2) swelling at the room centre is completely eliminated, (3) room closure rate is the same as the deformation rate of the yield pillar, (4) the two prestress openings experience much less creep closure than the uncontrolled single room. In comparing these two test sites which have greatly different extraction rates, you find one interesting lesson; that is, extraction rate is not necessarily a meaningful criterion for underground safety. These creep rate distribution patterns can be obtained by the Microcreep Meter in an observation ranging from 15 minutes to 1 hour, as illustrated in the actual recorder paper, with 6-channel operation, also shown in Fig. 7a.

An even more interesting comparison between the two test entries was made when the internal creep rate distribution patterns of their roof media were examined, as shown in Fig. 7b. The data was taken by the same Microcreep Meter. A number of probes of the Meter are self-anchored quickly in ordinary anchor bolt holes as shown in the diagram. The sharp peaks of the uncontrolled roof indicate the clay separation with different velocities at various depths. The first clay seam at 2.5 ft. is separating at a rate up to 1,000 times faster in the uncontrolled roof compared to the controlled one. The scale of the creep rate is given in logarithms. This difference in the roof separation rate is substantial; comparable to the difference between the speed of a supersonic transport at 1,500 miles per hour, and the speed of man's walk at 2 miles per hour.

APPLICATION

I have just described above the basic principle of the stress control method for which there are many applications in various types of mining. Many variations of mine layout developed by the stress control method principle have been proven useful in various potash mines where conventional methods of roof support were found to be unworkable. Some of them have abandoned the conventional mehtod and adopted the stress control method, resulting in dramatic improvement of the mine openings. Variations of the basic stress control layout were found to be necessary in order to adapt the stress control method to the particular excavation and haulage equipment available in the mines. Two examples of specific application of the Stress Control Method are illustrated below. Below the potash deposits of Saskatchewan at the depth of 3000-4000 ft., a large underground storage silo can be excavated safely and economically if the Stress Control Method is applied as shown in Fig. 8a. Prestress openings are made at the top and bottom of the silo. The dimension of a stress-controlled silo can be greater than 50 ft. in width, 150 ft. in depth and almost unlimited in length. An underground storage space of this type and size has obvious advantages for an economical mining operation.

For total extraction by a drilling and blasting method in a shallow bedded deposit, the Stress Control Method is applied in a different manner. One example is shown in Fig. 8b. The yield pillar is reduced to nothing by the proper control of delay time and room width depending upon the tensor properties of the ground materials. In the example shown in Fig. 8b, more than one stress controlled entries are advanced in parallel which provides a sufficient number of working fronts. The ground between the entries is removed in retreat to transform the three secondary stress envelopes into a single tertiary stress envelope. This final protective envelope is designed to form within the competent rock formations below the high pressure aquifer for minimizing the flood potential. In addition, no appreciable surface subsidance is allowed as the tertiary envelope is kept sufficiently below the soft soil formation. The plan of the design is shown in Fig. 8c. The development entries are stress-controlled inside the framework of the abutment pillars. Total extraction by drilling and blasting may be applied in the ground surrounded by the stress-controlled entry systems. In this manner, a high extraction rate can be maintained in the process of mine development, without the major expense and danger of supporting falling roofs. After completing the developmental work, a total extraction retreat can be made without the risk and difficulties of robbing small pillars. The final retreat will naturally cause surface subsidance as the major abutment pillars are removed. However, this retreat process makes the wide protective envelope to yield in compression, which minimizes the creation of a sudden shear fracture through the overburden. Consequently, it substantially reduces the surface disturbance and flood potential in total extraction mining.

CONCLUSION

In conclusion, the Serata Stress Control Method is a new concept of mining based on a quantitative analysis of in-situ stress and strain. The usefulness of this method has been proven in Saskatchewan potash mines. Application to other types of mines is now being conducted. The advantages of the Stress Control Method are, namely; (1) elimination of roof failure and floor heave, (2) elimination of the expense of anchor bolting, roof scaling, and other roof support measures, (3) major improvement of mine safety, (4) substantial increase of extraction rate, and (5) improvement of production efficiency by wider room widths.

I am greatly indebted to the following five engineers who assisted me in conducting the field work; Ken Pettigrew of Bunker Hill, formerly with Allan Potash, Don McKinlay of Allan Potaeh, Terry Wowk of Duval, David McIntosh of Cominco, and Julian Kopchynski of Sylvite.

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Figure 1a. Massive roof failure of Saskatchewan potash mine threatening safety and economical operation of mine.

Figure 1b. Failure of all artificial support measures by steadily progressing roof fall.

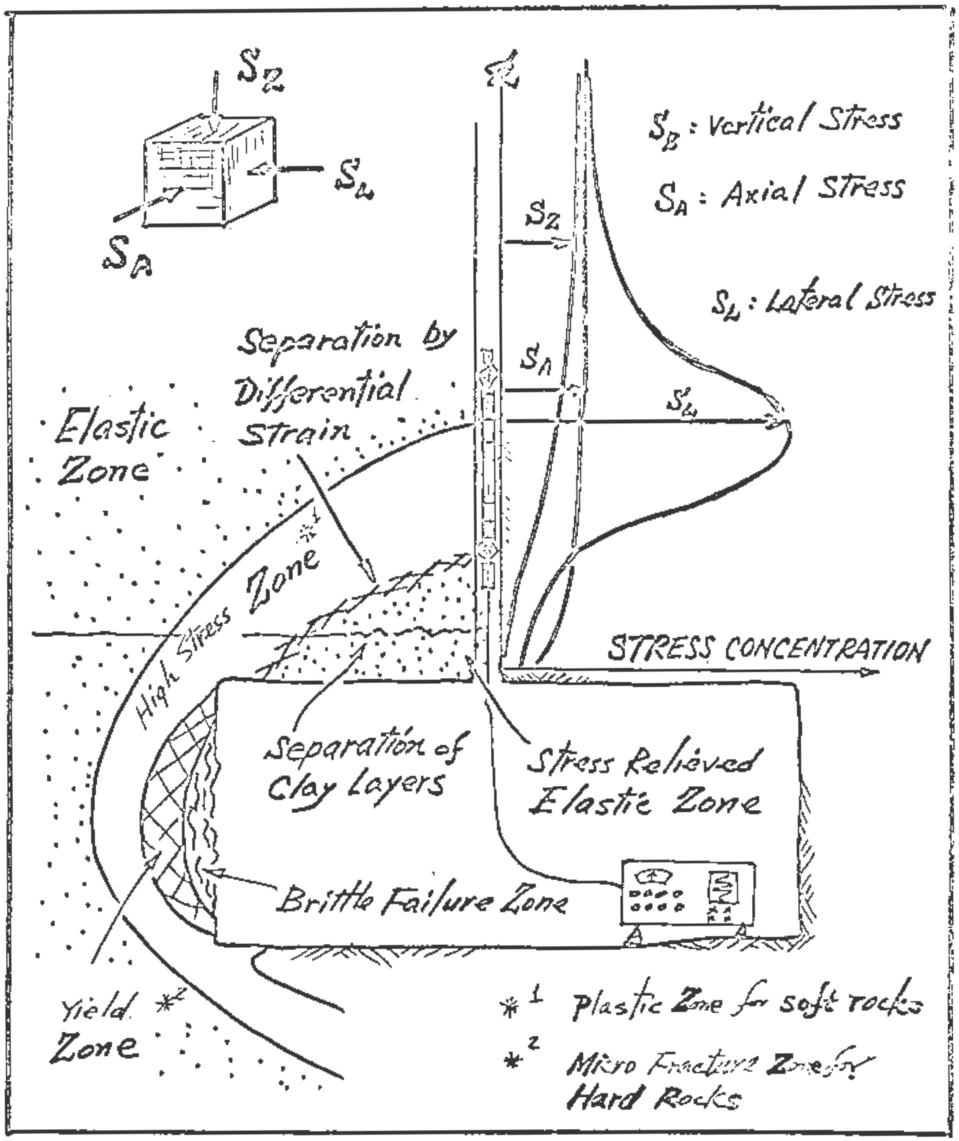


Figure 2a. General pattern of roof failure in bedded formations.

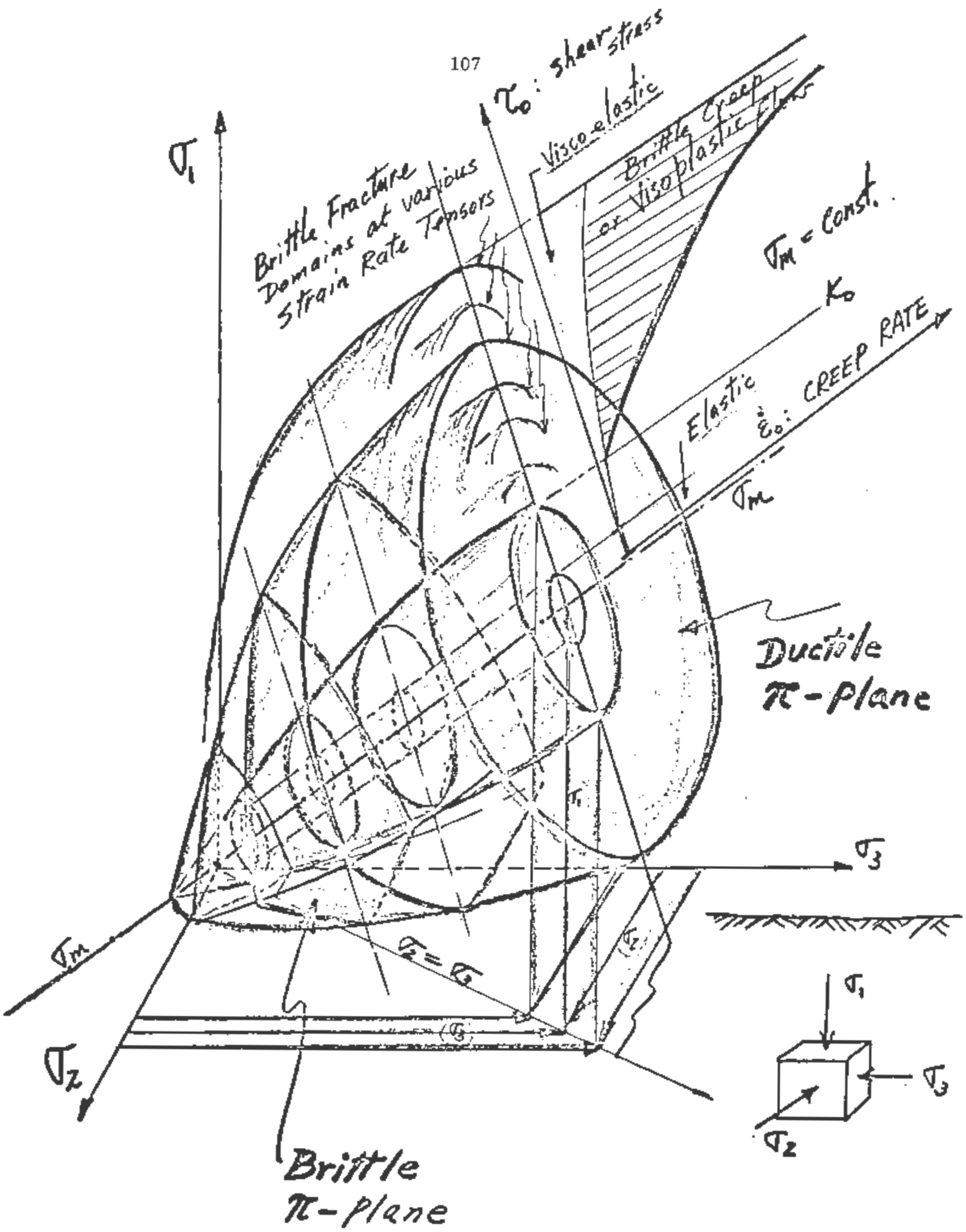
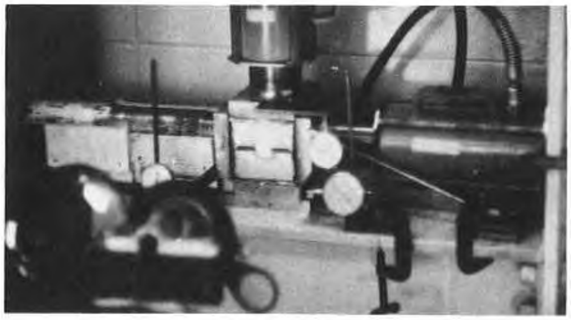
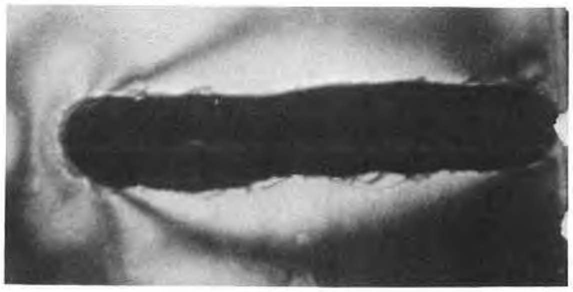


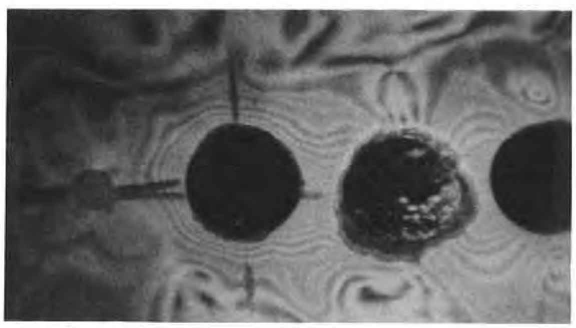
Figure 2b. General tensor behaviours of rocks.



(a) Mutual effect of double openings



(b) Effect of widening the width of openings



(c) Delay-time effect among multiple openings

Figure 3. Behaviours of openings made in rock mass observed by birefringent film coated over exposed surface under biaxial loading.

(a) Partial confinement

(b) Complete confinement

(c) Complete control of stress and strain tensors

Figure 4. Three dimensional model cavity analysis under various degrees of control on boundary conditions.

(a) Conventional creep station consisting of many anchor bolts measured by dial gauges

(b) Serata microcrup meter capable of quick measurement of creep rate distribution pattern

(c) Serata stressmeter capable of quick measurement of stress distribution pattern in single borehole

Figure 5. Development of underground instrumentation for measurement of stress-strain conditions and determination of structural safety.

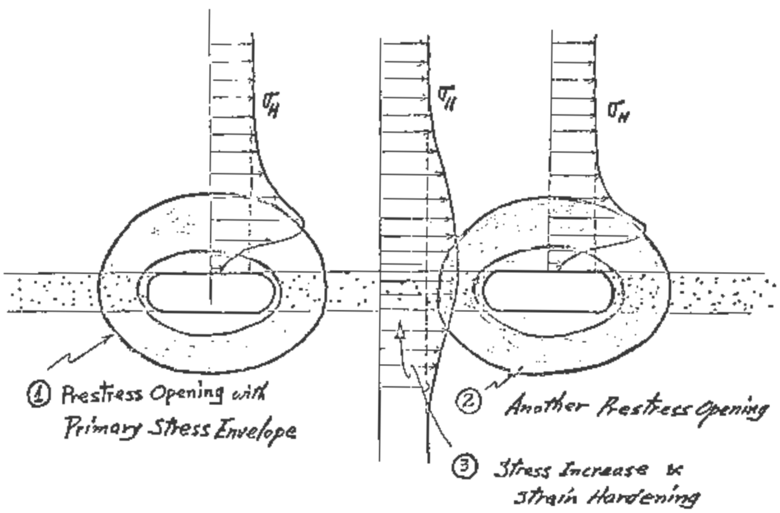


Figure 6a. Stress control method. (Step 1)

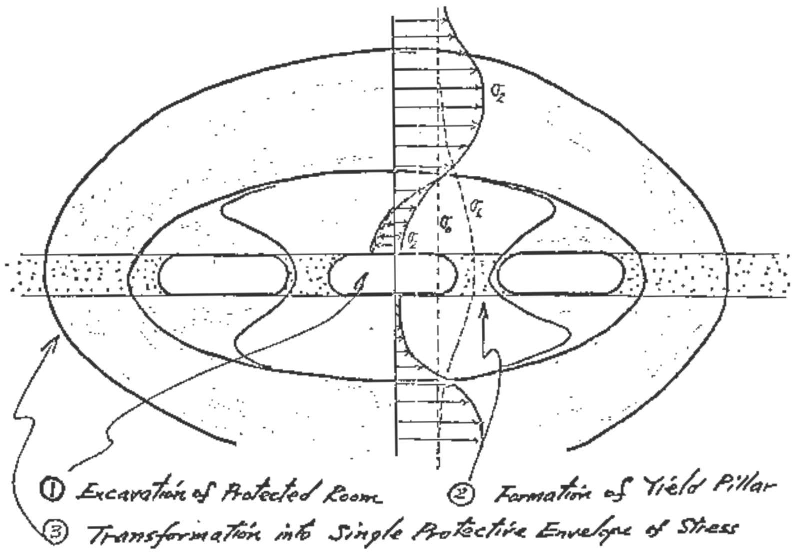
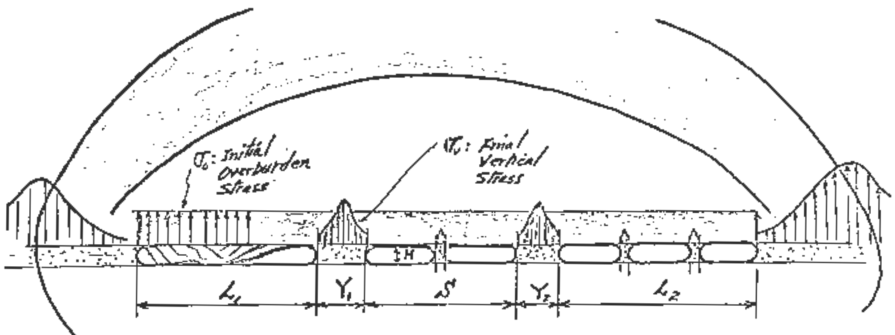


Figure 6b. Stress control method. (Step 2)



Quantitative Control of Design Parameters

① Regulation of Prastress Ratio ; $(L_1+L_2)/S$

② Yield Pillar Ratio : Y/H

③ Time Dalay : Δt

④ Stress Transfer Rate : $\Delta P/P_0$

Figure 6c. Stress control method. (Step 3)

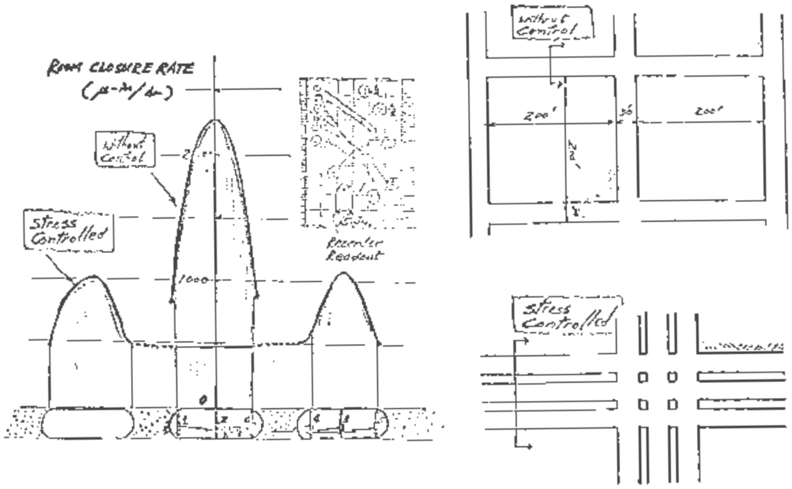


Figure 7a. Effectiveness of stress control method shown by room closure rate distribution.

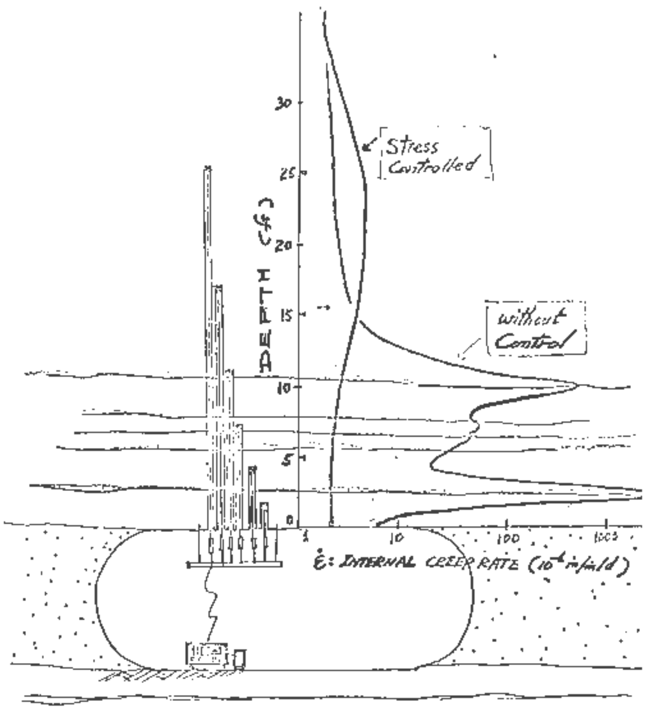


Figure 7b. Effectiveness of stress control shown by internal creep rate distribution.

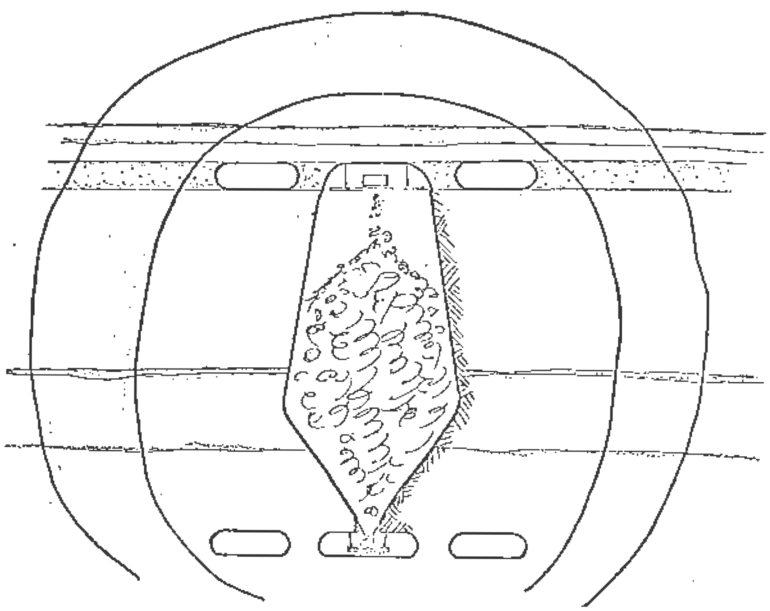


Figure 8a. Stress control for large underground silo.

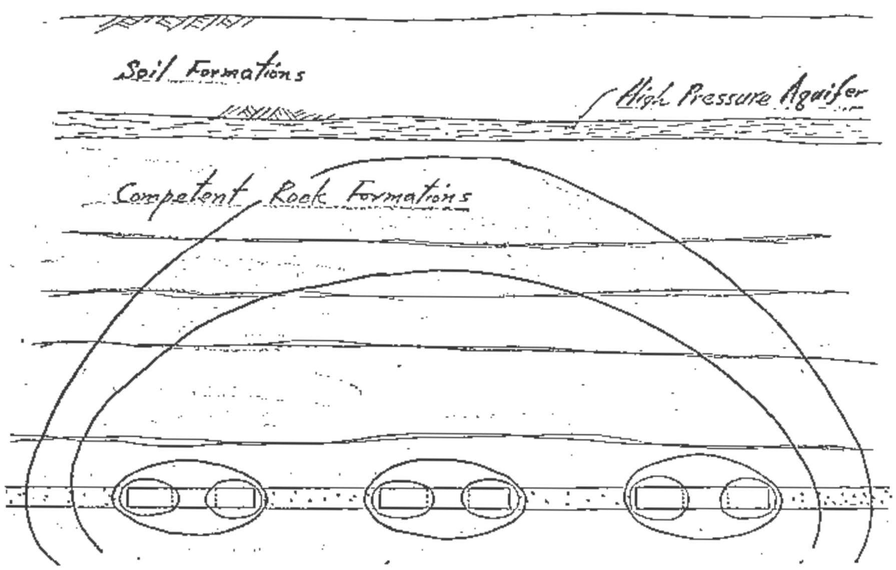


Figure 8b. Stress control for drilling blasting operation.

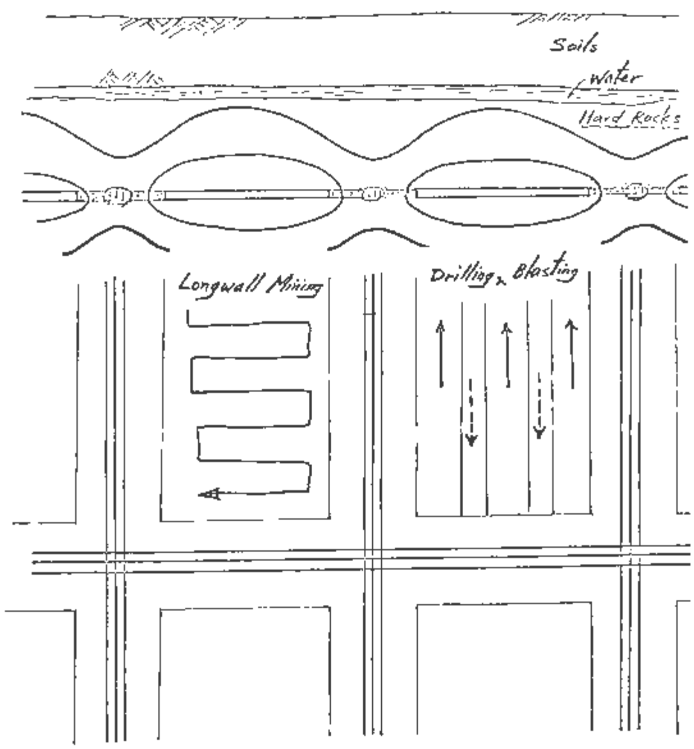


Figure 8c. Stress control for total extraction.

The Modification of Operating Dimensions within an Existing Gypsum - Anhydrite Mine

I. Weir-Jones*

ABSTRACT

Laboratory and 'in situ' studies of some of the mechanical properties of a gypsum-anhydrite deposit form the basis for suggesting profitability of the operation.

INTRODUCTION

Bord and pillar mining methods in stratified deposits offer the mine operator a low cost primary mining technique. The secondary procedure, pillar recovery, is, however, frequently expensive and it is, therefore, desirable to remove as much mineral as possible during the primary operation. Unfortunately, the maximum permissible extraction rate is invariably limited by pillar and roof stability, excessive extraction normally giving rise to unsafe, and hence uneconomical, working conditions.

The engineer who seeks to optimize the dimensions of an array of pillars and roadways must, therefore, achieve a compromise between the necessity of maintaining safe working conditions and the requirement to extract as much mineral as possible during the primary mining operation withing a given area. This paper concludes that, in the majority of cases, laboratory investigations must be extended by means of 'in situ' investigations in order to provide meaningful design data (1) (2) (3).

CONDITIONS PREVAILING AT THE MINING SITE

The two adjacent mines in which these investigations were made are situated in North West England. They have been in production for about 30 years and, during this time, workings producing both gypsum and anhydrite have been developed on two horizons over an area of approximately 1.5 square miles.

The mineral deposits are of Upper Permian age and they occur spasmodically within the beds known as the St. Bees Shales. These shales also contain the extensive anhydrite deposits which are mined in the Whitehaven regiou of Cumberland, and both deposits are almost certainly contemporaneous.

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The fact that mining takes place on two horizons has already been mentioned. The upper horizon is composed of a fairly discrete deposit of gypsum and anhydrite averaging about 12 ft in thichness, with up to 8 ft being the anhydrite layer. The deposit is surrounded by a considerable thickness of low grade gypsiferous marls and has, in the past, produced the bulk of the production, but it is now virtually exhausted. Production is now concentrated in the lower horizon, a deposit which is approximately 100 ft below the upper one and up to 120 ft thick. Most of this deposit is composed of low grade gypsiferous marls, and the portion which is mined as the lower horizon is seldom more than 20 ft thick, although, on occasion, up to 30 ft has been removed. The position of the economic zone in the lower deposit is extremely variable, hence the mining horizon is moved up and down within the deposit in order to produce material of consistent quality.

In general, the mines are favourably situated. There are no unstable overlying deposits, no problems exist regarding the support or drainage of the surface, the mining company is a major landowner, and no extraordinary conditions were imposed by the Mines Inspectorate. However, certain problems do exist which affect the design of the workings -

1) The topography of the land above the mines is extremely irregular, hence, even though the gypsum deposit dips gently to the west at about 10°, the thickness of the superincumbent strata varies from 150 ft to approximately 800 ft in slightly over half a mile. In view of the resultant variation in the vertical stress component, it would be reasonable for pillar dimensions to vary from place to place.

2) The working height in the lower deposit can alter quite appreciably within few yards. Variations in pillar height of 50 per cent within a distance of 250 ft are not uncommon, hence, with a constant pillar cross section throughout the mine, there is a considerable variation in pillar W/H ratio.

3) The material making up the mining horizon is completely heterogeneous, ranging from a soft, marly material to tough, amorphous gypsum, or to marls interspersed with lenses of satinspar, an example of which is shown (Figure 1). The whole deposit is a mass of thin bands of one type gypsiferous material or another, and the occurrence (or non-occurrence) of one type of material has apparently little effect on the position of the economic zone, and thus upon the relative position of the working face within the deposit.

4) The fact that the deposit is made up of a succession of thin bands means back stability is an ever present problem. There is always the likelihood that the roof layer will commence sloughing, and that this will continne until a stable arch is found. The net effect of this would be to introduce considerable tonnages of lower grade material into the production system.

Any proposed modification of the mining method wonld have to take into account the following factors:

a) The mines would continue to be worked by partial extraction, using the bord and pillar system.

b) Throughout the mines the overburden varies greatly and, consequently, so do the inherent pillar loads.

c) The height of the working face varies from place to place it cannot be kept constant because of economic considerations. Hence, the W/H ratio of the pillars is likely to vary considerably.

The three factors mentioned above are concerned with the dimensional characteristics of the mining layout. In view of this, as the permissible working dimensions are going to be limited by the mechanical properties of the material surrounding the opening, it is clearly important to obtain some relevant values for these properties.

LABORATORY EXPERIMENTAL WORK

The importance of obtaining relevant data concerning the mechanical properties of the gypsum has already been noted. Determination of the mechanical characteristics of the material from the deposit could be made in the laboratory or 'in situ'. Obviously, the laboratory approach would be much more convenient, a great deal cheaper, more rapid, and would lend itslef to careful control. However, in the of this gypsum, it was felt that the 'size effect' (3) (4) would have a marked influence upon results obtained under laboratory conditions, but hopefully these preliminary laboratory results would provide a basis upon which subsequent 'in situ' work could be developed. Consequently, it was decided that the laboratory phase of the programme would examine the following properties:

1) Uniaxial and triaxial compressive strength
2) Tensile strength
3) Elastic properties
4) Creep characteristics

In the shallower portions of the mines, it was most unlikely that the compressive strength of the ghpsum would impose limits upon the extraction rate achieved. However, mining would ultimately extend to a depth of 800 ft and conceivably mean pillar stresses in excess of 3,000 psi could be induced.

About 300 cylindrical specimens of different diameters and W/H ratios were subjected to various uniaxial and triaxial compressive tests. The results indicated that, for a constant W/H ratio, the U.C.S. was extremely sensitive to specimen volume (Figure 2), the following expression being derived:

$$S = 13{,}660\ V^{-0.396} \qquad \ldots(1)$$

Furthermore, the influence of radial constraint upon U.C.S. was marked, as indicated by the U.C.S. against W/H ratio curves (Figure 3). This was reassuring as it implied that pillars with a W/H ratio in excess of unity would most probably be capable of supporting a load considerably in excess of that predicted by Expression (1). The conslusion was borne out by the results of triaxial testing using specimens 3 ins. in diameter with a W/H ratio of 0.5. When the constraining pressure was 2,000 psi, the compressive strength was 5,200 psi, compared to the mean uniasial compressive strength of 3,140 psi.

Roof stability above the roadways was known to be a potential problem. However, the heterogeneous nature of the deposit made it most unlikely that applicable values for the tensile strength could be obtained. Nevertheless, almost 400 indirect tensile strength determinations were made, the results completely justifying the initial assumptions about the heterogeneity of the material. The results are presented as distribution curves (Figure 4) and an attempt has been made to make them more meaningful by dividing the samples into three categories, based upon their physical structure. However, this has done little to increase the relevance of the data, as the table below demonstrates.

Category	Description	Number of Specimens	Tensile Strength psi	Standard Deviation psi
Rock Type A	amorphous gypsum	136	352	145
Rock Type B	crystalline gypsum	105	156	79
Rock Type C	crystalline gypsum in amorphous matrix	155	291	110

On the basis of these results, bearing in mind that the categorization was, at best, arbitrary, it was clear that laboratory tensile strength determinations were going to yield very little information which would be of use for designing the mine roadways.

Cylindrical specimens with a W/H ratio of 1.0 exhibited negligible deformations when held at a constant compressive strength of 2,500 psi. In view of this, it was concluded that pillar creep would not present a significant problem in the mines.

The elastic constants of cylindrical specimens of various diameters with W/H ratios of 0.5 and 1.0 were determined. The elastic moduli were found to be sensitive to specimen volume and to specimen cycling - the value of the chord modulus between 200 psi and 2,200 psi increased from 1.36×10^6 psi to 2.23×10^6 psi in the course of six cycles. The effect of specimen volume upon the chord modulus between 200 psi - 2,200 psi for the sixth cycle is shown (Figure 5). The curve is similar to the U.C.S./volume curve (Figure 2), but not as steep. Presumably, a limiting value would ultimately be reached, and it was suggested that this would probably be between 0.5 and 1.0×10^6 psi.

The practical relevance of the results of the laboratory tests in this situation is debatable, e.g., the basing of actual design recommendations concerning optimum roadway widths upon the results of the tensile testing program would be most unwise.

Similarly, although the results of the compressive testing indicate where the probable value of the compressive strength of a pillar-sized specimen will lie, this information is not an adequate basis for actual pillar design. Consequently, in this situation, 'in situ' measurements of the mechanical properties using specimens of a representative size would be of the utmost value.

'IN SITU' EXPERIMENTAL WORK

The importance of 'in situ' testing in this situation has already been stressed. The mechanical properties for which relevant values would have to be obtained were the compressive and tensile strengths, and the elastic constants. In addition, it was expected that should creep occur, it would be measured. Furthermore, it was hoped to make an evaluation of the rigidity of the overlying strata.

A portion of the deposit was designated as the experimental site. Coring had indicated that the economic zone was 15 ft thick and it commenced at a depth of 200 ft. Furthermore, in this area, not only was the thickness of the economic zone constant, but so was its depth below the level surface. The site having been chosen, preparatory work commenced. It was proposed to block out a series of pillars by driving 10 ft wide roadways on 50 ft centres. These roadways would then be progressively widened by pillar slicing on one side of the roadway and the behaviour of the pillars and roof would be studied by means of borehole extensometers. At the same time, geodetic levelling at the surface would yield information about overall subsidence and general pillar deformation.

The intention was firstly to widen the roadways until the maximum stable width could be extablished. Once this had been done, pillar slicing would continue, until the horizontal and vertical pillar deformation measurements suggested that a critical stage had been reached. These measurements would also yield representative values for E and

Thus, the experimental work was intended to yield quantitative data concerning the load-bearing capacity of the seam material, the optimum roadway width, and the elastic properties of the pillars. Hopefully, the latter would aid the establishment of optimum roadway widths in other parts of the mines - previous workers having shown that the elastic characteristics of mine pillars exert a considerable influence upon the stresses induced above the workings (5). However, in this situation, pillar rigidity played an insignificant part in the question of back stability.

Two basic types of instrumentation techniques were used, constant tension borehole extensometers (6), and precise levelling. The boreholes were both vertical and horizontal, four of the vertical holes passing through the projected pillar centres, and two others through the proposed roadway intersections. Whatever the centres, and two others through the proposed roadway intersections. Whatever the position of the vertical boreholes, however, they all passed completely through the economic zone and terminated in the limestone some 15 - 20 ft below the zone.

Much of the core from these holes was used for laboratory testing and the tensile strength results obtained from one of the cores passing through what was to be an intersection are shown (Figure 6).

A regular precise levelling grid was laid out on the surface above the experimental site. An additional series of stations led away to a reference station several hundred yards north, well away from the possible zone of influence of the mining operation.

The vertical boreholes were instrumented and initial measurements made when the advancing roadways were about 400 ft south of the site. At the same time, the preliminary levelling was carried out. Measurements of both types were continued at weekly intervals, whilst the mining advanced and, when the boundaries of the experimental site were reached, the roadways, which were being driveu 16 ft wide on 50 ft centres, were reduced to 10 ft, the centre distance being maintained at 50 ft, but the centreline of the narrow roadways being offset 2 ft, i.e., an extraction rate of 36 per cent. The initial intention was to have an experimental site six pillars square, i.e., 300 ft sides. Practical considerations, however, made it necessary to reduce this to an area five pillars square. Once the roadways had penetrated the area and the pillars were blocked out, horizontal boreholes were put through the five centre pillars and equipped with screw-type extensometer anchor. Once this was completed, it was possible to measure accurately the transverse pillar deformation as the load was increased by the slicing operation.

RESULTS OF THE 'IN SITU' MEASUREMENTS.

The instrumentation work having been completed, roadway widening was initiated by the removal of a 2 ft thick web on the western and northern faces of the pillars, the slicing being restricted to these faces so that the extensometer stations on the opposite sides of the pillars were not disturbed. After each slicing operation was completed, subsidence and pillar deformation measurements were taken twice-weekly, until the strain rates became zero - at this point, an additional slice was removed. The successive deformations measured between the top and bottom of one of the pillars are shown (Figure 7). The other curves in this figure show the anchor 30 ft above the pillar, and the surface subsiding as slicing progresses. The total deformation of the surface and uppermost anchor is consistently about .020 inches less than the pillar deformation, suggesting that some relaxation occurred between the mining zone and the top anchor early in the course of the investigations. Clearly, however, this bed separation did not increase. This conclusion is borne out by the fact that, after week 30, the onset and termination of movement is all points in the borehole is virtually simultaneous. At week 10, surface subsidence is caused by the approaching mining, and at week 15, the highest anchor shows signs of the arrival of the subsidence basin.

Similar pillar deformation patterns were displayed by the results obtained from the horizontal boreholes. The rate of deformation was marked immediately after slicing, but it decreased at about the same rate as the vertical deformation curve and the onset of zero deformation occurred almost simultaneously (Figure 8).

When the roadways had been widened to 16 ft, it was noted that, although fracturing could be observed in the roof at several roadway intersections, no significant failures had occurred. However, the widening from 16 ft to 18 ft caused several areas of roof to collapse and precautionary scaling indicated that thr roof conditons were rapidly becoming undesirable. It was, thus, apparent that the existing mine roadway width of 16 ft, which had presumably been established as ideal by empirical methods, could not be improved upon, and thus would have to be maintained as the maximum during mining. Pillar slicing continued 5 ft at a time until the pillars were 15 ft wide and the roadways 35 ft. At this stage, the roofs of the roadways had all sloughed to a natural arch. The extraction rate at this point was nominally 91 per cent and the mean pillar load 2,230 psi and, although fairly considerable pillar deformations had taken place, average .68 ins., the extensometer measurements indicated that the four instrumented pillars were still deforming elastically. It should be noted that the procedure of permitting the roadway backs to assume a stable arch was unacceptable from a grade control position - this was unfortunate, as the arched roof gave every indication of being quite stable.

As a result of the vertical pillar deformation measurements, values for the 'in situ' chord modulus between 200 psi and 2,200 psi were obtained. The values were found to be approximately 500 x 10^3 psi, compared to a laboratory value of 2.23×10^6 psi. This suggested that either the modulus was quite sensitive to sample volume, or that the central portions of the pillars were carrying a load considerably in excess of the mean. The actual situation was probably a combination of these factors. The latter obviously played some part, as non-uniform deformations were measured along the horizontal boreholes passing through the pillars (Figure 8).

The onset of unstable roof conditions when the roadway exceeded a width of 16 ft show some correlation with the results of indirect tensile tests conducted in the laboratory. The first roof layer to slough was approximately 6 ins. thick and vertical features first became apparent when the roadways were about 15 ft wide, indicating a tensile strength of about 225 psi.

The results obtained by testing discs cut from the core indicated that the indirect tensile strength of the layer was 270 psi (Figure 6). However, the wide variation in the indirect tensile strength of adjacent layers indicates that an attempt to base specific span widths on the tensile strength of individual roof layers would be a most questionable procedure.

CONCLUSIONS.

Roof stability observations suggest that acceptable conditons may be maintained in roadways up to 16 ft wide by means of bolting with 5 ft bolts and screening where necessary. Greater roadway widths would necessitate either the formation of a stable arch, with consequential grade control problems, or the use of large numbers of long roof bolts. The limitation of roadway width to 16 ft is considered to be the economic and practical solution.

The 'in situ' experimental work showed that pillars 15 ft square and 15 ft high, i.e., a W/H ratio of 1.0, were stable when the mean compressive stress was 2,230 psi. The laboratory testing of specimens of varying W/H ratio indicated that the rate of increase in U.C.S. decreased rapidly when the W/H ratio of 1.0 (Figure 3). Hence, it was felt that, providing a pillar W/H ratio of 1.0 could be maintained, a mean pillar compressive stress of 2,200 psi was acceptable. This meant that, where the mining depth was 800 ft, the rate of extractiou would be approximately 64 per cent, i.e., a 16 ft wide roadway and a 24 ft square pillar. In those places where the economic zone thickness was 20 ft, the pillar W/H ratio would be 1.2, and hence there was no reason to suppose that the pillars would be unstable.

It was clear that the existing mining pattern, using 16 ft wide roadways on 50 ft centres, was much too conservative. The entire mine was being developed with an extraction rate of 54 per cent. The results of the pillar studies suggested that, down to a depth of 550 ft, an extraction rate of 75 per cent could be safely maintained, providing the economic zone thickness, and thus the pillar height, did not exceed 16 ft. From 550 ft, down to the deepest portion of the mine, the extraction rate was to be reduced to 64 per cent, with the same pillar W/H provisos applying.

Assuming that mineral reserves were equally distributed at all depths between 250 ft and 800 ft, this meant that the average rate of extraction would be approximately 69 per cent. This recommended average extraction rate represents a 28 per cent increase in output for a given portion of the deposit, with no additional increase in mining costs and, on this basis, the initial expenditure on the laboratory and 'in situ' instrumentation is seen to be more than justified.

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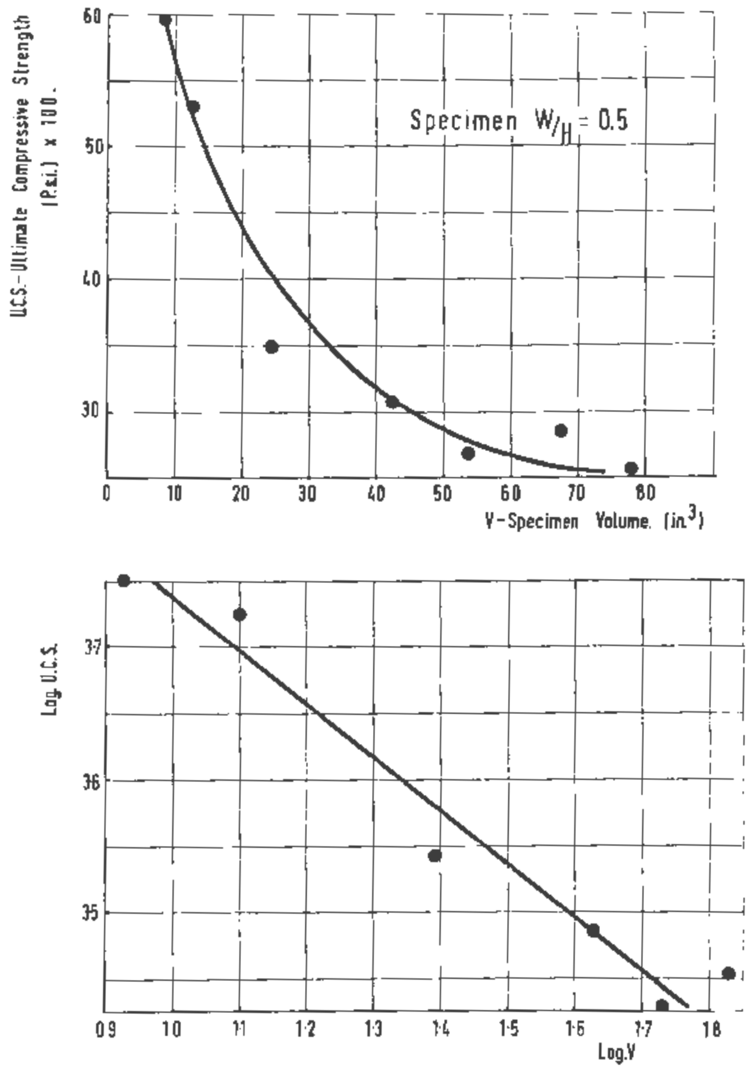
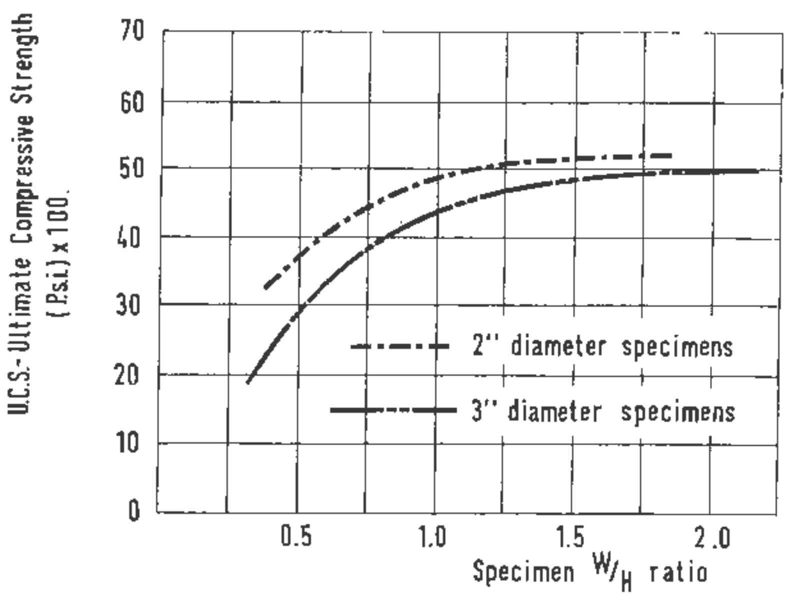
Figure 1: Typical laminated structure in the mining horizon. The satinspar lenses are approximately 2 ins. thick.



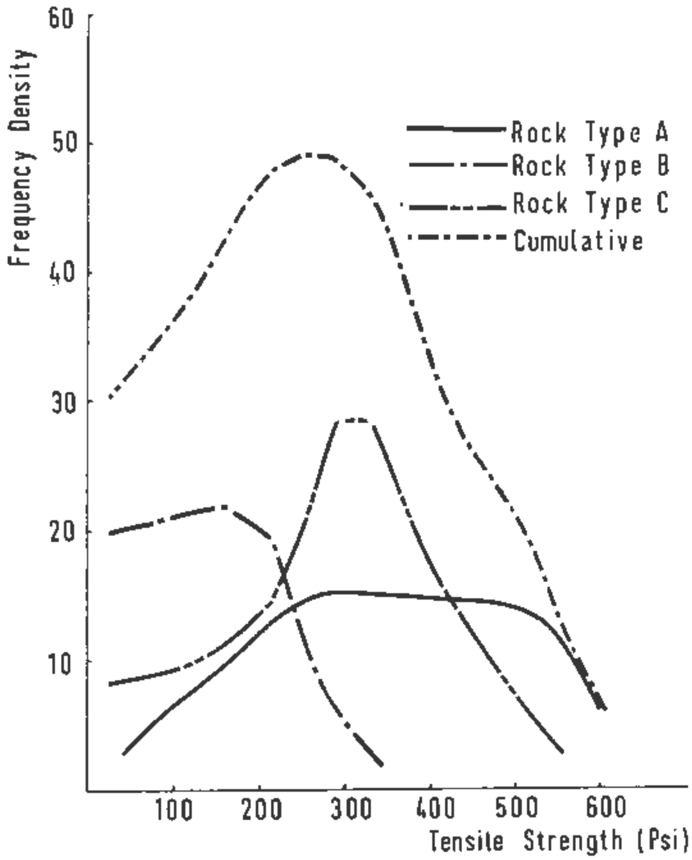
VARIATION OF COMPRESSIVE STRENGTH OF GYPSUM AS SPECIMEN VOLUME INCREASES

Figure 2



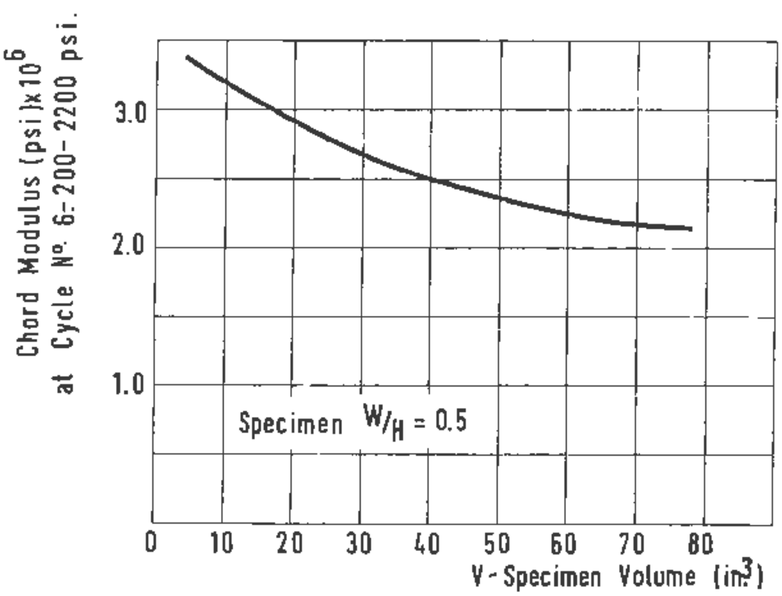
EFFECT UPON COMPRESSIVE STRENGTH OF VARIATIONS IN SPECIMEN W/H RATIO

Figure 3



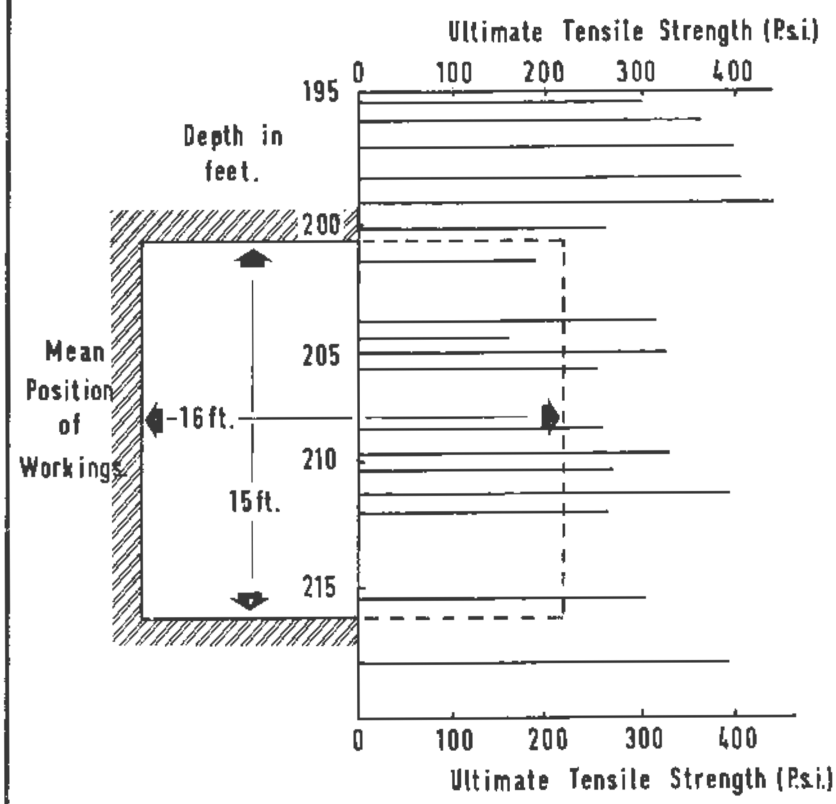
DISTRIBUTION CURVES OF TENSILE STRENGTH RESULTS FROM DISC SPECIMENS

Figure 4



VARIATION IN CHORD MODULUS AS SPECIMEN VOLUME INCREASES

Figure 5



TENSILE STRENGTH VARIATION AROUND A ROADWAY INTERSECTION AT THE EXPERIMENTAL SITE

Figure 6

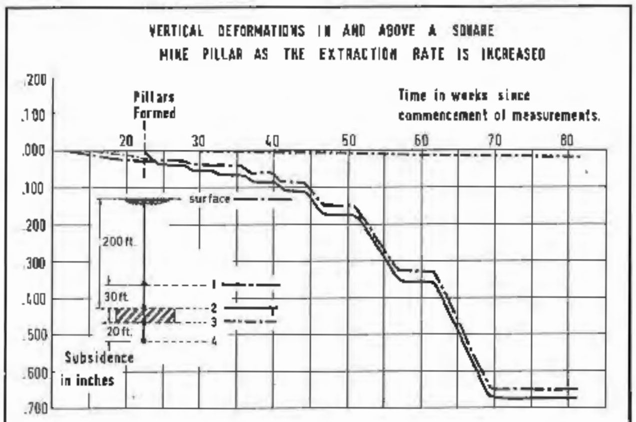


Figure 7

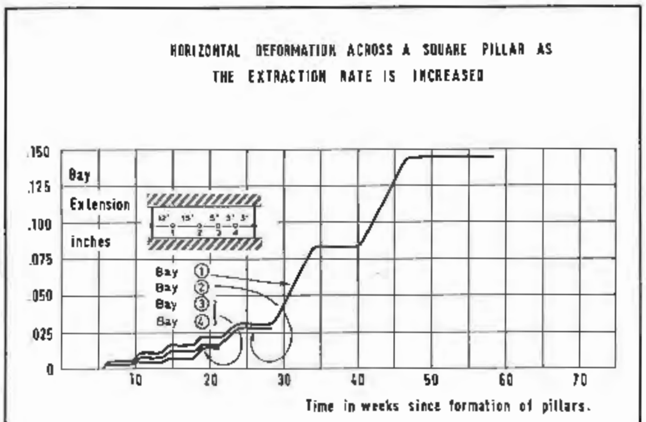


Figure 8

A Case Study of In Situ Rock Deformation Behaviour in the Silver Summit Mine, Coeur D'Alene Mining District

Samuel S. M. Chan* and Thomas J. Crocker**

ABSTRACT

Mining operations in the Coeur d'Alene district, presently entering deep horizons, will doubtless penetrate deeper. Consequently, maintaining safe underground openings through proper ground support becomes paramount.

The U.S. Bureau of Mines, Spokane Mining Research Center is currently developing a method to predict artificial support requirements based on a minimum of in-situ data. This paper discusses a portion of a continuing Bureau-sponsored project with field studies conducted at the Silver Summit mine in the Coeur d'Alene mining district. It emphasizes basic study of the in-situ deformation behaviour of rock medium around an underground opening as influenced by structural geology, in-situ stress measurement using the overcore stress-relief method, and laboratory physical property testing. Results obtained indicate that the geological structures do affect both the amount and rate of rock deformation which occurs during and following excavation.

INTRODUCTION

As stated at the recent advisory Conference on Tunnelling (37, p.4)[1], "Current practice for determining ground support requirements is, in many cases, only an educated guess. Better methods are needed to evaluate in-situ properties of rock and to determine their significance with respect to engineering design and to tunnel construction. Also, there is a need to develop rationale for the design of ground support based on the understanding of the nature of the interaction between the support and the ground."

Research performed in the past by the Spokane Mining Research Center (7) which consisted of periodically monitoring closure between opposing vertical and horizontal points (installed along mine accessways driven parallel to a steep vein, indicates that (1) opening closure over a 3-year period could have been predicted (based on the initial 3-month sample period); (2) the additional stress concentration caused by stoping increased the deformation rate in proportion to the previous rate (that is, divergence of deformation-rate curves

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1 Underlined numbers in parentheses refer to items in the list of references at the end of this report.

was accentuated in the same relative magnitude of order as indicated by the sample); and (3) based on these results, support-problem areas could be foreseen.

Usefulness of ground-deformation records in evaluating support need has also been acknowledged by others. Schwartz (38) conducted a study of coal-mine roadways and found the initial convergence rate later obtained for the 33 weeks of the test. He states that this information was useful in the selection of artificial support and also reduced costs.

The objective of the work described herein is to assist in establishing relationships between the rate of rock deformations around newly created excavations and the ground support requirements. The data presented were obtained from a case study conducted in the Silver Summit and consist of rock deformation records of the medium surrounding an underground opening during and following the excavation process, and results from in-situ stress measurements and laboratory rock physical properties tests. Because of their preliminary nature, the significance of the data relative to the stated objective cannot be ascertained; however, the information is presented to show the influence of various geological settings and the effects of the installation of artificial supports on the deformation records.

ACKNOWLEDGMENTS

The authors are indebted to the U.S. Bureau of Mines, Hecla Mining Co., and the University of Idaho, Bureau of Mining Research for their support, which made this investigation possible.

Appreciation is expressed to Messrs. NORMAN GRAHAM, PAUL E.HYDE and M.W. HUTCHINSON of the Hecla Mining Co., who have given the authors valuable technical assistance and pertinent information, and to TIMOTHY HACKETT of the Spokane Mining Research Center, who contributed in various phases of this project.

THE SILVER SUMMIT MINE

The Silver Summit mine is located about 1 mile west of Osburn, Shoshone County, Idaho (Figure 1). It lies approximately in the middle part of what is locally called the "Silver Belt" of the Coeur d'Alene district (35, p.1). Operated by the Hecla Mining Co., the mine is mainly in the development stage at present. The main entrance is the Silver Summit tunnel, located approximately at 2,640 feet above sea level; whereas the lowest working drift is the 4,000 level, which is about 1,336 feet below sea level. Predominant minerals, as in most of the Silver Belt deposits, are silver bearing tetrahedrite, galena, and some chalcopyrite.

GEOLOGY

Regional Geology

Rock of this region consist of three major groups: (1) Precambrian metamorphosed sediments of the Belt Supergroup, (2) Cretaceous and Tertiary igneous rocks, and (3) unconsolidated sediments of Tertiary to Quaternary age (8-9, 13, 16, 30).

Rocks in the "Silver Belt" are principally the Beltian, which can be divided into six formations: Striped Peak, Wallace, St. Regis, Revett, Burke, and Prichard. The rocks are mainly quartzite, argillites, and transitions. Regionally metamorphosed, they are in the greenschist facies (13).

Structurally, the Coeur d'Alene district has been sheared and faulted into a complex structural pattern described as a "structural knot" (16). Nevertheless, the major faults, as shown in Figure 2, conform to a general pattern, striking roughly northwest and dipping steeply to the south. Starting from the north, they are the Osburn, Polaris, Big Creek, and Placer Creek.

The most persistent fold is the Big Creek anticline, situated between the Polaris and the Big Creek faults and offset by the Silver Summit fault. The axis of the Big Creek anticline is approximately parallel to the Big Creek fault and dips about 70° to the south (35). Most of the silver-lead-zinc-copper deposits of the Silver Belt occur in the fracture zones of the north limb of the Big Creek anticline, particularly near the large faults.

Local Geology

Rocks in the Big Creek anticline are placed in the Wallace, St. Regis, and Revett formations. The Wallace formation covers most of the surface, but survives only as a strip between the Chester and Polaris faults at the 4000 level. The St. Regis, comprising most of the rocks down to the 4000 level, is mostly argillites at the top of the formation, becoming more quartzitic at the base. Consisting mostly of quartzites, the Revett formation is visible at the surface, north of the Polaris fault, and continues in that position down at least to the 5000 level. However--see Figure 2-- Revett rocks now appear at the 4000 level (19-20) at and near the axis of the Big Creek anticline.

HUTCHINSON (19) indicates that the important structures in the Silver Summit mine area are, beginning at the north, the Polaris fault, Chester vein-fault, Silver Summit fault, Transverse fault, and Silver Summit No.1 vein-fault. (See Figures 2 and 3.) The Big Creek anticline lies mostly to the south of the mine workings, but has left-lateral displacement along the Silver Summit fault, so that it lies both to the north and to the south of the 4000 southeast lateral at its present position. Other than the Big Creek anticline, very little folding has been observed in the mine area. The axis of the anticline lies to the south of the lateral drift, but it is left laterally displaced along the Silver Summit fault so that it again appears to the north, bracketing the mine workings.

Most of the rocks in the 4000 east lateral belong to the lower St. Regis formation (19, 20). Generally, rocks in and near the measurement stations (noted as SST-1, SST-2, SS-1 through SS-7 in fig. 4) consisted of medium to thick-bedded, fine-grained, greenish-gray, argillaceous quartzite, often interbedded with thin layers of very fine-grained, greenish quartzitic argillite and argillite. Lithologically, the rock at each station, as shown in Figures 5 through 7, can be divided into three rough categories: (1) hard, competent, fine-grained, thick-bedded quartzites, (2) comparatively incompetent thin interbeds of quartzite, quartzitic argillities, and argillites that are designated "Transition" rocks, and (3) argillites.

DEFORMATION MEASUREMENT

In-situ deformation or closure measurement is one of the most basic yet useful methods for investigation of in-situ deformation behavior of rock masses. In this method the total displacement between two certain points opposite to each other around the drift is measured. This total displacement includes the initial, short-term elastic deformation, the long-term elastic deformation (creep), and the expansion of the natural and induced fractures. The data from this type of measurement can be used as input into the finite element program to calculate the modulus of deformation of the rock masses and can be used as basic criteria for the design of ground support systems.

Nine closure measurement stations were installed along the east lateral of the 4000 level. Lithologic and structural conditions, as well as the work schedule, dictated the localities of the measurement stations. For reasons noted below, a portion of the east lateral was selected as the test site:

1. The lateral was being drifted at the time of our field investigation.
2. Because the drift intersects the beddings of the rock formation at a small angle, significant deformation was therefore measurable.
3. Since the drifting advanced into a virgin ground, minimum influence (if any) would be expected from other underground openings.

Measure Procedures

For each station the methodology included the installation of six specially designed rock bolts into the medium surrounding the drift. (Figure 8 shows the ideal orientation of the measuring points.) Then a mechanical type extensometer was used periodically to measure the distance between two opposite points such as A-A', B-B', and C-C'. D-D' was discarded because of technical difficulties which resulted from covering of the measurement point in the floor by the rail track.

In this project a station was installed as close as possible to the face--within 12 inches--to monitor as much as possible the deformation associated with excavation. Initial readings were taken approximately 1 hour after installation. Immediately after the removal of the blasted rocks, a second set of readings were recorded, followed by several sets of readings at hourly intervals. The time intervals of measurement was extended to days, and then weeks as the drift face was advanced. In this manner, the short-term initial deformation as well as the long-term deformation of the rock medium surrounding the drift was determined.

The measuring points, shown in Figure 9, were installed in the following manner:

1. Holes 1-3/8 inches in diameter and 16 inches in length were drilled into the rock at the designated positions by means of a jumbo.
2. A bag of Rock-Loc[2] epoxy was mixed and placed into the hole.
3. A specially made rock bolt was then inserted into the hole. The bolt was made of carbon steel, 12 inches long, 3/4 inch in diameter, with two ends threaded, having a 1-3/8-inch, two-way

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2 Reference to specific trade names is made to facilitate understanding and does not imply endorsement by the Bureau of Mines.

expansion shell attached to one end and a double-nut head to the other.
4. The bolt was tightened by a socket wrench. The epoxy mix was therefore filled into the anchorage portion of the bolt and consolidated, presuming that there was no significant creep.

The extensometer used was designed by the Spokane Mining Research Center. It is a stainless steel, telescopic, spring-action type, with a range from 8 to 12 feet. A separate caliper ranging from 0 to 6 inches, with 0.0001 inch as the smallest scale division, was used to record the distance between the two measuring pins near the centre of the device. Actual distance was obtained by summing the distance defined on the telescopic tube and the caliper readings.

EXPERIMENTAL DATA

Data obtained from the nine deformation measurement stations are expressed in terms of drift closure, in respect both to the time of measurement and to the corresponding distance between the measurement station and the advancing face. Tabulation of the data is presented in Appendix I.

Opening deformation was plotted against time for selected stations (Figures 10-12). Regardless of the individual variations among these curves, they reveal a general trend, from which a generalized deformation curve--such as the one shown in Figure 13--may be made. In this curve, section I (from point 0 to point A) represents the short-term, elastic deformation; section II (from point A to B) is the initial time-dependent portion; section III (upper curve, beyond point B) illustrates the hypothetical, long-term time-dependent deformation portion. Section III exists if the rock masses were left in place without artificial support. Section IV (lower curve, beyond point B) is the continuing creep curve that occurs when rock bolting in and near the station was completed.

Table 1 contains information calculated from data in Appendix I. From this table and Figures 10-12, the following interpretations may be delineated:
1. Without exception, the highest total deformation as well as the highest maximum elastic deformation occurs at the mid-point position of the drift (line B-B), excluding direction D-D' for which data were not available.
2. Maximum elastic deformation in the B-B' direction (0° orientation) is about three to four times that obtained in the A-A' direction (45° orientation) for station SS-1, SS-4, and SS-7, but it is high as eight to nine times that of stations SS-2 and SS-3, which are located in less competent quartzitic argillities. This condition indicates a lower in-situ modulus of deformation in the argillites, assuming the loads to be equally applied to the rock masses to begin with.
3. A rock burst, observed by the author, with a relief of elastic strain energy estimated at 4.5 x 10^6 ft.-lb. occurred in an area 120 feet from station SST-2, and 135 feet from SST-1 at 849.5 hours. The complete cycle of strain buildup, relief, and reaccumulation was recorded (Figure 10a).
4. Stations SST-1 and SST-2 are located 15 feet apart in competent quartzite without different structure. Nevertheless, the ratio of their maximum elastic deformation is 6 to 1, and the ratio of their

total deformation is over 3 to 1. The different initial distance to the drift face might have caused the deviation. Perhaps the variations in the effect from blasting have also contributed to the difference.

5. Maximum elastic deformations of stations SST-5 and SST-6 are both 0.05 inch in the B-B' direction. When this figure is compared on the same base with SST-2, SST-3, and SST-4 (which are located in the same type of rock), the following ratios are obtained, respectively: 1/5, 1/7, and 1/9. The fact that the drift and the short lateral interesect the beddings at different angles (Figure 6) together with the effect of blasting is the principal cause of deviations.
6. Besides station SST-1, station SST-4 has the highest total deformation. The location of station SST-4 near the intersection may be a prime factor.
7. Station SS-1 is very near a major fault. When it is compared with other stations that are situated in similar rock type but are farther away from the fault zone (such as station SS-3), some differences in elastic deformation and creep rate are observable. Listed below are the data:

Station	Orientation	Elastic deformation	Creep rate	Time to reach creep
SS-1	B-B'	0.200"	3.89×10^{-5} in/hr	200 hrs
SS-3	B-B'	0.360"	1.8×10^{-5} in/hr	50 hrs
Ratio of SS-1/SS-3:		0.55	2.16	4

8. A pair of 6-foot rock bolts were installed in station SS-1 just 12 inches below regular points B-B'. A comparison shows that the elastic deformation of the rock in the B-B' area is four times that of the area under the rock bolts. Their total deformation also maintains a similar ratio. Their creep rates do not deviate from each other significantly. However, the rocks under the 12-inch bolt area required 200 hours to reach creep, but only 50 hours for those that are under the 6-foot rock bolt. It also shows that the 12-inch bolts used in this investigation were affected by the blast fractures, whereas the regular 6-foot rock bolts were less affected and perhaps not affected at all. A comparison between the initial elastic deformation in the rock masses within the zone of influence of the 6-foot rock bolts and that of the mine rocks tested in the laboratory under similar applied loads (e.g. 8,000 psi to 15,000 psi) may help to derive the actual short-term, maximum, elastic deformation of the rock masses, in spite of the presence of the blasting and natural fractures.

   The resutl of the comprison of the total deformation (or displacement, counting the creeping and blasting effects) between the rock masses in the 12-inch bolts and that of the 6-foot bolts have outlined the scope of the effect of blasting that could be used as criteria in determining the depth of the "active" fracturing zone in which support is needed. The effective length of the rock bolt can then be accurately chosen. Future research should also include 4-foot bolts for this comparison purpose.
9. In general, the thick-bedded quartzite has elastic deformation approximately three times that of the thin beds of argillite or argillaceous quartzite. This indicates that the argillites

TABLE I

Deformation Behaviors of Rock Masses in the East Lateral of the 4000 Level, Silver Summit Mine (Calculated by Tim Hackett)

Sta.	Orient.	Bedding & Rock Type	Max. Elastic Deformation Inch	Creep Rate ($\Delta d/T$) In/Hr	Reach Creep Hours
SST-1	0°(BB')	Thickly bedded, relatively pure quartzide	∿0.600	5.1×10^{-5}	∿700
SST-2	0°(BB')	Same as SST-1	∿0.100	5.5×10^{-5}	∿700
SS-1	0°(RKB)	Thin to medium-bedded argillaceous quartzite to quartzitic argillite	∿0.050	2.56×10^{-5}	∿ 50
	0°(BB')		∿0.200	3.89×10^{-5}	∿200
	45°(CC')		∿0.050		∿ 50
SS-2	45°(AA')	Thin to medium-bedded argillaceous quartzite to quartzitic argillite	∿0.030	7.98×10^{-5}	∿ 10
	0°(BB')		∿0.240	1.22×10^{-5}	∿ 60
SS-3	45°(AA')	Thin-bedded quartzitic argillite and argillite	∿0.040	4.61×10^{-6}	∿ 20
	0°(BB')		∿0.360	1.8×10^{-5}	∿ 50
SS-4	45°(AA')	Medium to thin-bedded argillaceous quartzite, quartzitic argillite	∿0.150	1.365×10^{-5}	∿300

yielded plastically long before the quartzite did, and that the quartzite might have been assuming most of the load in an interbedded condition. Because of its buildup of elastic-strain energy, rock bursts might therefore more likely be initiated in quartzite when the critical condition is reached.

10. Creep rate is lower in the thin beds of argillite. The thick-bedded quartzite required 400 to 600 hours to reach the creeping stage, whereas creep in the thinly bedded argillite ranged from 50 to about 200 hours. This indicates that by the time the points of the measurement stations were installed, a portion of the deformation as well as the relaxtion of the fracutured zone had already taken place in the thin-bedded argillite, and that the more competent quartzite therefore took up most of the load and deformed more. This confirms the statement mentioned in the above paragraph (point 9).

ROCK PROPERTY DETERMINATION

Testing techniques are confined to conventional laboratory methods; therefore, the detailed procedures are omitted. Since this phase of the research is still in progress, only representative data are shown. The authors are cognizant of differences between laboratory and in-situ data, in determination of rock strength and its elastic modulus. Data obtained from this phase of the investigation will therefore be used as a supplement, rather than as direct substitution.

Compressive and Tensile Strength

Diamond-drill core samples from the 4000 level of the Silver Summit were used for all rock testing mentioned herein. Being the closest diamond drill hole to the over-coring site during the investigation, DDH SS-40-05 was the source of core samples.

For uniaxial compressive strength testing, all samples were sawed to an approximate diameter to length ratio of 1:2; average diameter was about 1.4 inches, and average length was approximately 3.0 inches. A surface grinder was employed to maintain parallelism of the sample ends to an accuracy of 10 minutes. A Soiltest Model CT-714 compression machine with a load rate control was used for compressive loading. Strain indicator readings were taken at 5,000-pound load intervals.

To date 16 argillaceous St. Regis quartzite and quartzite samples with various grain sizes and structures have been tested. Their uniaxial compressive strength ranges from 9,300 psi to 40,400 psi, depending on their argillite content, grain size, texture, fracturing system, and bedding orientation (see Table 4). This range suggests that a great number of tests and statistical data processing are necessary to ascertain actual rock strengths. Figure 14 indicates the stress-strain curves of a homogeneous, fine-grained competent quartzite, as well as its uniaxial compressive strength.

To obtain some basic parameters of rock strength, 10 competent samples, with approximately a 1:1 diameter-to-length ratio, have been tested by the Brazilian method. Tensile strengths of these slightly argillaceous quartzites range from 2,307 psi to 5,124 psi. Table 2 illustrates some relations between tensile strength and structural, mineralogical conditions of rocks.

TABLE II

Tensile Strength of Some St. Regis Quartzites from the East Lateral of the 4000 Level, Silver Summit Mine

Sample Number	Rock Type	Structural Condition	Bedding Orientation	Tensile Strength (psi)
1. SS-40-05-516(2)	Argillaceous quartzite with 40% sulfides and siderite	Thinly bedded, sharp boundaries	45°	3,408
2. SS-40-05-62 (2)	Quartzite, slightly argillaceous	Fine grained	80°	3,612
3. SS-40-05-668(2)	Quartzite	Very fine grained	70°	5,124
4. SS-40-05-670(3)	Quartzite	Fine grained distinct beddings	50°	4,503
5. SS-40-05-729	Quartzite, slightly argillaceous	Minute fractures existed	50°	3,455
6. SS-40-05-823(2)	Quartzite	Very fine grained	60°	4,442
7. SS-40-05-852(2)	Quartzite, slightly argillaceous	Fine grained	80°	3,635
8. SS-40-05-897(2)	Quartzite, slightly argillaceous	Fine grained	70°	3,845
9. SS-40-05-976(2)	Quartzite	Fine grained, several fractures existed	50°	2,307
10. SS-30-05-1168(2)	Quartzite	Fine grained	45°	3,582

Mean Tensile Strength...3,791 (psi)
Standard Deviation.... 726 (psi)
Percent Deviation..... 19%

Triaxial Testing

To simulate confining conditions of the rock masses with regard to bedding orientations and major planes of weakness and to discover, based on deformation characteristics, a meaningful correlation between laboratory and in-situ data, triaxial testing was conducted.

Sample dimension and preparation for the triaxial testing duplicated uniaxial tests. Conventional procedure and equipment for triaxial loading tests were used (1, p.16). Four samples of argillaceous quartzites with 45° beddings (with respect to the axis of the diamond drill hole) were tested under various confining stresses. The results are shown in Table 3. The internal friction angle derived from these tests is 50°, and the initial shear is low.

Lateral Deformations and Poisson's Ratio

Since thus far only a few tests have been made, it is premature to derive any definitive data concerning the lateral deformation and Poisson's ratio. However, for one sample, Poisson's ratio was calculated to be 0.13, and for another sample it was 0.21. Both axial and lateral deformations of the rock appear to be linear beyond about 5,000 psi, which indicates the presence of microfractures and voids regardless of the dense composition of the rocks.

Modulus of Elasticity

Sixteen samples of argillaceous quartzite and quartzite were tested uniaxially for compressive strength and moduli of elasticity. Figure 14 shows results which indicated that both modulus of elasticity and uniaxial compressive strengths increase with the decrease of argillite contents, fractures, planes of weakness, and grain size. As shown in Table 4, the compressive strength ranges from 12,500 psi to 40,500 psi, and the modulus of elasticity ranges from 2.14×10^6 psi to 13.0×10^6 psi. It is therefore meaningless to show the average figures.

IN-SITU STRESS DETERMINATION

For in-situ stress determination, the conventional stress-relief overcoring method was used. The equipment, provided by SMRC, consists of the following:

1. A hydraulic drive diamond drill with a 7-hp air motor that attained rated horsepower at about 1,500 rpm.
2. An EX bit and core barrel.
3. A 6-inch masonry bit with core barrel.
4. A three-component borehole deformation gauge of the U.S. Bureau of Mines, and table.
5. Three Budd Model P-350 strain indicators.
6. A set of gauge insertion devices.

Test Site

The overcoring test site(Figure 5) is located in a crosscut of the east lateral of the 4000 level of the Silver Summit mine, near deformation measurement station SST-1. The centreline of the crosscut intersects almost perpendicularly to the bedding. Rocks near the drill site are medium to thick bedded, slightly argillaceous quartzite of the St. Regis formation. All beds dip nearly vertical. A minor fracture, dipping steeply to the south, occurs near drill holes No.1 and No.2.

TABLE III

Triaxial Testing Data of Some St. Regis Argillaceous Quartzites from the East Lateral of the Silver Summit Mine

Sample Number	Bedding Orientation	Failure Angle	Confining Stress psi	Ultimate Axial Stress psi
SS-40-05-852 (1)	∿45°	75°	1,000	8,100
SS-40-05-943	∿45°	80°	2,000	25,926
SS-40-05-688 (2)	∿45°	80°	3,000	37,175
SS-40-05-688 (3)	∿45°	78°	4,000	47,025

TABLE IV

Uniaxial Testing Data of Some St. Regis Quartzites and Argillaceous Quartzites from the East Lateral of the Level, Silver Summit Mine

Sample Number	Rock Type	Structural Condition	Bedding Orientation	Uniaxial Compressive Strength (psi)	Modulus of Elasticity (psi)
SS-40-05-485 (1)	Quartzite, slightly argillaceous	Fine grained, competent	Homogeneous within a thick bed	15,547	2.14×10^6
SS-40-05-485 (2)	Quartzite, slightly argillaceous	Fine to medium grained, with network type microfractures	Same as above	12,461	2.92×10^6
SS-40-05-500 (1)	Quartzite, slightly argillaceous	Same as above	Very thin beds, 90° to the axis	21,807	4.81×10^6
SS-40-05-540 (1)	Quartzite	Very fine grained	Homogeneous	29,700	13.00×10^6
SS-40-05-540 (2)	Quartzite	Fine grained grained	Homogeneous	28,107	6.30×10^6
SS-40-05-579 (1)	Quartzite	Fine grained	Homogeneous within a thick bed	40,423	7.26×10^6
SS-40-05-808 (1)	Quartzite, slightly argillaceous	Fine grained	Same as above	24,875	4.23×10^6
SS-40-05-1020(1)	Quartzite	Fine grained	Same as above	22,346	8.92×10^6

Test Procudure

The basic procedure for determination of absolute stress (27, pp. 413-417) was modified for the application of the three-component borehole deformation gauge.

For each hole, a 6-inch bit was used to advance 6 feet to 10 feet into the rock in order to be away from the stress concentration zone. An EX-size hole was then drilled approximately 4 feet farther into the rock mass. (The centres of the 6-inch hole and the EX-hole were coincided.) Then the deformation gauge was inserted in the EX-size hole, and the initial readings with the Budd strain indicators were taken. That portion of core containing the gauge was then overcored concentrically with the 6-inch bit until the bit advanced a few inches beyond the position of the gauge. At 1-inch intervals, strain indicator readings were taken. The difference between the initial reading and the others, multiplied by the calibration factor, gave deformation in that direction.

The 6-inch overcore was taken to the laboratory for the determination of its modulus of elasticity biaxially and triaxially (12, 26). The same procedure was repeated until three holes in different directions with different inclinations were completed.

Calculation of In-Situ Stress

Despite some difficult ground conditions, readings were successfully obtained in three holes. A typical overcoring stress-relief deformation pattern is shown in Figure 15.

All data obtained were processed by the Denver Mining Research Center (17). Calculation of in-situ stresses in the overcoring test site is shown in the stress ellipsoid (Figure 16).

If an average specific gravity of the overburden rocks is assumed to be 2.8 gm/cc--or a weight density of 175 lb/ft^3--and a total depth of overburden of 5,475 feet is assumed, the calculated stress passed on overburden is 6,652 psi. Therefore, a difference of 1,576 psi exists between the calculated vertical stress based on overburden and that determined experimentally.

The deviation of the 1,576 psi of the measured vertical ground stress from that of the estimated overburden stress could have been caused by one or more than one of the following factors: (a) the stress concentration that might have existed near the opening, (b) the influence of any possible geotectonic forces nearby, (c) the possible deviation of the weight density of the rock masses from that of the average 2.8 gm/cc, and (d) the actual modulus of deformation of the rock masses probably differs considerably from the one used for the calculation of ground stresses. Detailed descriptions of the other potential cuases for error and suggestions for improvement are, however, mentioned in another report (5).

Combining Figure 5 and Figure 16, it is readily seen that the direction of the major principal stress is approximately the same (with less than 3° difference) as that of the central axis of the overcoring hole No.1. They are both roughly perpendicular to the strike of the beddings. In this manner, the minor principal stress is found to be in parallel with the bedding direction approximately horizontally. The intermediate principal stresses will therefore coincide with the direction of the vertical stress very closely.

The explanation of the high magnitude of the major principal stress, besides the possible existence of the local stress concentration, the deviation in the modulus of elasticity used, and any other experimental error, is the possibility of the influence from the tectonic forces that might still be acting on the Osburn fault and its immediate vicinity since the test site is not too far from the fault (see Figure 1); and the direction of the major principal stress is at a small angle (roughly less than 45°) to that of the strike of the Osburn fault.

SUMMARY AND CONCLUSIONS

This study reveals the following pertinent facts:

1. A study of the long-term deformation curves (tabulated in Appendix I) shows clearly that geological structures do affect both amount and rate of creep deformation. They further illustrate that the installation of rock bolts in many cases reduced the slopes of the deformation-time curves, which in turn may stabilize the drift opening.
2. Rock deformation varies with the change of the angle of intersection between the strike of the rock formations and the long axis of the drift. Higher deformation is expected when the angle is smaller.
3. In an interbedded situation, argillites may yield plastically, which leaves the relatively competent quartzite to accept most of the stresses.
4. In general, both the modulus of elasticity and uniaxial compressive strength of the St. Regis rocks increase with the decrease of argillite contents, fractures, planes of weakness, and grain size.
5. Most of the St. Regis quartzites deformed linearly under uniaxial or triaxial loading conditons.
6. Relatively pure quartzites indicate three times the elastic deformation than that of the argillites and argillaceous quartzite.
7. Minimum calculated stress in the rocks is about 5,400 psi in compression, and is along the direction paralleling the strikes of the rock formations. Maximum stress occurring in the structure is approximately 15,200 psi. When conditions so favour, rock failure may be expected to occur along this direction.

In summary, the practices herein employed of measuring initial deformations of rock masses near and around a drift (as used in this investigation) have proved effective. The application of the overcoring stress-relief method for determining stresses in rock masses, along with the laboratory testing of rock strength and elastic modulus, is also very helpful in delineating useful information for the stabilization of underground openings in the Coeur d'Alenes.

This project is still in progress. Research plans include further laboratory physical-property testing, in-situ determination of modulus of deformation (using the Goodman jack), and the finite-element method to achieve closer approximations in structural analysis.

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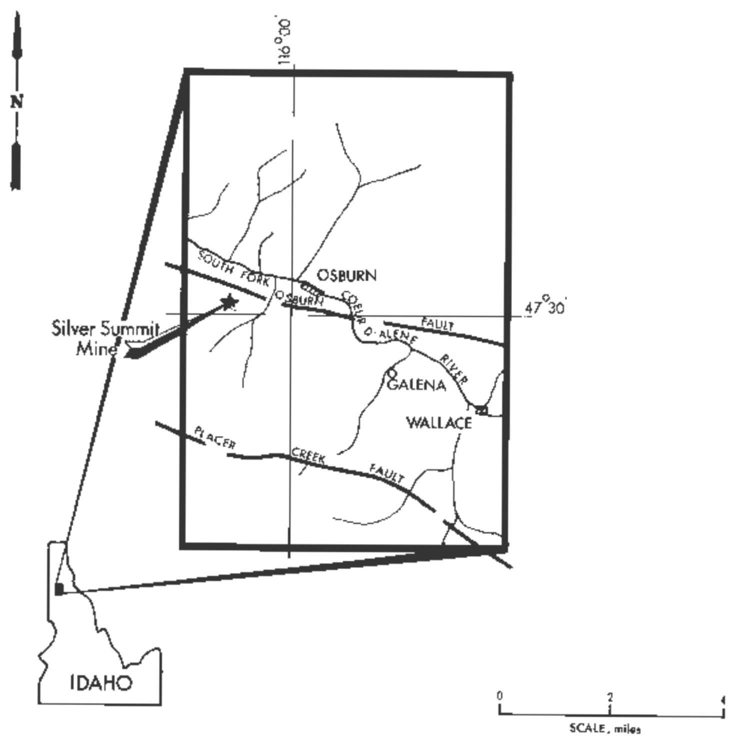

Figure 1. Location map Coeur d'Alene area

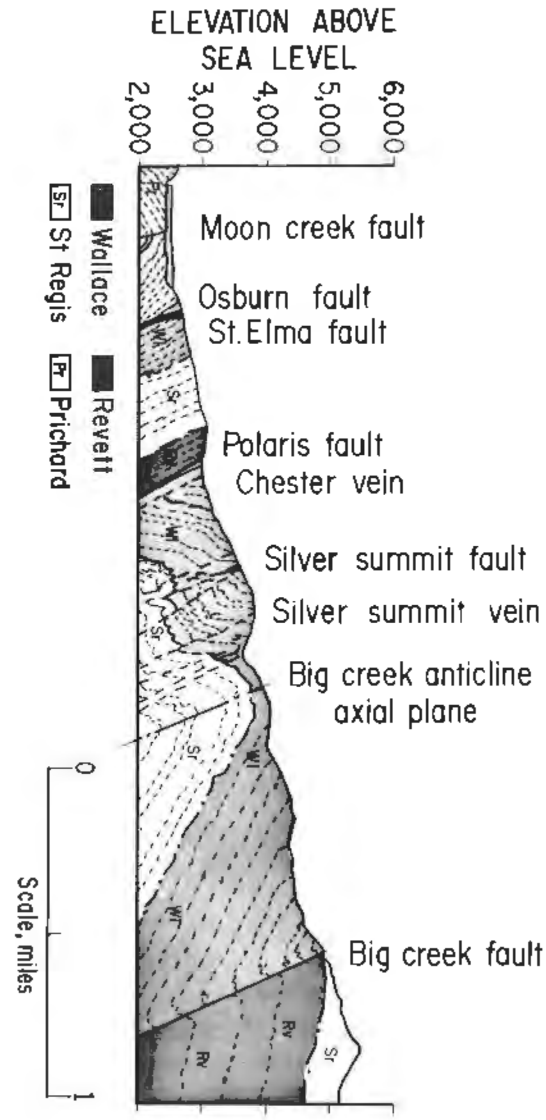

Figure 2. North-south vertical section near Silver Summit mine

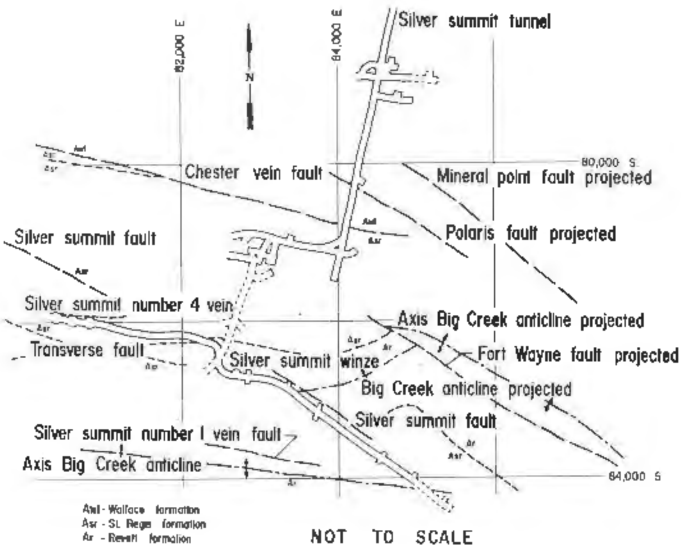

Figure 3. Structural geology, 4,000 level

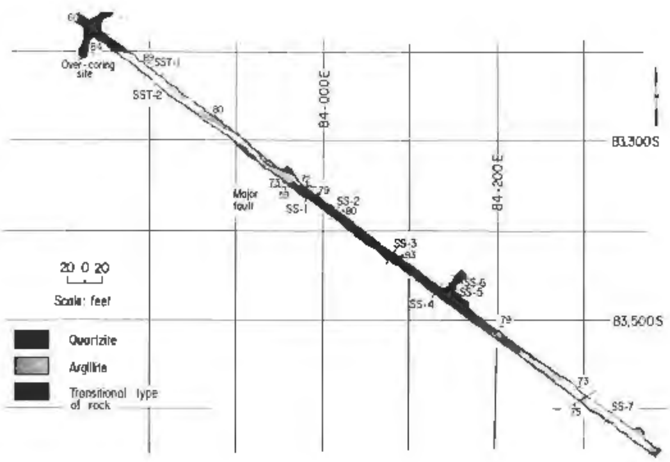

Figure 4. Location map of closure stations

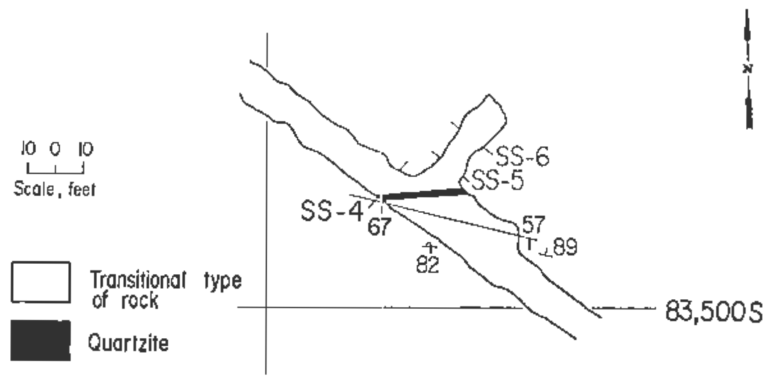

Figure 5. Detailed geology - overcoring site, stations SST-1 and SST-2

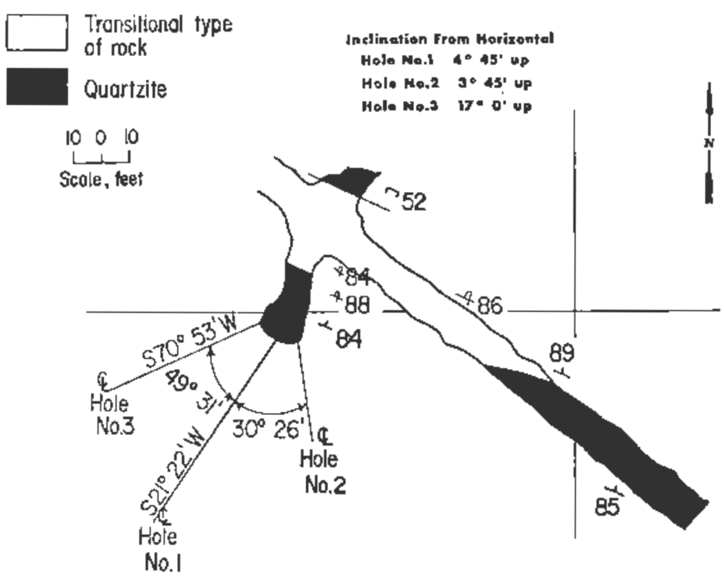

Figure 6. Detailed geology - stations SS-4, SS-5 and SS-6

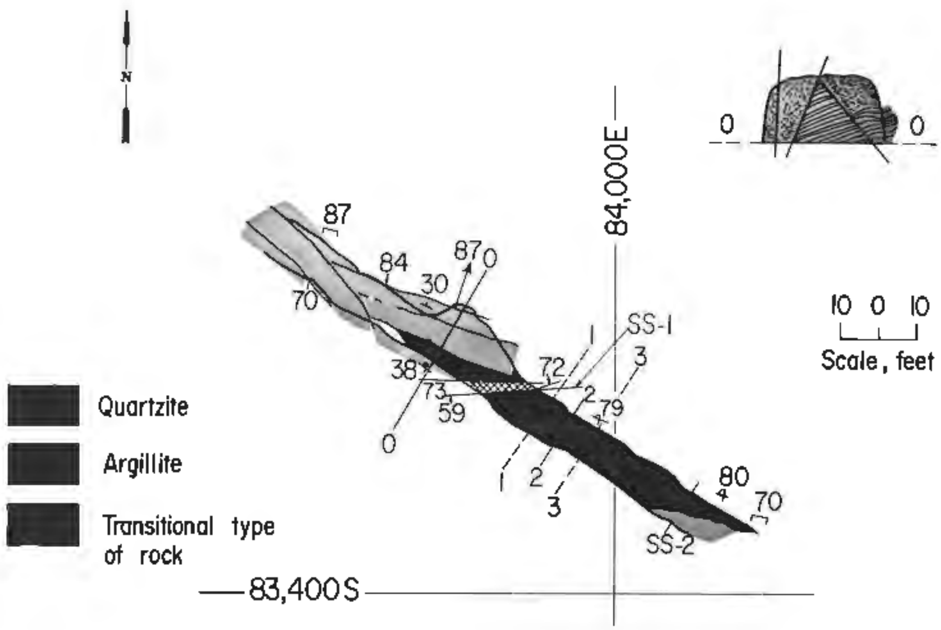

Figure 7. Detailed geology - stations SS-1 and SS-2

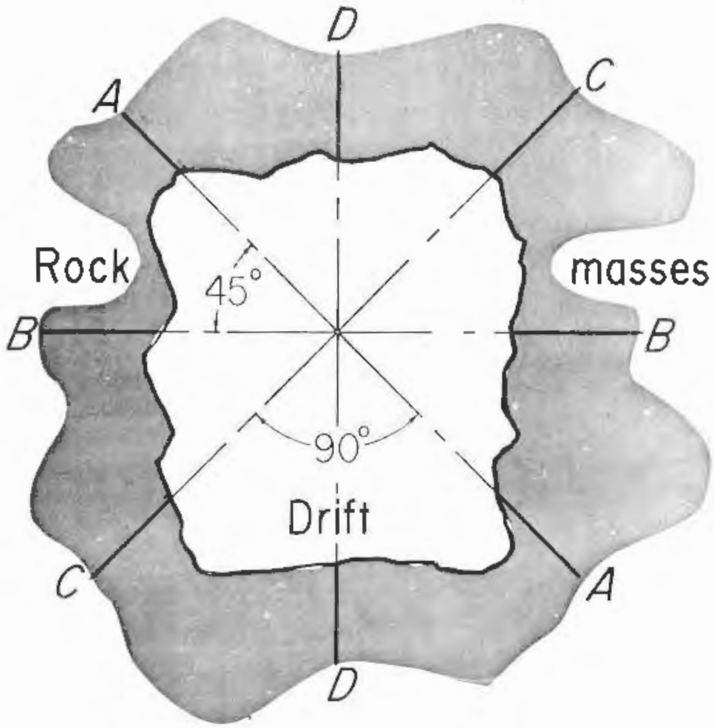

Figure 8. Orientation of measuring points

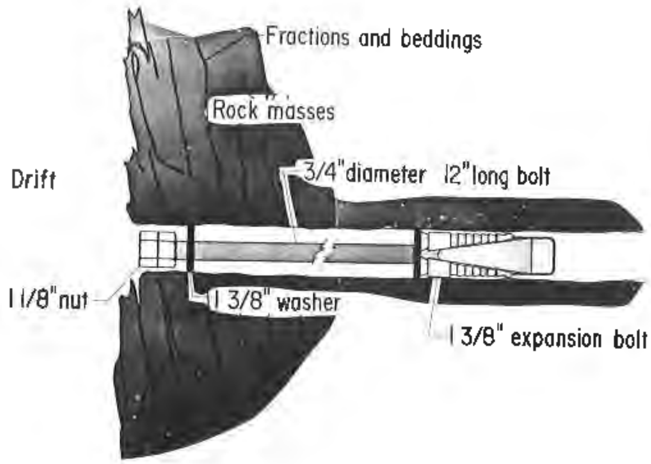

Figure 9. Details of measuring point

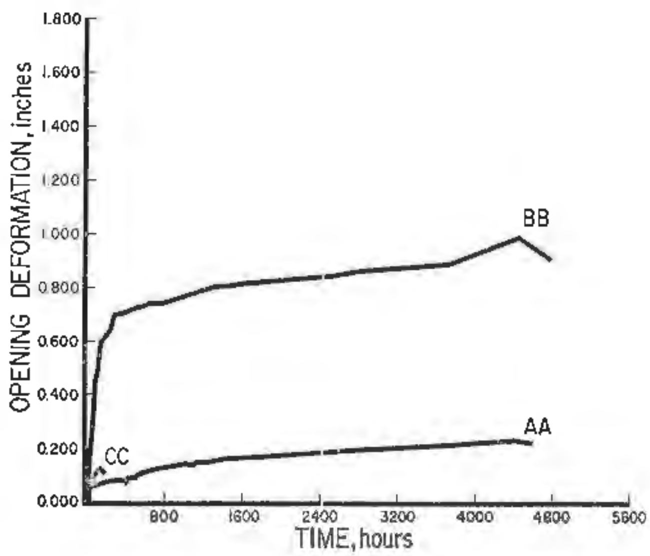

Figure 10. Time versus closure - stations SS-5 and SS-6

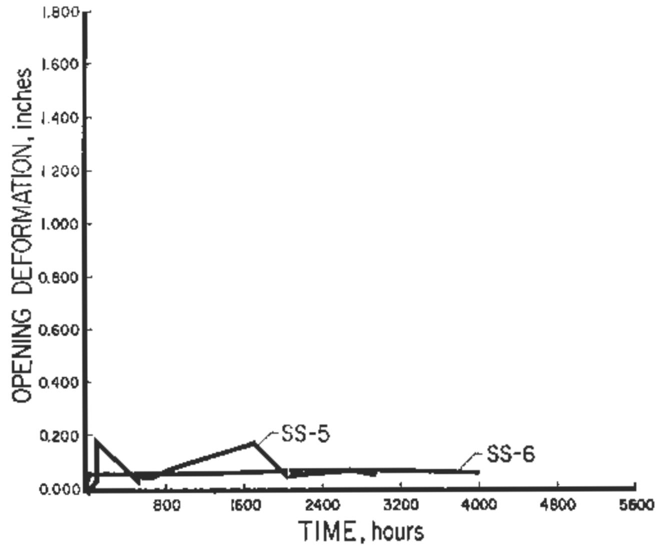

Figure 11. Time versus closure - stations SS-5 and SS-6

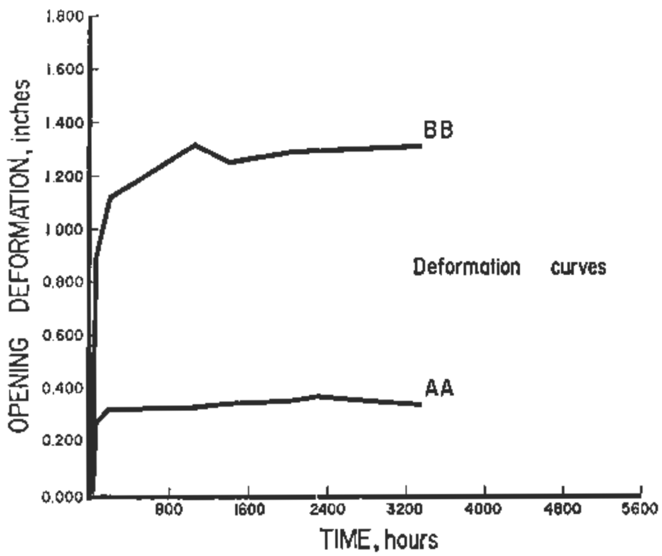

Figure 12. Time versus closure - station SS-7

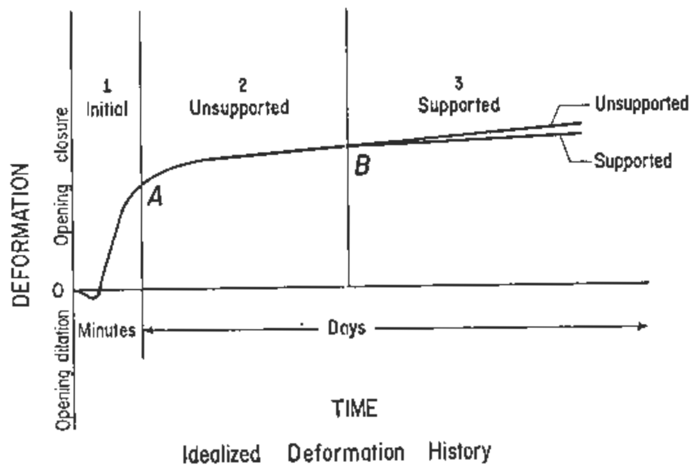

Figure 13. Idealized deformation history

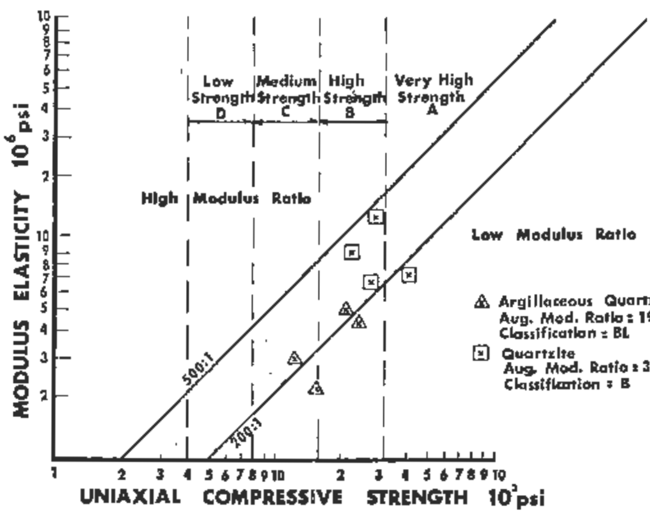

Figure 14. Uniaxial compressive strength versus modulus of elasticity

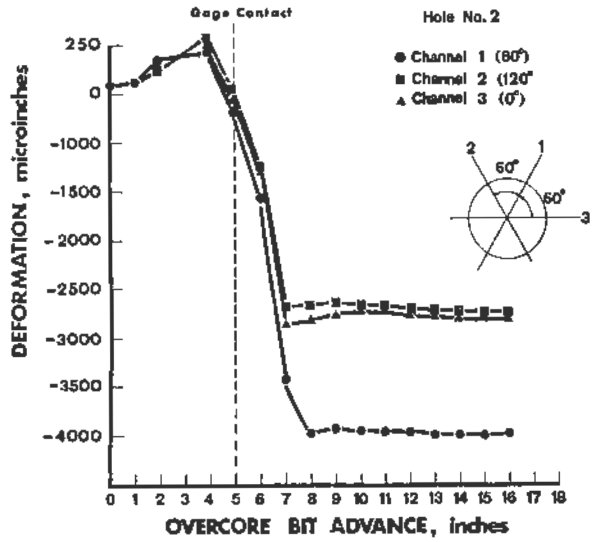

Figure 15. Typical overcoring stress-relief deformation pattern

STRESS	MAGNITUDE, psi	AZIMUTH, degree	INCLINATION, degree
σ_1	-15,234	204.7	104.9
σ_2	-8,229	174.8	17.0
σ_3	-5,437	112.5	98.1

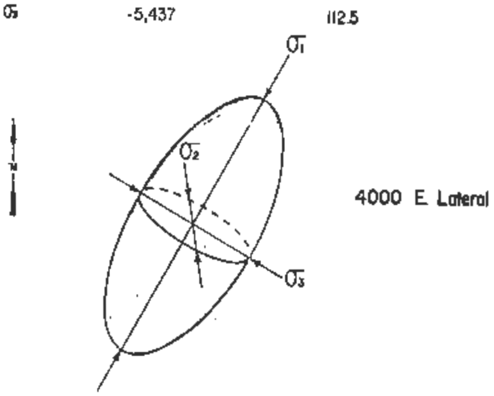

Figure 16. Stress elipsoid

Some Aspects of Bearing Capacity of Rock Mass

B. Landanyi* and A. Roy**

ABSTRACT

This paper deals with two particular aspects of the problem of bearing capacity of rock mass. The first is the effect of depth of embedment on the failure of rock beneath a circular indenter. The second is the effect of stratification and jointing on the bearing capacity. The former has an obvious application in evaluating the bearing capacity of drilled-in piles and caissons, while the latter is commonly encountered in rock mass foundations of massive structures.

Possible solutions for the two problems are discussed in the paper in terms of the limit analysis of the theory of plasticity. Proposed solutions are compared with experimental evidence obtained in a series of bearing capacity tests carried out by the authors in a columbium mine.

RESUME

Cette communication traite deux aspects particuliers du problème de la capacité portante du rocher sous une charge concentrée, notamment, d'une part, l'effet de la profondeur initiale du poinçon et d'autre part, celui de la fragmentation du rocher. Le premier effet a une importance évidente dans l'évaluation de la capacité portante des pieux forés et des caissons dans le rocher, tandis que le deuxième concerne surtout les problèmes de fondation des structures massives.

Des solutions possibles pour ces deux problèmes sont discutées en termes de l'analyse limite de la théorie de la plasticité. Les solutions proposées sont ensuite comparées avec les résultats d'essais obtenus dans une série d'essais de charge portante in-situ effectués par les auteurs dans une mine de colombium.

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INTRODUCTION

The problem of bearing capacity of both soil and rock can be considered from two different standpoints: On one hand one may be interested in how to produce failure in the material with a minimum load or minimum energy consumption. On the other, one may be more interested in determining the highest load under which failure in the material will not yet be initiated. The former has an obvious field of application in all mechanical excavation, drilling and comminution processes. The latter, however, is of more interest to the designer of structural foundations and mining support structures.

There are essentially two different approaches by which one can try to predict the design failure loads in the above two fields of application. On one hand, after assuming a suitable failure mode, one can try to predict the mean value of all failure loads and their most probable statistical distribution around the mean. A selected acceptable probability of failure or non-failure would then determine the lower and the upper bound of the failure loads to be used in the design of foundations and rock breaking tools, respectively. This approach is obviously the one usually applied to brittle failure phenomena in rock substance, and is most successful for a random distribution of inherent flaws in the rock.

On the other hand, one can consider that the failure mode itself is directly affected by the distribution and orientation of inherent discontinuities, and can try to predict the bounds to the probable failure loads by assuming, say, for the upper bound a mode essentially unaffected by the discontinuities, and for the lower bound a mode which is most affected by the discontinuities. Once the bounds have been determined, the corresponding design loads can be found by using suitable overall safety factors with the predicted failure loads.

The second approach comes quite close to the method used in the theory of plasticity for finding upper and lower bounds to the true failure load. The method which is based on the Plastic Limit Theorems developed originally by Drucker, Greenberg and Prager (1952), is strictly valid only for bodies or assemblages of bodies of elastic-perfectly plastic material, and excludes processes in which energy is dissipated by friction. In spite of the latter restriction, which obviously concerns materials such as soils, rocks and concrete, it has been found that, even in such materials, the theorems provide useful information if not the full answer.[4].

The application of limit theorems to soils has been discussed by Drucker and Prager (1952), Drucker (1953, 1954), Shield (1955), Drucker(1961) and more recently by Liam Finn (1963, 1967), Chen (1966, 1969) and Lysmer(1970). In the field of brittle materials, a successful application of limit theorems to various problems in rock and concrete has been shown by Chen (1966), Chen and Drucker (1969), and Chen (1970).

There is, however, one important difference between what is implied by the limit theorems and what may actually occur in a natural earth material such as rock or rock mass. The limit theorems are valid for a homogeneous continuum and represent a convenient method for bounding the true failure load when its value cannot be found by any other direct method. This obviously implies that there is one and only one failure load to which eventually both bounds converge.

If, however, the limit theorems are applied to a nonhomogeneous discontinuum, they may still be a useful tool for bounding the failure loads but it can hardly be expected that they will necessarily converge to a single failure load. Rather, they may be expected to converge to as many true failure loads as many distinct most unfavorable failure modes are available.

These latter considerations mean, for example, that the Prandtl's bearing capacity solution, which is known to be the true solution for a weightless rigid-ideally plastic homogeneous material, may represent only an upper limit to the failure loads observed in a non-homogeneous and discontinuous mass. On the other hand, a simple lower bound solution, which takes into account the presence and orientation of discontinuities, may come quite close to the lower limit of true failure loads for the same discontinuous mass.

It follows from the same considerations that, in order to find a safe design load for a rock foundation by using the limit analysis approach, one may be more justified to look for a convenient lower bound solution than to use the Prandtl's solution as it is usual in similar problems in soil mechanics. This view has been expressed by the senior author in a previous paper[16] in which some simple lower bound solutions for the bearing capacity of rock substance have been shown and compared with experimental information.

Following the same line of thought, the present paper develops some simple methods for evaluating lower bound bearing capacities for an embedded cylindrical punch similar to a drilled-in pile, as well as for a footing resting on a jointed rock mass. For the former, an experimental check has been obtained by a series of in-situ bearing capacity tests.

BEARING CAPACITY OF AN EMBEDDED CYLINDRICAL PUNCH

GENERAL

Very little information is available actually on the effect of the depth of embedment of an indenter on the rupture of rock under concentrated loading. This information is, however, of a considerable practical interest in the design of drilled-in piles and caissons in rock. The total bearing capacity of an embedded pile is obviously composed of the end bearing and the lateral shear resistance. However, because of possible elastic stress concentrations around the pile which may lead to rock burst and loss of lateral contact it has been suggested [6], at least for shallow embedments in rock, that the pile be designed only on the basis of its end-bearing capacity. The question to which the present study attempts to get an answer is how much gain in the end-bearing capacity is obtained when the depth of embedment is increased, and how reliable this gain is in view of the statistical character of rock failure properties.

An initial limited experimental information on the subject has been obtained at a laboratory scale and has been described in a previous paper [16]. Additional more complete experimental data have been obtained in the present study from a series of indentation tests performed in a columbium mine.

EXPERIMENTATION

Test location and rock description

A series of rock penetration tests by a 1/2 in diameter circular punch in pre-drilled holes has been carried out at the second level of the

"St.Lawrence Columbium and Metals Corporation Mine" located at Oka, Quebec. The tests were performed in a rock recess excavated for that particular purpose along a mine roadway.

The recess was located in a zone of slightly alterated carbonatite rock. The carbonatite is a complex rock composed of calcite (about 70%), pyroxene, biotite and apatite. At the test site the rock was fine grained with about 500 grains per sq. in, and showed three mutcally nearly perpendicular systems of joints, with an average spacing of 15 in . One system of joints was practically normal to the applied load.

Mechanical properties of carbonatite

In view of subsequent evaluation of test results in terms of rock properties, a series of control tests has been performed on rock specimens taken from the test site. The specimens have been obtained by drilling 23 one-foot-deep holes in the test wall in the direction of load application. The diameter of the cores was 7/8", $1\frac{1}{4}$" and 1-5/8", respectively. For all strength tests, the samples have continuously been kept under water in order to preserve natural water-saturated state of the rock. A detailed account on specimen preparation and laboratory testing procedure has been given elsewhere [23]. The number of specimens involved in each separate type of test varied from 5 to 11. Main properties determined were: Young's modulus, Poisson's ratio, uniaxial compression strength, direct and Brasilian tensile strength, and triaxial compression strength under a confining pressure of 6,000 psi.

Table I presents the main properties of carbonatite (mean value and per cent standard deviation) valid for an effective volume of 2 cu in of rock, which corresponds approximately to the size of highly stressed zone in the indentation tests. Figure (1) shows the corresponding ultimate failure envelope for the rock.

Table I : Properties of carbonatite

Young's modulus	E =	5×10^6 psi	($\pm$ 29%)
Poisson's ratio	ν =	0.25	($\pm$ 47%)
Uniaxial compression strength	σ_c =	5890 psi	($\pm$ 7%)
Uniaxial tensile strength	σ_t =	- 530 psi	($\pm$ 16%)

Triaxial compression strength at σ_3 = 6000 psi; σ_1 = 30,000 psi

Apparatus and testing procedure

Figure 2 shows the apparatus used for performing in-situ rock penetration tests. The indenter was a flat-ended cylinder of 1/2 inch diameter, made of heat-treated steel. The load was applied at the indenter by a 60-ton hydraulic jack, propped against the opposite side of the recess. The penetration of the indenter was observed on two 0.001 inch division dial gauges located on both sides of the indenter. The load was previously calibrated in terms of pressure gauge readings.

Before the start of each test, a small initial load was applied to keep the apparatus in place and to enable the displacement gages to be adjusted to zero. The load was then increased in 25 to 50 steps until failure occurred. At each step, the load was kept constant until the displacement practically ceased. Close to failure, the latter condition could not be quite fulfilled because of high steady-state creep.

After the test, the mode of failure, as well as the size and the shape of the crater when visible, were noted.

For the embedded tests, a half-inch diameter hole was first drilled to the desired depth, using a special flat-ended drill bit.

Test results

In total, 65 tests have been carried out their embedded depths varying from 0 to 2.5 in . From the total, only 51 were retained as acceptable, while the others were rejected for any of the following reasons:

(1) The failure occurred along an existing joint, giving a much lower penetration resistance.

(2) The enlarged part of the indenter came into contact with the rock surface before failure was attained.

(3) A piece of the apparatus failed.

The observation of the rock after the test showed the presence, at the contact with the indenter, of a cone made of powdered rock and kept together by capillary cohesion. Similarly, as described in [16], a bulb of crushed rock was found immediately below the indenter. In the embedded tests, the crushed bulb was surrounded by a radially cracked rock.

Figure 3 illustrates some typical examples of the observed craters and the associated rock damage.

Figure 4 shows a typical pressure-penetration curve obtained in one of the tests. It will be seen in the figure that there is a large permanent deformation preceding failure. The failure point is mostly clearly discernible on the curve as a sharp break in the slope. However, after that point, the load can still be raised to a certain higher level at which cratering occurs, or at least cracks appear at the surface. Both levels have been noted for each test and plotted against the corresponding depth of embedment. It will be seen in Figure 5 that the scatter of test results is quite considerable, which can be explained by the random variance of properties of a natural jointed rock face, resulting in a slightly different failure mode in each test. Since the original intention of the paper was to predict the lowest failure loads for foundation design purposes, it was decided to use the test results only for defining the upper and lower limits of observed failure loads. The limits are shown again in Figure 6 for the purpose of comparison with theoretical predictions.

THEORY

According to what was said in the previous paper [16], a reasonable estimation of the failure load for a deep circular punch can be obtained by assimilating the penetration of the punch to the expansion of a spherical cavity at the level of the punch tip. For a brittle rock behavior and a negligible ambient pressure, when the rock fails by radial cracking around a crushed core, such a failure load q_f can be determined from the equation

$$q_f = p_{ult} (1 + \tan \phi_c) + c_c \qquad (1)$$

in which p_{ult} is the ultimate cavity expansion pressure given by

$$p_{ult} = (Qu + S_c) \left[3 \alpha \frac{Qu}{E} + e_{av} \right]^{-a} - S_c \qquad (2)$$

where

$$\alpha = 1 - \frac{1 - \nu}{\sqrt{2n}} \tag{3}$$

$$a = 4 \sin \phi_c / 3 \, (1 + \sin \phi_c) \tag{4}$$

$$S_c = c_c \cot \phi_c \tag{5}$$

$$n = Qu/(- T_S) \tag{6}$$

The other symbols denote:

c_c = Coulomb cohesion intercept for crushed rock

ϕ_c = Coulomb angle of internal friction for crushed rock

E = Young's modulus for rock substance

ν = Poisson's ratio for rock substance

e_{av} = average volume strain (positive for volume increase) undergone by the rock in the crushed zone when passing from intact to pulverized state

Qu = Uniaxial compression strength of the rock substance

T_S = Uniaxial tensile strength for the rock substance (with negative sign)

In the considered case:

$E = 5 \times 10^6$ psi

$\nu = 0.25$

$Qu = 5890$ psi ($\pm$ 7%)

$T_S = -530$ psi ($\pm$ 16%)

and assuming,

$c_c = 0, \quad \phi_c = 40^0$

$e_{av} = 0,$

one gets from Eqs.(1) and (2),

$$q_f/Qu \quad = \quad 38.2$$

It will be seen in Figure 6 that the failure load so estimated falls about half way between the extreme bearing capacities measured at $D/B > 4$, and may be useful for estimating the mean value of failure loads in deep punching.

As mentioned in the introduction, an upper limit of failure loads for rock mass can be obtained by using Prandtl's failure model. If the cohesion determines a major part of the bearing capacity, as in the present case, the ratio q_f/Qu for a circular load at the surface can be calculated from [16].

$$q_f/Qu \quad = \quad N_c \cdot s_c/2\sqrt{K_p} \tag{7}$$

in which N_c is Prandtl's bearing capacity factor given by

$$N_c = \cot \phi \, [K_p \exp (\pi \tan \phi) - 1] \tag{8}$$

s_c is the shape factor, which may be taken according to Meyerhof, [19] for a circular load,

$$s_c \quad = \quad 1 + 0.2 \, K_p \tag{9}$$

and

$$K_p \quad = \quad (1 + \sin \phi) \, / \, (1 - \sin \phi) \tag{10}$$

Since, according to Figure (1), for solid rock, $\phi = 37^o$, one gets from Eqs. (7) to (10)

$$q_f/Qu \quad = \quad 25.3$$

for a circular load at the surface.

On the other hand, according to Meyerhof's theory based on Prandtl's model and applied to the case of a bored pile, [19], one finds for the end-bearing capacity of a deep bored pile

$$q_f/Qu \approx 66$$

When the two limiting values are plotted in Figure 6, it is found that they represent fairly accurate upper bounds to the measured bearing capacities, as expected.

A simple lower bound solution for the bearing capacity of rock under a flat cylindrical punch can be obtained from the discontinuous equilibrium solution with one vertical stress discontinuity, described by Shield (Figure 7a). If it is assumed in Figure 7a that the central cylindrical zone I under the punch has already been deformed so that vertical radial cracks have divided the surrounding zone II into radial blocks, simple statics shows that the central cylinder will not fail before the lateral confining pressure attains the uniaxial compression strength of the surrounding mass. If it is assumed that no loss of strength has occurred in the cylinder I before the mass II comes to the limit of yielding, it can be shown that the lower bound according to this model is [16]

$$q_f/Qu = K_p + 1 \qquad (11)$$

with K_p given by Eq. (10).

Taking $\phi = 37^{\circ}$ for solid rock, Eq. 11 gives

$$q_f/Qu = 5.02$$

The effect of depth of embedment has never been considered in connection with this particular model. A tentative proposal for this purpose, which follows the same line of thought, is shown in Figure 7b. For a punch at depth D it is assumed that the central rock cylinder I of length X will not fail until the surrounding radially cracked mass II yields. The lateral resistance is now increased with respect to the previous case by the ratio $(D + X)/X$.

Taking, according to the Coulomb's theory

$$X = B \tan (45^\circ + \phi/2) = B \sqrt{K_p} \qquad (12)$$

one gets

$$q_f/Qu = K_p \left(1 + \frac{D}{B\sqrt{K_p}}\right) + 1 \equiv (K_p + 1)\, d_c \qquad (13)$$

where

$$d_c = 1 + 0.5 \frac{D}{B} \cos \phi \qquad (13a)$$

Eq. (13) predicts a linear increase of lower bound resistance with the depth of embedment, but is obviously acceptable only at shallow depths, not exceeding about 5 or 6 diameters of the punch.

For $\phi = 37^\circ$ and $D/B = 6$, it yields

$q_f/Qu = 17.1$

It will be seen in Figure (6) that the latter two values represent fairly conservative lower bounds to the measured bearing capacities, as expected.

The above three modes of failure have been seen to define acceptable upper and lower limits, as well as an average value of the measured failure loads for embedded indenters. From a practical point of view, the upper limit can be used for designing rock breaking tools, while the lower one may be found useful in the design of drilled-in piles in rock. For the latter, it can be seen that, even with the usual factor of safety of 3, it would allow to apply end-bearing pressures larger than the uniaxial compression strength of the rock (taking into account the size effect), which is much more than it is allowed in most building codes.

Accordiug to Eq. (13) and taking into account the size effect, the proposed allowable end-bearing capacity for a cylindrical pile embedded in a sound rock at a depth of not more than 6 times the diameter, will be

$$q_{all} = \frac{Q_{u,red}}{F_s}(K_p + 1)(1 + 0.5\,\frac{D}{B}\cos\phi) \qquad (14)$$

where F_s is the safety factor.

Within the range of scale governed by only one particular type of discontinuities, the reduced value of Qu in Eq.(14) can be obtained by using the usual power form for the size effect

$$Qu,\ red \ = \ Qu \left(\frac{B_p}{B_s}\right)^{-\frac{3}{\alpha}} \qquad (15)$$

where Qu and Qu, red denote the uniaxial compression strengths valid for a specimen of diameter B_s and for a pile of diameter B_p, respectively. The exponent $3/\alpha$ depends on the type of brittle material and is known from experimentation to increase with increasing brittleness from about 0.20 for less brittle to about 0.50 for very brittle materials.

Finally, when substituting into Eq.(14) the mean value of $Q_{u,red}$, one should keep in mind its statistical character. As a result, the allowable end-bearing capacity, q_{all}, even if it contains a usual factor of safety, includes nevertheless a certain finite probability of failure, however small. A better alternative to dividing $Q_{u,red}$ by F_s would, therefore, be to use in Eq.(14) a value of $Q_{u,red}$ corresponding to an acceptable probability of failure.

EFFECT OF GEOLOGICAL DISCONTINUITIES

In all previous considerations it has been assumed that the rock was essentially homogeneous and isotropic, i.e., that no geological discontinuities were present in the mass that would affect the bearing capacity. This is valid for small scale rock indentation as performed in the present study, but in general, at a larger scale one would have to take into account not only the effect of discontinuities bnt also that of the ambient pressure and the weight of the failing rock mass.

The problem can be solved by adopting the Prandtl's failure mode [1 ,22] but such solutions become increasingly complicated if more than one joint system is present. It is the purpose of the following to show only a simple lower bound solution to this problem, which follows the same line of thought as in the previous case of a solid rock.

It is obvious that, in the long strip problem, the proposed lower bound solution is nothing else but the wedge analysis [20] applied to the bearing capacity problem. The problem of finding q_f consists in this analysis in finding the limiting lateral pressure σ_L which can keep the central block in limiting equilibrium under the vertical pressure q_f . Here, σ_L is the Rankine passive earth pressure the average value of which, for a homogeneous and isotropic rock mass with Coulomb parameters c and ϕ and the effective unit weight γ, is equal to (Figure 8a)

$$\sigma_L = p_n \; K_p + S_c \; (K_p - 1) + 0.5 \; \gamma \; X \; K_p \qquad (16)$$

where p_n is the effective overburden pressure, $S_c = c \cot \phi$ and X is given by Eq. (12) .

On the other hand, if σ_L is known, the value of q_f can be calculated from

$$q_f = \sigma_L K_p + S_c (K_p - 1) \tag{17}$$

in which the weight of the wedge I has been neglected [6].

Substituting (16) into (17) and taking into account (12) one gets

$$q_f = p_n K_p^2 + S_c (K_p^2 - 1) + 0.5 \gamma B K_p^{2.5} \tag{18}$$

which is identical to the form given in [6] and [25].

A completely analogous lower bound solution can be obtained if the rock mass contains two systems of discontinuities oriented so that, for minimum bearing capacity, the expected slip will follow the discontinuities, as in Figure (8b). In the latter case, according to the theory of failure of stratified materials [14, 15]

$$\sigma_L = (p_n + 0.5 \gamma B \cot \beta_1) K_{p2} + S_{c2}(K_{p2} - 1) \tag{19}$$

which, should be substituted into

$$q_f = \sigma_L K_{p1} + S_{c1} (K_{p1} - 1) \tag{20}$$

to give the corresponding lower bound bearing capacity of rock mass. In (19) and (20), the passive earth pressure coefficients K_{p1} and K_{p2} should be evaluated from

$$K_{p1} = \frac{\sin (2 \beta_1 + \phi_1) + \sin \phi_1}{\sin (2 \beta_1 + \phi_1) - \sin \phi_1} \tag{21}$$

$$K_{p2} = \frac{\sin (2\ \beta_2 + \phi_2) + \sin \phi_2}{\sin (2\ \beta_2 + \phi_2) - \sin \phi_2} \tag{22}$$

Also,

$$S_{c1} = c_1 \cot \phi_1 \tag{23}$$

$$S_{c2} = c_2 \cot \phi_2 \tag{24}$$

in which β_1 and β_2 are the orientations of the two joint systems with respect to the major principal stress directions, while $(c_1\ \phi_1)$ and $(c_2\ \phi_2)$ are Coulomb strength parameters for shear failure along the joints (1) and (2) .

An interesting and useful point about the type of analysis is that it can easily be carried out by a semi-graphical method as shown in [25] and in Figure 9. The graphical method may even be preferable because it enables to visualize clearly the most unfavorable failure mode, and enables moreover, to consider also non-linear failure envelopes. Actually, as shown in Figure 9, for a vertical load acting on a horizontal surface, it is not even necessary to trace the Mohr circles but only two successive rectangular triangles up to the respective failure lines.

It is obvious from Figure 9 that in a general case, when the two joint systems are not so unfavourably oriented, one has to look for the minimum q_f-value by combining all probable failure modes. E.g., had β_1 in Figure 8b been very small, the q_f for failure along the joints (1) would have been higher than that accross the solid rock, so that q_f' would have been the lowest failure pressure. In general, for a vertical load and nen-symmetric joint orientation, there may be as many as seven different failure modes to consider (One through the rock substance, one along the joints only on each side, and two combined modes on each side.

Finally, it should be noted that the simple theory of shear of a stratified material used here, which assumes only two distinct rock mass failure modes: either along the discontinuities or across the material, is an over simplification. In a real jointed rock mass, failure can actually occur also by a combined mode, described by Patton [21] and Ladanyi and Archambault [17]. The latter have proposed a modified shear strength equation for jointed rock mass (Eq.27 in [17]) enabling the failure envelope to be determined from the shear properties of rock substance and rock joints respectively, as soon as the orientation of the joints with respect to the general shear direction is known. It is noted that the equation enables also to consider a progressive failure mode in which failure is preceded by a gradual loss of interlock. Figure (10) shows a schematic application of the latter method to a case in which there are two mutually perpendicular systems of joints.

The above analysis of bearing capacity of jointed rock has been made without respect to the form of the footing. Strictly, it is valid for a long strip, but taking into account the probable presence of a third perpendicular system of joints eliminating the hoop stress, the same solution represents at the same time the lowest limit for a rectangular or circular footing as well. Finally, the effect of depth of embedment in the rock mass can be taken into account similarly as in the homogeneous case (Figure 7b) by multiplying the lateral resistance σ_L by the ratio $(D + X)/X$, in which, however, X is equal to $B \cot \beta_i$ β_i being the slope of the most unfavorable failure plane for the central zone.

CONCLUSIONS

Two particular aspects of the problem of bearing capacity of rock mass have been studied in this paper: the effect of the depth of embedment and the effect of the presence of geological discontinuities. For the former, an experimental study of rock indentation by a flat circular indenter has shown a general increase of bearing capacity with the depth of embedment, but also a considerable scatter of experimental results. It is found, however, that experimental points fall within the bounds which can be determined by using the concept of limit theorems of the theory of plasticity. It is concluded therefrom tentatively that in the rock mass, due to its non-homogeneous and discontinuous character, the bounds to the bearing capacity may not necessarily tend to a single value, as in an ideal homogeneous rigid-plastic material. As a consequence, a true solution for a homogeneous material may become an upper bound for such a non-homogeneous material, while a simple lower bound solution for the former may come close to the true solution for the latter. A conservative approach is therefore proposed for the foundation design purposes, based on a simple lower bound solution. The same approach is then extended to cover also the presence of one or two sets of geological discontinuities.

ACKNOWLEDGEMENTS

This work has been carried out at the Mining Engineering Department of Ecole Polytechnique, Montréal. The authors want to thank the Mines Branch, Department of Energy, Mines and Resources, Ottawa, for the research grant by which this study has been made possible. The authors wish to thank also the St.Lawrence Columbium & Metals Corporation for permitting them to carry out the tests underground, and in particular their staff of the Columbium Mine at Oka, Québec, for preparing the test site and for offering invaluable help during the testing. Valuable comments made by Dr. D.E. Gill, which have been of much help in improving the clarity of presentation, are highly appreciated.

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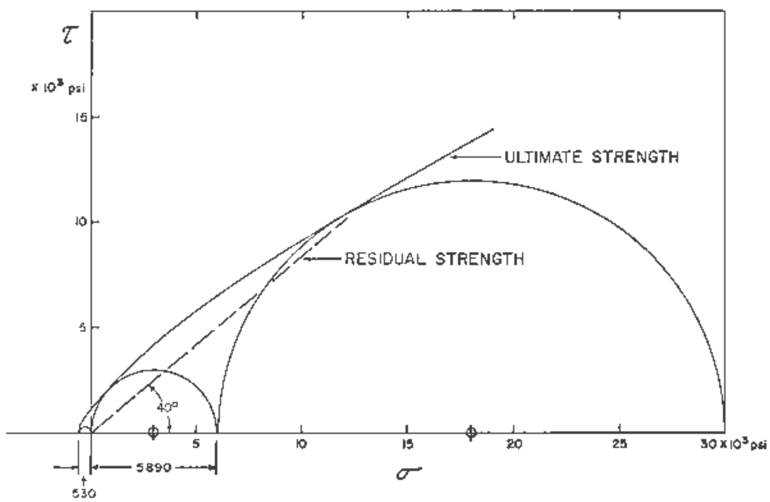


Figure 1. Ultimate and residual strength of carbonatité

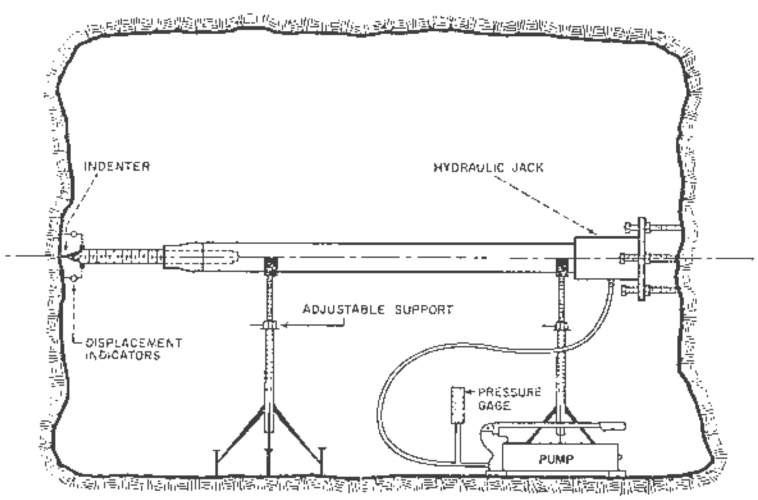


Figure 2. Schema of testing rig for bearing capacity tests

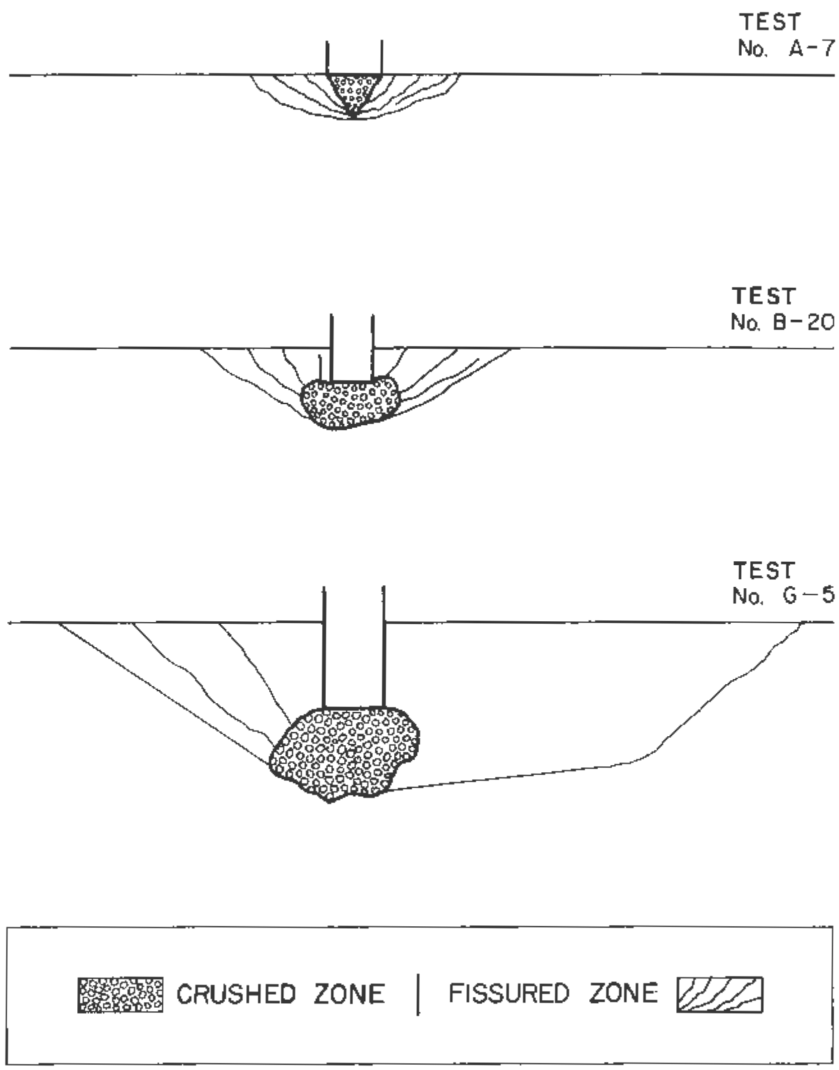


Figure 3. Types of failures ovserved in the tests

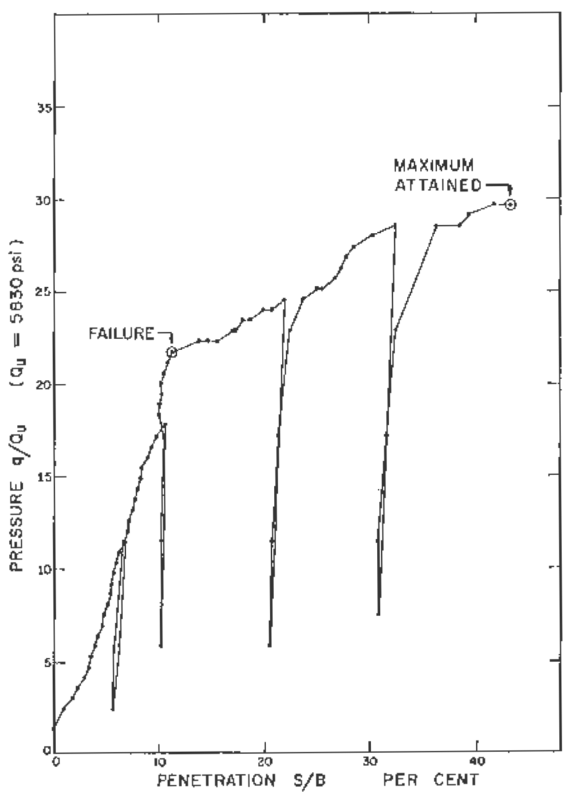


Figure 4. Pressure-penetration curve obtained in test B-18 (Indenter diameter: B=0.50 in; depth of embedment: Do=0.665 in; Do/B= 1.33)

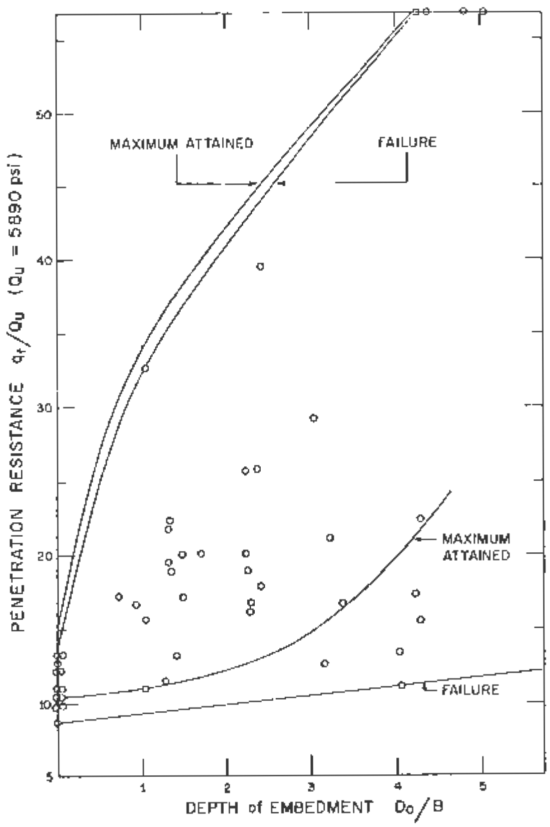


Figure 5. Penetration resistances observed in the tests at different depths of embedment

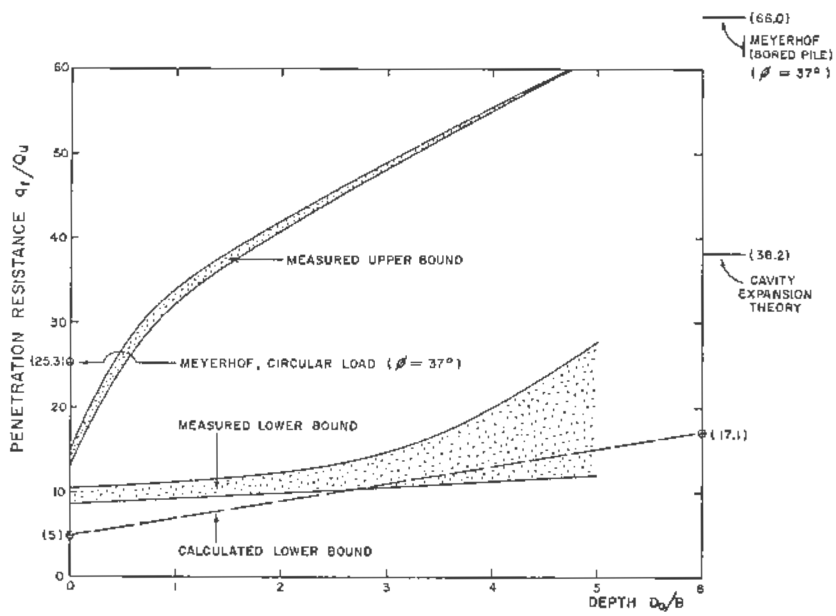


Figure 6. Comparison of experimental results with principal theories

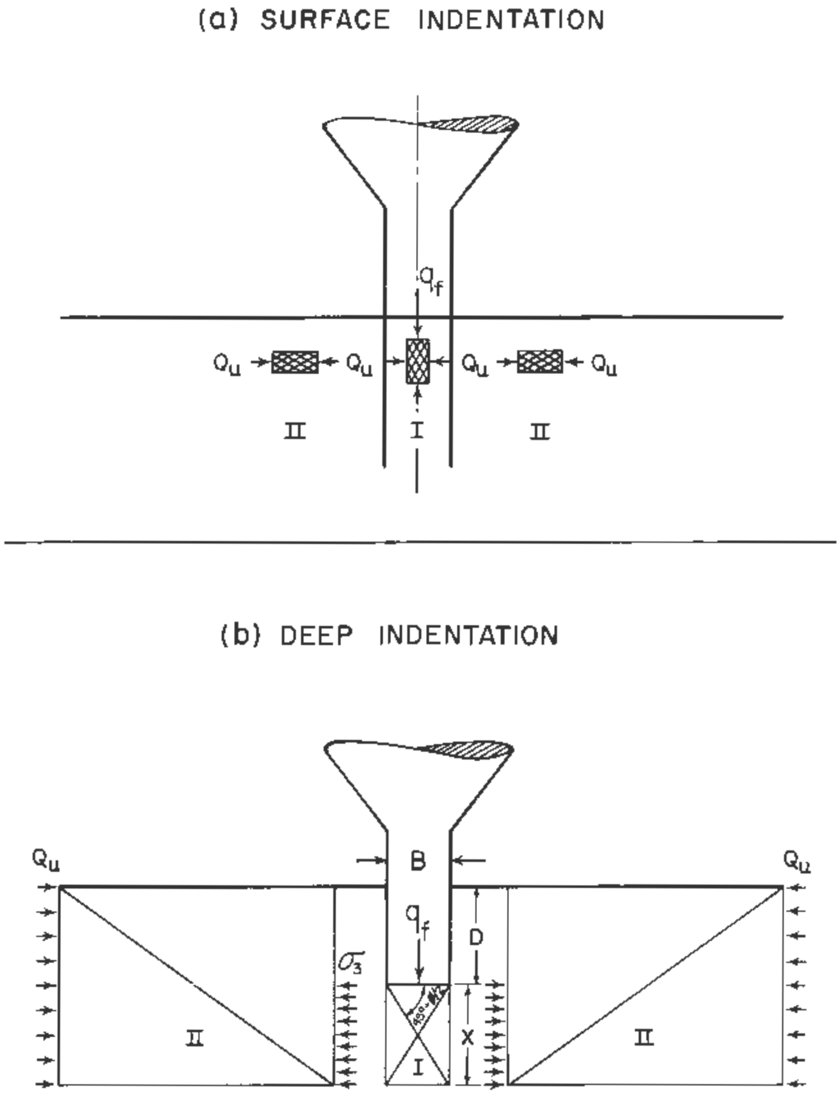


Figure 7. Stress fields for lower bound indentation analysis

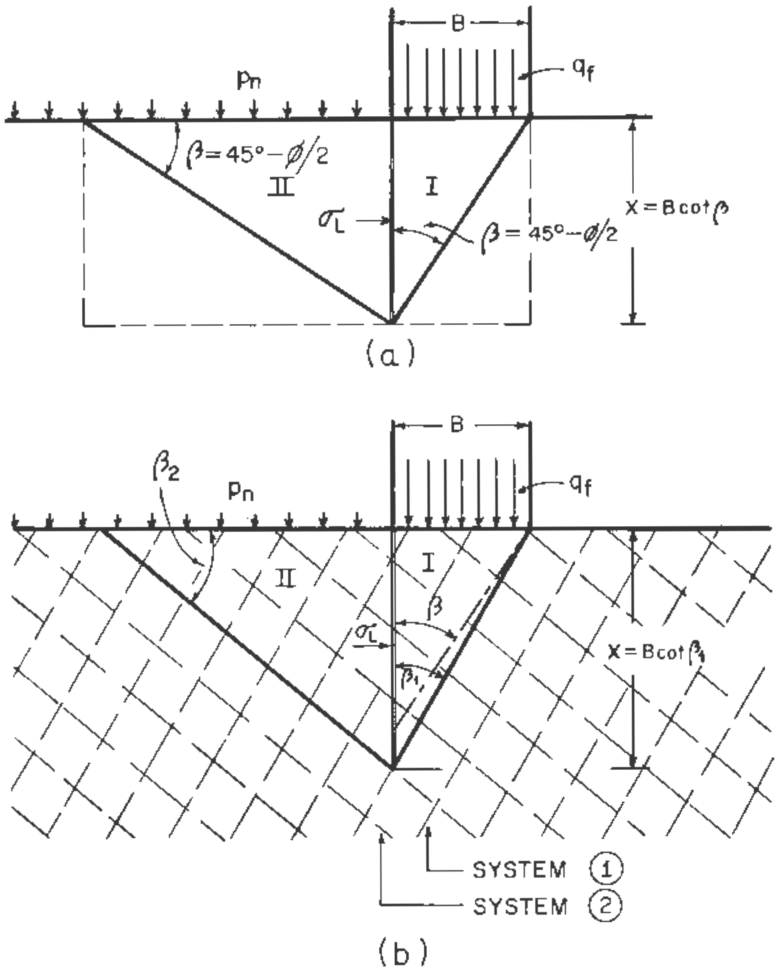


Figure 8. Stress fields (a) for homogeneous, and (b) for jointed rock mass

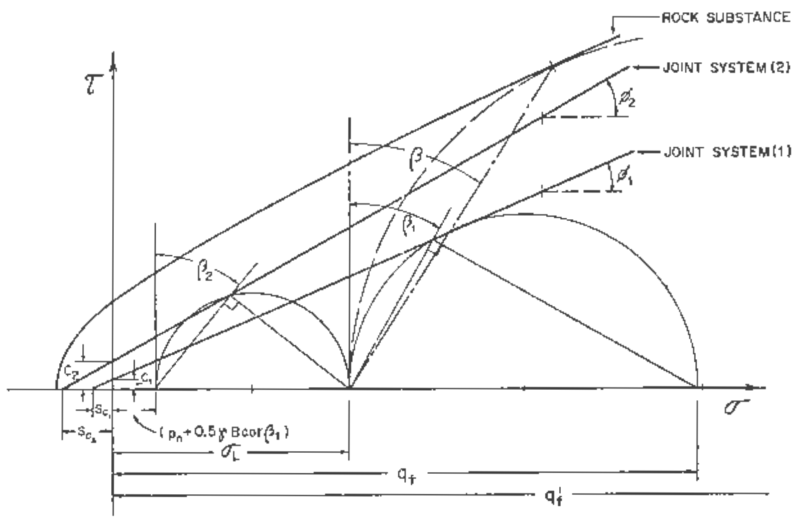
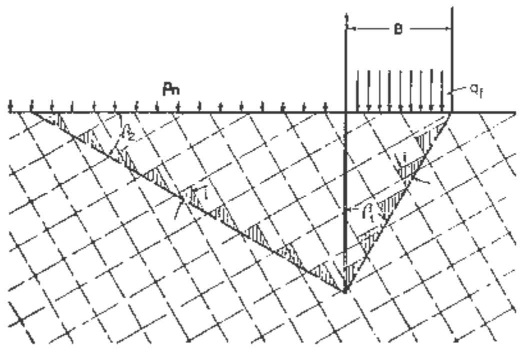


Figure 9. Graphical method for lower bound bearing capacity determination in jointed rock



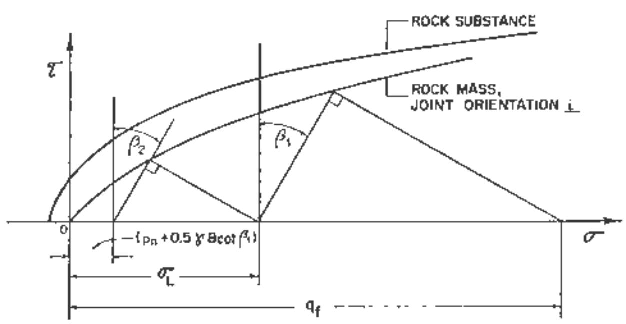


Figure 10. Case of overall sliding planes deviating from joint direction

Influence of Topography on the Pre-Mining State of Stress

W. G. Pariseau *

ABSTRACT

The finite element technique has been used to analyze the influence of topography upon the pre-mining state of stress. A two-dimensional (plane strain) region of moderate relief (1200 ft) is developed through a simulated erosional sequence leading to a series of identical and symmetric peaks and valleys. The resulting state of stress is non-uniform as far below the valley floor as the peaks are above. Relatively high vertical stresses occur beneath the peaks, and relatively high horizontal stresses occur below and above the valley floor. A small region of failed material developes near surface about the peaks.

INTRODUCTION

The analysis of stress about a proposed surface or underground excavation and the follow-on design has as essential prerequisites a knowledge of: (1) the properties of the material encompassing the excavation, (2) the geometry of the proposed excavation, and (3) the applied loads. The latter may be static or slowly varying such as those due to gravity, tectonism and temperature change or transient such as those due to seismic and blast waves. But in common usage, "applied load" refers to the static state of stress existing in the vicinity of the opening prior to excavation - the pre-mining state of stress. The purpose of this paper is to examine the influence of topography on the pre-mining state of stress.

Two well-known methods that are frequently employed for the estimation of the pre-mining state of stress consist of: (1) measurement of the in situ stresses and (2) calculation of the vertical stress as the unit weight of material multiplied by the depth and the horizontal stress as the vertical stress multiplied by a constant (vertical shear stresses are assumed to be absent). Both methods as generally used involve the tacit assumption that the stress so determined is uniform over a rather large region, a region approximately an order of magnitude greater in linear dimension than the opening and perhaps throughout the entire ore deposit.

Although the assumption of a uniform state of stress over a large region may be justified in deep mines situated in areas of low topographic relief, where mining depths are of the order of the relief of surface topography, the state of pre-mining stress may possibly be sufficiently non-uniform to invalidate both methods of estimating the pre-mining state of stress and consequently the follow-on engineering. Strictly speaking, a non-uniform stress state would not invalidate the in situ stress measurement approach. However, the number of such measurements required to adequately define a non-

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uniform pre-mining stress state would likely entail a prohibitive cost to a mining venture and perhaps to many civil projects, so that as a practical matter some other approach would be required.

In this paper a simplified problem involving the estimation of the pre-mining state of stress in a region of moderate topographic relief is posed. The results of an analysis of stress obtained by the finite element technique are then presented. A discussion of results and a short conclusion follows.

FORMULATION OF THE PROBLEM

A fundamental difficulty in formulating any problem involving the state of stress in the earth is quite literally deciding where to start. Regardless of the mathematical idealization of material behavior, one must decide at the outset what the zero of stress and strain is to be. Even an elastic material when loaded by gravity will exhibit stress changes upon further loading that depend not only on the loading path (1,2)*, but upon the previous loading history as well**. The final state of stress, the one used in design evaluation, will be a function of the initial stress state and the stress changes caused by the additional loading or unloading.

The geological evolution of a region of moderate topographic relief would in general appear to involve a rather complex loading sequence due to recurrent episodes of accretion and erosion alone, not to mention tectonic processes. The problem of defining a satisfactory zero of stress and strain is thus by no means a trivial one without important consequences. However, despite the inherent difficulties of the problem, one must begin somewhere.

In the present instance, the earth's surface is assumed to be level prior to dissection by down-cutting streams. This is the reference configuration with respect to which displacements and strains are to be computed as erosion proceeds and topographic relief developes. In the reference configuration, the material is assumed to be stressed by gravity under complete lateral restraint. Thus, initially the vertical stress σ_v is the unit weight of material multiplied by the depth, and the horizontal stress σ_h is the vertical stress multiplied by a constant***. There are no vertical shear stresses.

As erosion proceeds, material is removed from the region; valleys are formed and deepened. A general uplift occurs that is greatest in the valley floors and least about the adjacent peaks. The state of stress changes throughout the region; it is no longer uniform. A region of moderate relief has developed, but no mining has yet taken place. The problem that is now posed is: calculate the pre-mining state of stress in this region.

The problem of calculating the pre-mining state of stress in a region of moderate topographic relief has been examined under the following assumptions and in the following way: The material is assumed to be homogeneous, isotropic, linearly elastic, non-hardening

* Numbers refer to corresponding items in REFERENCES at the end of paper.

** In fact the former implies the later.

*** If ν is Poisson's ratio, then $\sigma_h = (\frac{\nu}{1-\nu})\sigma_v$

and to fail according to a Mohr-Coulomb criterion. The region is assumed to be originally level and stressed uniformly by gravity under complete lateral restraint. An idealized series of identical peaks and valleys that are long in comparison to their lateral dimensions is then formed by a sequential removal of symmetric, chevron-shaped layers of material as illustrated in Figures 1 and 2. The problem is one of plane strain. The stress, strain and displacements caused by removal of the first layer are added to the initial quantities to obtain the final stresses, strains and displacements in the remaining material. The final state of stress is the initial state of stress as regards the removal of the second layer and so on. The process of computing the effects of removing an individual layer of material and adding these to the initial state to obtain the final state is repeated until the present configuration of the region is obtained. The last computation results in the desired pre-mining state of stress.

The requisite calculations have been made using an elastic-plastic finite element computer program due to Dr. H. D. Dahl (3). Constant strain triangles are employed, and the master stiffness matrix is re-assembled as element failures dictate. The incremental nature of the solution process accounts for loading path effects. The element mesh used in the problem is shown in Figure 3.

RESULTS

The main results of the previously described calculations are presented graphically in Figures 4 and 5 which show contours of vertical and horizontal stress concentrations. The vertical reference stress is computed as the product of unit weight and depth from the original surface, and the horizontal reference stress is a constant times the vertical reference stress. The actual stresses can be obtained by multiplying the reference stresses by the appropriate number in Figures 4 and 5. The principal stress directions are indicated in Figure 6. Compression is reckoned positive; the greatest compression is the major principal stress σ_1. The total uplift of the valley in this problem was 30 inches, and the total uplift of the inter-valley peak was 25 inches. The surface displacement caused by removal of the last layer of material is shown in Figure 7.

The material properties used in the problem were:

Unit weight .... 156 pcf
Young's modulus .... 10^6 psi
Poisson's ratio 0.25
Compressive strength ... 960 psi
Tensile strength .... 80 psi

Young's modulus, the compressive strength, and the tensile strength were obtained through an order of magnitude reduction of the laboratory rock properties, that is, all dimensional moduli were reduced by a factor of ten from the values measured in the laboratory for application to the field problem.

DISCUSSION OF RESULTS

The results shown in Figures 4 and 5 show clearly that in this case the pre-mining state of stress is significantly influenced by topographic relief. The influence of 1200 ft of vertical relief generated over a horizontal distance of 1800 ft would be noticeable to depths in excess of 1200 ft. As was anticipated, the influence of relief extends as far below the valley as it does above.

The horizontal stress is more strongly influenced than the vertical stress, particularly above the valley floor. Near surface it exceeds the vertical stress. The horizontal stress concentration is roughly the same over each horizon between peak and valley floor. Below the valley floor, the horizontal stress concentration increases more rapidly below the valley than beneath the peak. Over horizons below 3500 ft beneath the valley floor, the horizontal stress concentration tends to become uniform once more.

The vertical stress concentration tends to be uniform on inclined planes approximately parallel to the surface below 200 ft to a depth of 1200 ft below the valley floor. On horizons at greater depths the vertical stress concentration tends to become uniform.

With few exceptions, all element stresses are compressive. The exceptions occur near the peak where several elements fail. The failures occur with one normal stress tension and the other compression (in the plane of deformation). It is difficult to classify such failures as tension or shear failures, since little is known about rock behaviour under such conditions. A suggestion of cracking subparallel to the surface presents itself as does the phenomenon of exfoliation. These failures occurred as the last layer of material was removed. The first failures occurred in the fourth layer during removal of the third. Subsequent failures which were few in number occurred in a similar fashion, near surface. The failed material is removed during the next deepening of the valley. The shaded elements in Figure 3 failed at sometime during the erosional sequence as simulated on the computer.

Speculation concerning some practical implications of these results may be in order. For example, if flat ore deposits in this region were exploited by room-and-pillar methods, relatively heavy vertical pillar loads would be expected under the peaks and, abnormally high horizontal stresses might lead to excessive spalling of pillars below the valley floor. Above the valley floor, the horizontal stress concentration is about the same on each horizon. If bed separation in the roof occurred, the high horizontal stress would improve roof stability to a point by reducing sag tension. Conditions near the peak would appear to be rather adverse because of the presence of tensional stresses and the existence of zone of material that had failed prior to mining. In practice, a detailed design would be required to ascertain the precise influence of topographic relief upon a mine design. The influence of a high horizontal stress, in particular, would warrant careful consideration as would the shear stress.

CONCLUSION

The results of this study are strictly applicable only to the idealized problem that was investigated. However, these results should illuminate somewhat the general problem of assessing the influence of topographic relief on the pre-mining state of stress.

Although no field data is presently available for a direct experimental comparison with the computed results, the method appears to be reasonable, and in fact simulation of erosional sequences leading to present topography may be the only practical way of estimating the pre-mining state of stress in regions of significant relief or in regions of non-uniform stress. The material model was deliberately kept simple in this example, however complexities such as anisotropy and heterogeneity due to the presence of different rock units, veins, dikes, sills and so forth can be introduced into the analysis. Effective stresses have been implied throughout. In practice, an analysis of this type should be used in conjunction with a carefully planned series of field measurements in order to obtain the maximum benefit at least cost.

ACKNOWLEDGEMENTS

Grateful acknowledgement is made to the Montana Tech Foundation and Computing Center for financial support and use of facilities.

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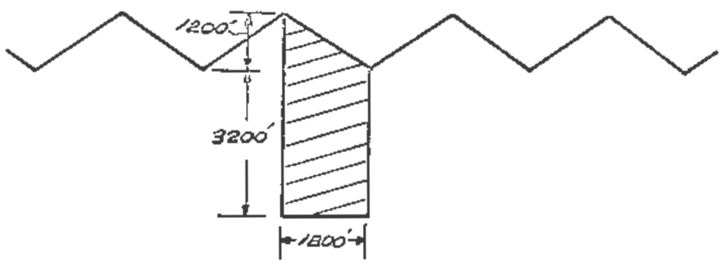

Figure 1. Model topography - a series of symmetric peaks and valleys exhibiting 1,200 ft of vertical relief

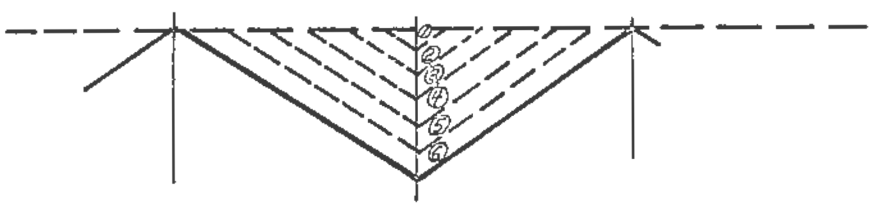

Figure 2. Simulated valley erosional history # a sequence of removal of six symmetric, chevron-shaped material layers

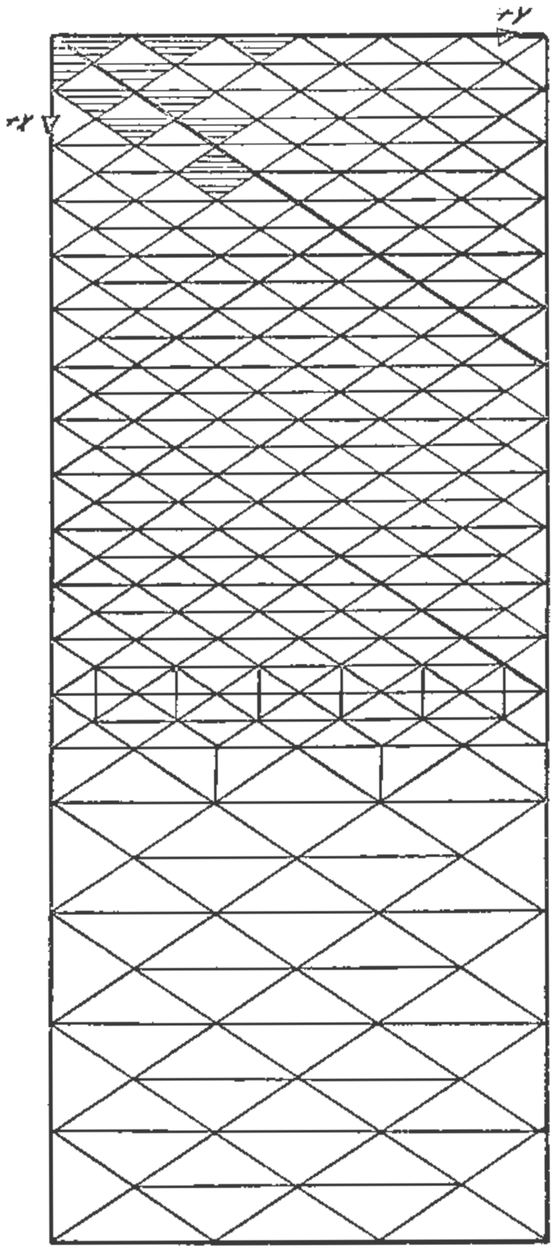

Figure 3.

Finite element mesh
used in the study

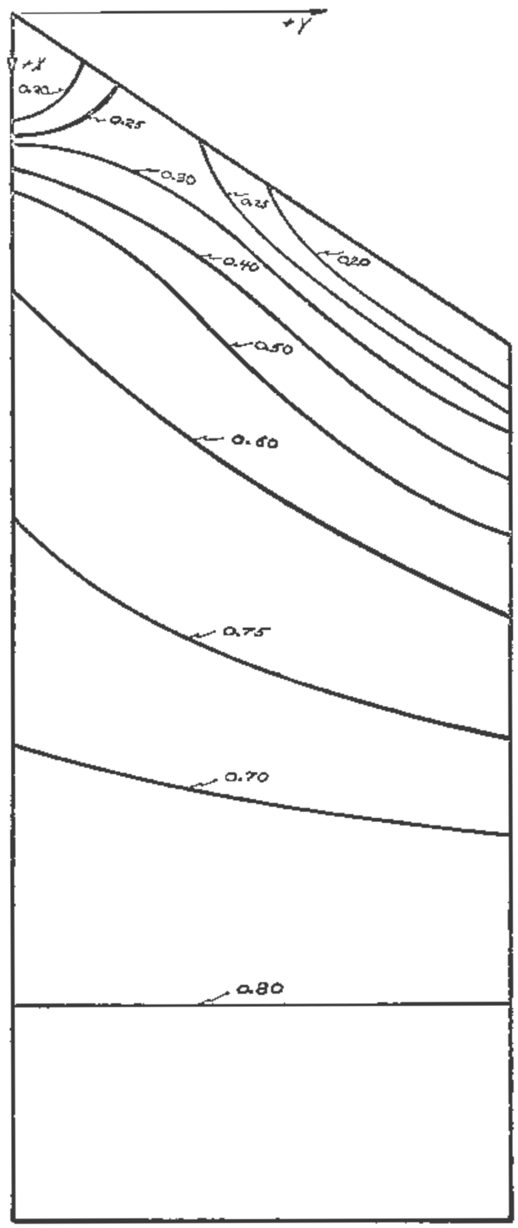

Figure 4.
Contours of vertical stress concentration

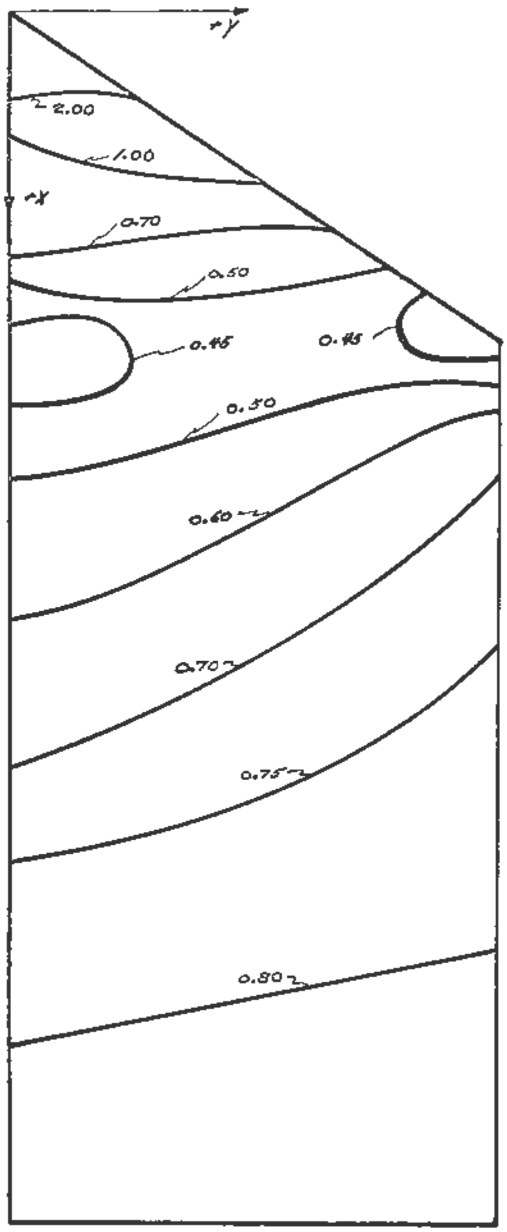

Figure 5.
Contours of horizontal stress concentration

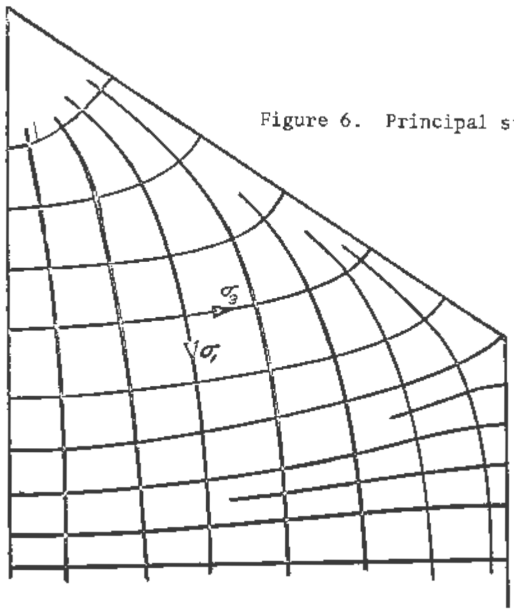

Figure 6. Principal stress directions

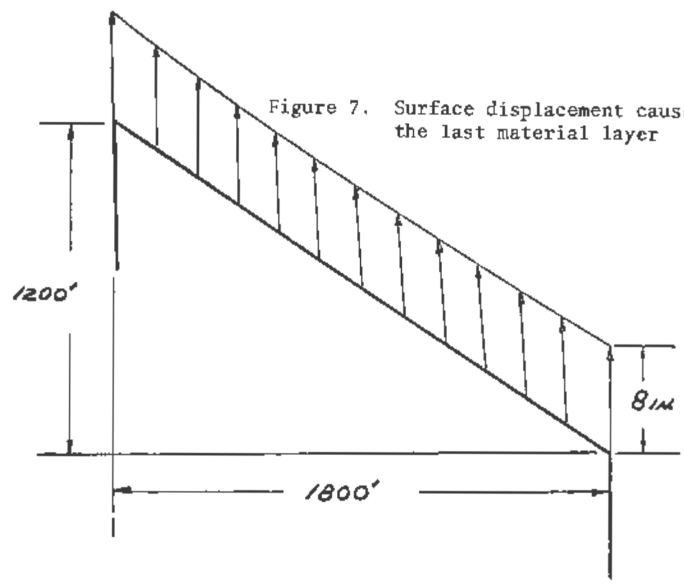

Figure 7. Surface displacement caused by the removal of the last material layer

Mine Design - A Systems Approach

K. Wardell*

INTRODUCTION

It goes without saying that a consultant is never consulted unless someone has a problem. His primary function therefore is that of problem analyst and, hopefully, problem solver. Since everybody has problems, of one kind or another, and since I frequently found that my analyses and preferred solutions were different from those prescribed by others, I was led to ask whether there existed any general methodology for problem analysis and solution.

Of course, something depends upon how, and by whom, the problem is posed. Sometimes, it may be represented as purely technical in character, cleansed of any sordid cost or commercial implications. I can only say that I have never yet been involved in a mining problem where the cost implications did not, in fact, turn out to be highly significant.

Problems are also frequently presented as being within a highly specialised field of knowledge and experience such as engineering geology, mineral exploration, mine mechanisation, ventilation, transportation, benefication etc. Again, my experience is that it is rarely possible to generate a solution to such a problem simply within the context of a single specialisation.

Analysis of most mining problems reveals elements that are not quantifiable, and their solutions involve assumptions and extrapolations which are sometimes of heroic proportions. Inevitably, they raise questions of uncertainty and risk.

Clients posing mining problems may be potential investors and/or mine operators and managers. Their level of technical knowledge may vary considerably. But, their intellectual capabilities, their breadth of knowledge and their diversity of experience frequently enable them to pick out the flaws in analysis and to pose searching questions, in a most disconcerting way.

It is my belief that mining problems, including those of rock mechanics and mine design, do fall within a general framework of problem analysis and problem solving. This is generally called "systems thinking", "systems engineering" or, "the systems approach". Contributions to the underlying philosophy have come from many different disciplines (1). I hope this paper will do a little to indicate its relevance to problems in rock mechanics and mine design.

GENERAL SYSTEMS CONCEPTS

A system can be defined as a complex unity consisting of a number of different elements or sub-systems which are interdependent and are designed to serve a common aim. In this sense a mine can be considered as a system see Figure 1. Its aim is the

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optimal efficiency of the mine as a whole. The concept of efficiency involves numerous criteria, the more important of which are:

minimum risk of complete failure.
minimum capital cost.
minimum manpower.
minimum total operating cost.
minimum maintenance of equipment.
minimum support for and maintenance of underground roadways.
maximum output per unit time (shift/day/year).
maximum life of equipment.
maximum percentage of mineral extraction.
maximum safety.
maximum consistency of performance.

Not all these criteria are quantifiable but, optimal efficiency requires that the mine is designed to satisfy all relevant criteria of efficiency (quantifiable or otherwise) to the best possible extent.

Optimal efficiency is not measurable in terms of any single criterion. Nor can optimal efficiency of the whole be achieved by maximising the efficiency of one of its sub-systems because changes in one sub-system produce interactions with others and may cause a malfunction of the system as a whole. Malfunction, in this sense, describes a failure to satisfy adequately one, or more of the various criteria of efficiency.

A mine operates within a set of environments - geological, physical, social, political, technical and economic. There are strong, highly complex inter-relationships between a system and its environments. The comprehensive design of a new mine, or the modification of an existing one, involves consideration of all environments. In this paper I shall be concerned largely with extraction systems in relation to physical/geological environment.

If the adaptability of an extraction system (e.g. room and pillar, longwall, sub-level caving etc.) to a variety of environmental states tends to be uncertain, the system can be regarded as poorly structured. It then needs to be re-structured so that its adaptability is certain or, at least, more certain. Ability to develop system specifications for optimal efficiency depends, very largely, on the extent to which the system can be re-structured. The criteria of efficiency must be clearly defined. Unchangeable constraints and the whole range of variables have to be comprehended. Sub-systems and their inter-relationship with the physical/geological environment must be recognised.

Although this paper is primarily concerned with the phyiscal/geological environment, other environments cannot be ignored. They impose their own particular constraints. For example, political/social environmental constraints are represented, in part, by the existing law and regulations affecting the safety, health and welfare of mine workers and by the law generally. The technical constraints are represented by the present state of knowledge and experience concerning the design and operation of mining techniques and equipment. The economic constraints are represented by product markets and proceeds, by availability and cost of capital and, by the availability and cost of manpower.

One important criterion of system efficiency is the minimum risk of complete failure i.e. where the whole, or a major productive part of a mine has to be totally abandoned. Failure of this kind may result from a major catastrophe such as fire, flood explosion or from ground collapse. Examples of the latter are the disasters at the Coalbrook mine in South Africa, at Champagnoles in France and at Mufulira in Zambia.

In developing design specifications for a mine, all the criteria mentioned in paragraph 1.1* must be considered but, the list is not exhaustive. Additional criteria may exist and require satisfaction. Every new mining situation presents a unique set of environments and involves special criteria and constraints.

"Ultimately all policies are made and all systems designed and chosen on the basis of judgements. The question is whether such judgements have to be made on the basis of inadequate and inaccurate data, ambiguous or undefined issues and conflicting personal opinions or, whether they can be made on the basis of an analysis of adequate and reliable information, relevant experience and clearly defined issues"(4). The systems approach is not a model for mine design nor a substitute for judgment nor is it a decision making technique. It aims specifically to define issues, to define and analyse the interdependence between sub-systems, systems and environments and, to indicate alternative solutions to problems. In reality, it is nothing more than good design engineering.

THE PHYSICAL/GEOLOGICAL ENVIRONMENT

No two physical/geological environments are identical. Even at the same mine the physical/geological environment may change significantly from one section of the mine to another. If one considers different mines in a particular locality or, mines in different localities in the same country or, mines in different countries or, mines working different minerals, the variability is immense. Effective mine design must, therefore, be based on some comprehension of the total pattern of phenomena which affect or, are affected by, this variability. Because a system design is effective in one environment is no guarantee of its effectiveness in another. The interactions between a particular system design and a particular environment have to be assessed against the widest possible background of knowledge and experience.

The design of mining systems has, all too frequently, proceeded on the basis of analogy. A proved and effective system in one situation is used as a model for a new situation at the same, or at another mine. An analogy pre-supposes that two compared situations have certain properties in common, although little, if anything, may be known about the correspondence in structure of the two situations. If the function which relates the outcome to the variables is not truly known, it is impossible to know how well, or otherwise there is correspondence between the structure of any two situations.

Only weak inferences, (if any), can be drawn from an analogy. These inferences are, nevertheless, sometimes given a credibility which they would only deserve had they been derived from the manipulation of a model with a structure based on analysis, and/or experiment, and which could be demonstrated to correspond, both statically and dynamically, and in an acceptable degree, to the real structure.

Some of the variables in the physical/geological environment, which have to be considered in developing design specifications for longwall mining are shown in Figure 1. This diagram illustrates how these specifications are linked and interact through the variables in the physical/geological environment. The logic of the systems approach compels attention to the interdependence between mine geometry, mining technique and equipment design specifications and their interactions through the physical/geological variables and through other environmental constraints.

The virtual uniqueness of the physical/geological environment for each particular mining situation implies a need for unique mine design, mining technique and equipment design specifications for each mining situation. This would pose problems for manufacturers who would have wide variations in equipment specification and short production runs. The result would be higher equipment costs and slower deliveries.

However, the specification of more or less standard ranges of equipment is, logically, only conceivable if the mine design can be varied so as to modify the interactions between equipment design specifications and the physical/geological environment.

This proposition has never been fully recognised. Indeed, the usual assumption has been that mining technique and equipment specifications could be developed which would cope adequatedly with all interactions with the physical/geological environment and that mine design had little, if any, modifying effect.

The systems approach underlines the basic fallaciousness of this assumption and, if supporting evidence is required, it is available from the extremely wide performance range obtained from virtually identical sets of equipment operating in different physical/geological environments and, from the frequent failure of a given set of equipment to operate consistently at, or near to, its theoretical potential.

It can rightly be argued that performance variations can, and do, arise as a consequence of many other complex, interacting factors such as differing labour attitudes, skills and practices; different standards of operational planning, supervision, maintenance, supply and transport. However, the development of adverse ground control conditions can have a debilitating effect on these factors. A frequent explanation for the failure of a mining technique to operate consistently near its potential is the development of adverse ground control conditions generated by failure to think through how best these could be modified by planned control of the interactions between the physical/geological environment, mine design, mining technique and mining equipment.

SOME GENERAL OBSERVATIONS ON PROBLEM SOLVING

I have often wished that the term 'Rock mechanics' had not superseded the earlier terminologies of roof or strata or ground control. Unconsciously perhaps, the substitution tends to divert attention away from the primary objective, which is how to exercise control over the physical/geological environment.

The subject will always tend to be regarded as somewhat academic by mine operators unless its study is a response to the need to maintain or improve the efficiency of mines and mining systems. This can only be achieved through the introduction of design changes which enable resources (knowledge, research, men, materials, equipment, facilities, capital, time) more effectively. Effectiveness in the use of resources may be measured by:

a) increase, or decrease, in resource requirements without a corresponding change in the volume or cost of, or profit on, production,
b) increase, or decrease, in exposure to risk,
c) change in relative value measured against stated criteria.

The problems which best respond to a systems approach are those which are, in part, qualitative, or ill-structured, because they contain both knowns and unkowns. In mining, the unknowns are often dominant and, for this reason, require carful analysis. Typical of such problems are:

a) those which are concerned with performance over long periods of future time,
b) those for which there are alternative solutions,
c) those which require large investment and may involve high elements of risk,
d) those which depend on currently incomplete technical knowledge,
e) those which cannot be completely stated as to cost or time requirements,
f) those which are inherently complex because of the combination of resources required for their implementation.

Mines display most, if not all, of these characteristics.

It is not possible to specify a precise level of success in solving mine design problems. One can only try to recognize all the pertinent elements, to structure the problem, state the possible alternative solutions and the risks and uncertainties attaching to them. The mine designer may identify the underlying problem in a different way to the mine operator. If he finds the operator's assessment of the problem is redundant, contradictory or insufficient, he will have to abandon it and make a re-statement. His function may well be, in part, to define the problem.

It should be possible to abstract both the general and the special characteristics of a mine design problem. Firstly by asking whether the design problem is the same as in a previous case, but the environment in which it arises is different. Secondly, whether the environment for the design problem is the same but the problem itself has changed.

A systems approach to a mine design problem conceives to be intrinsically complex even though, superficially, it may appear to be simple. It requires that the problem be divided into its component, but related parts, which can ultimately be formally re-structured for solution. It requires a formal study effort because, to develop the components of a solution, the problem must be understood in terms of its detailed processes and these must be properly related to avoid logical inconsistency.

Some generality must be sought because, without the ability to generalize, complex mine system operations become a divergent set of inputs, processes and outputs, never twice the same; a chaos of causes, results, coincidences, accidents and successful or unsuccessful outcomes.

A system has to be studied within an objective framework if one is to try and identify its characteristics or parameters, and attribute certain properties to them. Because of the quantitative/qualitative nature of many problems of mine design, they are not easily reduced into logical components. It is for this reason that judgment, intuition and experience must play a role as well as the manipulative mathematical techniques, familiar in the solution of purely quantitative (scientific) problems.

In considering quantitative/qualitative problems, it may be objected that it is not possible to be 'scientific'. Of course, experiment with large scale, complex systems can rarely, if ever, be carried out in a research laboratory. One may, however, adopt the Terzaghi "observational approach" i.e. the continuous evaluation of observations and new information for re-designing as needed while construction is in progress. What is important, however, is the attitude rather than the duplication of techniques. In common with scientific experiments, complex systems exhibit:

a) problems that arise from the observation of phenomena and the accuracy of observations,
b) a concern to understand the meaning of information through a knowledge of what it is supposed to describe,
c) the fact that analysis of the problem may raise questions as to the reliability of information and how it is obtained,
d) the fact that if precision and accuracy of information are in question, it will be necessary to question the validity of any conclusions derived from the information,
e) problems caused by improper understanding of cause and effect relationships inferred from a given situation.

Nothing arbitrary, ambiguous or inconsistent is allowed in carefully planned and controlled physical experiments such as those carried out by the scientist in a laboratory. The scientist relies heavily on his ability to plan and control the inputs and conditions of experiment. This control, plus the application of physical laws and the rigour of mathematics enable him to perform his experiments to a desired level of accuracy and equip him to attack certain complex problems from a strictly quantitative base.

Unfortunately, in mine design, as in general systems problems, it is virtually impossible to create the consistent, unambiguous inputs and processes under which the classical, scientific experiment flourishes. To the extent that mathematics, or physical laws, are used, they do not have the same pervasive power in the solution of general system problems. In a general system there are inputs which have only transient states and relationships; the problem is more that of determining how a system relates to a variety of environments.

The task of systems engineering must be different from that of the pure scientist by the very nature of the problems. This does not preclude, however, the use of scientific method. One example is the particular contribution of physicists, mathematicians, biologists, chemists, statisticians and other "pure scientists" in the development of operational research. Originally, this was the application of scientific method to the evaluation and solution of military and logistics problems during the last war and since applied, on a rapidly expanding scale, to a host of "ill structured" business and commercial problems. Their task, as is that of those concerned with problems of rock mechanics and mine design, is largely that of bringing structure into apparently ill-structured problems.

Of first importance is the data available about a problem. Numbers and information constitute data - they are not a phenomenon. To analyze and explain a phenomenon, data must be related, in some meaningful way, to other data and formally presented as an explanation of a problem or, as a solution. The ultimate test is the conclusiveness of the explanation. This rests on the ability:

a) to demonstrate an outcome in advance of its occurrence, or,
b) to predict an outcome that is not demonstrable but which does, in fact, occur.

Whatever the nature of the problem, one temptation has to be recognized and rejected. That is, to simplify the problem until the mathematical intractabilities have been removed, (and all semblance to reality lost); to solve the new simplified problem, and then to suggest that this was the problem which really required solution. No engineer should be so dazzled by the elegance of the mathematical results as to forget that the practical operating problem has not been handled.

PROBLEM SOLVING AND MINE DESIGN

Mine operators must seek to avoid catastrophe arising from complete failure of ground control as typified by the examples given at paragraph 2.6. They also have to minimize the resources required to avoid partial failure of ground control as typified by local roof falls, floor heave, deformation of roadways, rock bursts and bumps, inadequate support systems, etc.

Complete failure of ground control almost invariably arises from the inadequacy of the mine design in relation to the physical/geological environment. That is to say, extraction areas extend until they exceed critical dimensions for stability, or pillar dimensions are chosen which have an inherently low factor of safety or some vital interaction between these parameters and the particular physical/geological environment is overlooked or ignored. Mine design must take into account the structural properties of the rock mass and of mineral pillars (2). But, these structural properties depend:-

a) upon the uniformity, or otherwise, of the while physical/geological environment, and
b) upon the interactions between that environment and the changing dimensions of mining extraction.

These circumstances are dynamic. Consequently, the mine design may have to be changed if the interactions are to remain consistent. Failure to understand this has undoubtedly led to many of the ground control failures experienced in mining.

Partial failure of ground control is frequently attributed to the inadequacy of the support systems underground i.e. of roof bolting, timbering, back-filling, powered supports etc. However, the specifications of support systems underground can be effective if:

a) general control of the main rock mass is attained by adequate mine design, and
b) if the specifications are drawn up so as to take into account the full range of physical/geological conditions which are either known, or may be supposed, to exist.

Changes in the lithology of the roof and floor beds, and in the characteristics of a seam or orebody, undoubtedly influence the effectivess of support systems and must be considered.

Complete ground control failure may involve loss of life, loss of production and even, in extreme cases, the loss of a whole mine. All, or any, of these occurrences are strongly counter-productive. Underground support systems which are unnecessarily complicated or over-designed on the one hand or, partly ineffective on the other, must be avoided. Their effects on machine utilization, on manpower deployment, on day to day safety measures, on non-productive activities and on maintenance are also counter-productive.

In many cases, consideration of support system specifications, and their modification, is left to the mine operator, who adopts an 'ad hoc' approach and rarely conceives these problems in relation to the more general question of main strata control and mine design.

A third question is concerned with mineral conservation, i.e. how to mine out the maximum quantity of mineral from a given seam, or orebody, in a given location, subject to the constraints of maximum safety and minimum cost. It would be counter-productive to increase percentage of mineral extraction if that involved significantly increased risk of failure and/or production costs which, in turn, made the mineral product non-competitive in the market.

The mine operator is the ultimate decision maker and the function of the specialist in rock mechanics and mine design is primarily to advise and to inform the judgment of the operator. See paragraph 2.8.

Advice on problems is of little, if any, value, unless it is implemented and its validity tested in practice. In seeking to achieve such implementation, the advice, and the analysis on which it is based, must be comprehensible to the decision maker. It is not for him to try and raise his comprehension, either mathematically or otherwise, to the level of the specialist. Rather it is for the specialist to present his analysis and advice in terms which are comprehensible to the decision maker.

The evaluation of any problem in rock mechanics or mine design is incomplete without a detailed documentaiton of its purpose, its assumptions, of the methodology used and the conclusions reached. Without such documentation, a clear understanding of the significane and limitations of the conclusion(s) is unavailable. No prudent decision maker would base a major decision on blind trust in the analyst and his conclusions. There is no substitute for a clear understanding of the evaluation to lend credibility (not necessarily agreement) to the results.

The value of highly esoteric mathematics to a mine operator is doubtful. By expending some effort, imagination and thought, the analyst can usually portray complex mathematical relationships in simplified (perhaps graphical) form. No judicious decision maker can be expected to endorse a conclusion or recommendation, the rationale and derivation of which he cannot fully understand. It is the responsibility of the analyst to present the documentation in an appropriate manner. It is preferable to use simple, understandable techniques to arrive at a new optimum recommendation that is implemented, rather than use esoteric techniques to define a precise optimum recommendation that runs the risk of being relegated to oblivion because the responsible decision maker(s) cannot comprehend the rationale underlying the techniques employed.

Conclusions derived by the use of complex mathematics, computers, and so on, may agree with the intuitive judgment of the decision maker and so, be implemented. However, the most valuable and significant evaluations are generally those that indicate the non-obvious or non-intuitive conclusions. These evaluations, if they are to have a high probability of acceptance, must be documented and presented in such a manner that the active evaluation process can be clearly followed.

It is appropriate to comment here upon the use of models, whether these are mathematical or physical. Models are, to some extent, a substitute for directly relevant experience and it is, of course, essential to supplement direct experience with analysis. Without any deprecation of the use of analytical models, the reality with which we deal is exceedingly complicated and dynamic. To construct models that are simple enough to be handled, even by our most sophisticated analytical or computational devices, it is often necessary to make unproved - and therefore risky - abstractions. For this reason, no all-purpose model can be relied on. Instead, an analysis may have to use many different kinds of model of widely varying technical elegance, complexity and size. Neglect of the qualitative factors, and over-emphasis of the calculations is misleading and may be dangerous.

Decisions cannot, therefore, be the automatic consequence of a computer program or of an application of model analysis alone. For complex problems, intuition and judgment must continue to supplement systematic analysis and as the questions become broader, this must happen to an increasing extent. To make such judgment and intuition most effective is the primary purpose of the systems approach.

UNCERTAINTY AND RISK

In this section I have drawn freely from ideas expressed in Casagrande's classical paper (3) on uncertainty and risk. Although concerned primarily with earthwork and foundation engineering, the conclusions he reached have an application to engineering generally. He suggested that the proper use of "calculated risk" should involve the two following distinct steps:

a) the use of imperfect knowledge guided by judgment and experience, to estimate the probable ranges for all pertinent quantities that enter into the solution of a problem.
b) the decision on an appropriate margin of safety, or degree of risk, taking into consideration economic factors and the magnitude of losses that would result from failure.

In those two sentences Casagrande adopts a basically systems approach with careful emphasis on the imperfections of knowledge, the value of judgment and experience, the representation of uncertainty by probable ranges for quantified parameters, the element of decision taking, the specification of safety margin or degree of risk, and the economic or cost implications of failure.

Perhaps I may make one more quotation from Casagrande's paper. "If judgment based on empirical knowledge played an important role in such (major design) decisions, why not admit it? If unknown risks or calculated risks were involved because of the large gaps in our understanding (of the mechanics of soils), why not admit it? Authors who, a posteriori, try to rationalize their decisions or make theories fit the facts, merely reveal their limited grasp of the realities (of applied soil mechanics), they hinder rather than promote progress; and they mislead their younger and less experienced colleagues".

In relation to the physical/geological environment, mine design is applied rock mechanics. But, a mine is a structure in rock and its design is not complete until the end of the life of the mine. The margin of safety incorporated in the structural design of a mine has to recognize the full range of uncertainties involved and must bear a direct relationship to the magnitude of the potential losses, both in terms of cost and human life.

Only if the range of uncertainty is relatively small can it be expressed in the conventional terms of a numerical factor of safety. When the range of uncertainty is large, we should speak of a 'margin of safety' which will be based primarily on experience and judgment.

CONCLUSIONS

Mine design or, applied rock mechanics in mining, is concerned fundamentally with design specifications for excavations and support systems in rocks. Such specifications have to take full account of the whole physical/geological environment and of the fact that a mine is a developing structure which cannot be complete until its life is at an end. The primary objectives are avoidance of total, or partial, ground failure consistent with the optimum efficiency of mining operations. The designer of mines must, inevitably, be concerned with uncertainty, risk and the economic implications of his activity.

This paper adds nothing to the theory of rock mechanics, profers no experimental results and conclusions, describes no field observations and advertises no successes, or failures, in mine design. It was not intended to do so. The most I hope for is that it may contribute a little to the mutual understanding between those who have the problems and those who seek to assist in solving them.

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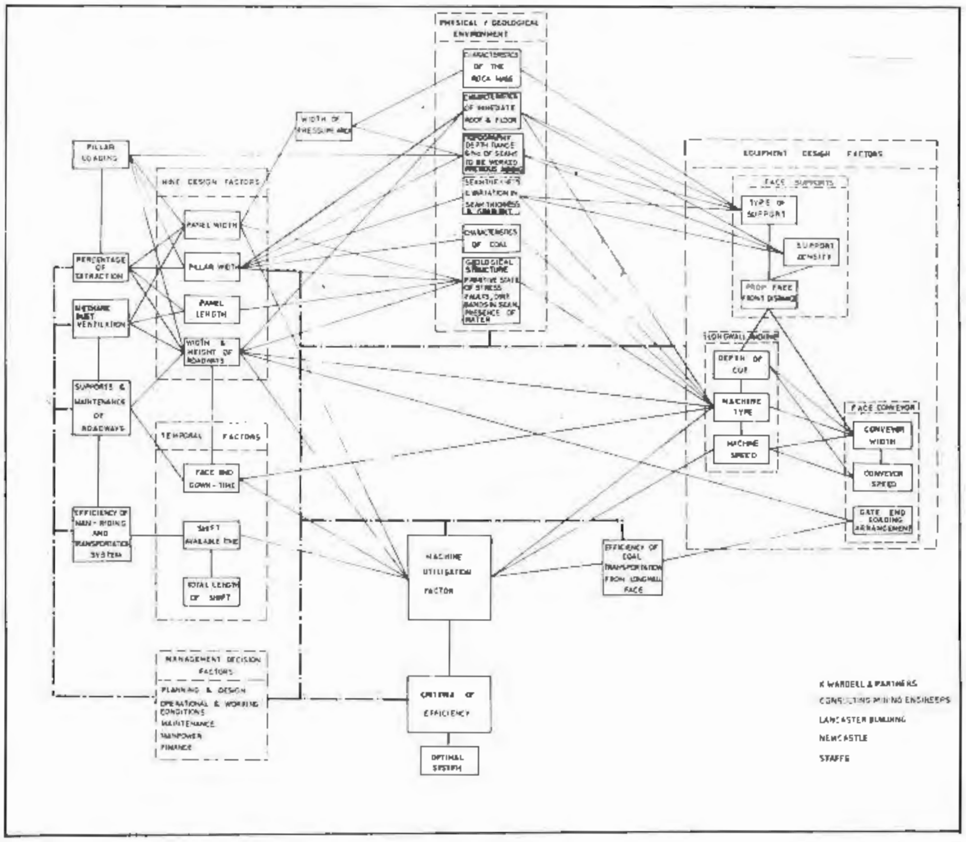

Figure 1. Longwall system concept

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