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Rock Mechanics Instrumentation for Mine Design

Proceedings: Bureau of Mines Technology Transfer Seminar; Denver, Colo.; July 25, 1972

Compiled by the Technology Transfer Group,
Office of the Assistant Director–Mining, Washington, D.C.

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Washington, D.C.

ABSTRACT

This report contains the papers presented at the Technology Transfer Seminar on Rock Mechanics Instrumentation for Mine Design held by the Bureau of Mines on July 25, 1972, in Denver, Colo. Seminar attendance consisted of representatives from coal, metal and nonmetal mining industries, universities, mining consultants, instrument manufacturers, and other Government agencies.

The purpose of this meeting, the first in a series of seminars, was to present the philosophy and mode of operation of the Technology Transfer Program, to give an overview of some of the Bureau's progress in mine structural design methods, and to describe their latest instrumentation developed for rock mechanics investigations in coal and metal mines. The report includes the opening remarks by the Bureau of Mines Deputy Director--Mineral Resources and Environmental Development, who discussed the new Technology Transfer efforts; three papers on mine design; four papers on instrumentation; and a summary.

INTRODUCTION

The Technology Transfer Program is designed to move newly developed coal mine health and safety research technology from the Bureau's research laboratory and field test stages into the coal mines where it can be used. While the Bureau has always been active in technology transfer through its technical publications and through cooperative research with mining companies, this new program is aimed at insuring that all potential users (coal mine operators) are fully informed about what newly developed technology is available and how it can be utilized to solve their mining problems.

Technical papers presented herein discuss mine design philosophy, basic considerations for design, and the use of rock mechanics instrumentation and structural analysis. Devices for determining the in situ stress and elastic constants of mine rock, rock pressure changes, and displacements are described in detail.

The need for further meetings and cooperation between the coal mining industry and the Bureau is discussed as a way of accelerating the application of research results generated under the Coal Mine Health and Safety Research Program.

OPENING REMARKS

by

T. A. Henrie[1]

On behalf of the Bureau of Mines, I would like to welcome you to our first Mining Research Technology Transfer Seminar. We are gratified that so many of you have taken the time to attend. We feel that the material to be presented here today is significant to the problem of coal mine health and safety, and I am sure that as a seminar it will fulfill its obvious purpose, that of exchanging knowledge, in some cases the results of original research, by lectures, reports, and discussions. This is the essence of Technology Transfer.

When Congress enacted the 1969 Coal Mine Health and Safety Act, it initiated what is probably the most extensive and comprehensive industrial health and safety program ever undertaken to promote the welfare of a single class of industrial worker. It is a many-faceted program, involving promulgation and enforcement of regulations, technical support, worker education and training, and a broad-based program of research. Unstated, but implicit, is the fact that the mineral commodity involved is our Nation's most abundant and important energy mineral resource.

The first intent of the research portion of the program was to conduct such studies, research, experiments, and demonstrations as may be appropriate to improve working conditions and practices in coal mines and to prevent accidents and occupational diseases originating in the coal-mining industry. When the Bureau received the first funds from Congress, we immediately began the research most urgently needed to provide knowledge of improved working conditions and practices and to prevent accidents and diseases. To date the Bureau has received $65 million for such research and has spent $40 million. As those of you who have been associated with research can well understand, organizing and initiating such a massive program was a Herculean task; and we in the Bureau, while admitting our lack of a crystal ball to pick guaranteed research projects, are very pleased with the results and accomplishments which are emerging from the research pipeline.

These accomplishments themselves, however, present a problem to those of us who share the commitment to effect the desired change. How do we bridge

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[1]Deputy director--Mineral Resources and Environmental Development, Bureau of Mines, Washington, D.C.

the gap between research and practice? The problem of getting research into practice is difficult, wherever the research setting. In my 21 years in metallurgical research we faced this problem repeatedly: How could we help industry to recognize the benefits we knew would be the result of the use of our research?

Congress, through the act, was quite specific in its insistence that this research must lead to improvements. The Bureau, however, does not mine coal for commercial use. Since we are not users of our technology, we had to find another way to see that this research produced the results Congress intended. The Bureau is neither the first nor the last Federal agency to face this problem. We were fortunate, however, in being able to benefit from the growing experience of other Federal agencies. Surveying their experience and our own experience since the formation of the Bureau in 1910, it became clear that it would take a concerted effort on the part of all concerned if the American coal miner is to benefit. We decided to initiate such a "Technology Transfer" program.

Technology Transfer is more than a fashionable combination of words. Indeed, it is probably unfortunate that it is so fashionable, since today's fashions so often become yesterday's fads, and such faddishness would reflect unfavorably on what has been, is, and will continue to be, an excellent concept.

To digress for a moment, the Federal Government has always been considered to be the natural agent for performing either that research too complex for private initiatives or research with high social value which cannot be capitalized upon by our economic system. Since the Second World War, the Government's role has increased with society's expectations. As pointed out by the recent Brookings Institution Report on "Setting National Priorities," for a while "the idea persisted that if one could identify a problem and allocate some Federal money to it, the problem would get solved." The actual situation, as it concerns the solution of social problems, suggests that this technique has a rather low assurance of success.

Success, based on our survey, seemed to be associated with active programs to insure the followup of research. The Agricultural Extension Service, for example, has a recognized positive impact on farming because it stayed with its research results right to the fields to see that farmers had every opportunity to make use of that Agency's technology--in a phrase, Technology Transfer.

There are two types of transfer that make up the basis of the technology transfer concept: horizontal and vertical. The horizontal transfer, by which we mean taking technology developed for a particular use and applying it to some unrelated need by means of adaption, has perhaps received more publicity because of the emphasis given it by DOD and NASA, is part of our plan. This type of transfer is beneficial because it increases our funding leverage. The primary emphasis for the Bureau, however, as for the Agricultural Extension Service, is on vertical transfer, or moving the technology from the laboratory into its intended use. This is the logical direction for our effort and the one we are best equipped to pursue by virtue of congressional funding. We intend to do everything possible to assure that the proven research results from our own research centers and our contract research program are successfully implemented in our Nation's coal mines.

The technology process starts with the dual tasks of an inventory of research and an identification of need. Ideally there should be a one-to-one match. Realistically, we expect to find needs for which we have no research and research that meets no need. As these mismatches are discovered, they will go back for reprograming. The matches will pass at appropriate times into the technology transfer process. The resultant output should then be ready for any or all users. We are hopeful that there will be early research results for each of the users. You who are coal operators will hear of the Bureau's instrumentation for mine design at this seminar today. For enforcement, we have identified some promising avenues through our contract on "Industrial Hazards Associated With Underground Coal Production." Technical support will soon have available to them the much needed engineering guidelines and criteria for illuminating the working face.

A critical component of the Technology Transfer Process is the identification of need. This task cannot be done without your help. Perhaps more than in any other area, identification of need is and must be a partnership effort. Mine operators, inspectors, and technical support personnel must communicate with mining research people in open, honest dialogues. Effective dialogue should not occur only under emergency conditions or under crash program contingencies. Information withheld or excessively filtered is of no value to the researcher. A good working partnership compensates for the weaknesses of both parties and capitalizes on their strengths. The Bureau and the Industry need that kind of partnership.

Inventory of research tasks requires a critical mind and honest questioning. Again this job needs your best efforts and understanding. This inventory is not a one-shot effort. It must be a continuously ongoing process that receives responsible input from those with an understanding of the problems involved.

Initially, the technology transfer program will concentrate on Bureau of Mines research. Hopefully, we will soon be able to accommodate the R&D effort within the industry and by outside groups. Much valuable work has been performed by industry, and we feel that our mining research organization can expedite validation of industry results.

There are two basic technology transfer mechanisms--convince and require. The talents needed to successfully accomplish transfers are, of course, considerably different for these two basic methods. From our results to date, we are convinced that successful technology requires marketing skills, and it is this approach that we think the bulk of our efforts must take because of the nature of most of our research and the structure of the act. In other words, we would rather convince than require. To date, the Technology Transfer Group has identified a number of ways the Bureau can improve its research use record. With seminars, we will try to encourage face-to-face dialogue with those who can best use our technology. As a result of this dialogue, we will initiate the in-mine demonstrations that will verify the improved or new technologies to make the mines safer and more healthful places to work. These demonstrations will be carried out under cooperative agreements strategically selected to provide a comprehensive data base. If warranted and if desirable to do so, enforcement can then use this data base as support to promulgate new or alternate standards.

This seminar is but one of those we expect to have to help us in our task of bringing our research into the mines. The next seminar will address itself to the problems of plugging oil- and gas-well-penetrating coalbeds. We also expect to hold additional seminars and presentations on illumination, methane control, hazard warning devices, and other subjects from our research program in the ensuing months.

During this seminar and those which follow, you will hear repeated requests for two elements vital to this program--your cooperation in joint demonstrations and a discussion of your needs. We sincerely hope that you will help us help you.

Thank you.

PHILOSOPHY OF DESIGN

by

Leonard A. Obert [1]

Compared with the time that man has been mining underground, the concept of designing a mine is a relatively recent innovation. What is meant by "designing a mine?" An engineer considers rock as a structural material with measurable properties; a mine is a structure in a rock continuum amenable to structural analysis, and hence to prior design. In the past, mines have been developed on a more or less cut-and-try basis, taking into account previous experiences in similar deposits. Although many mines have been developed and operated on this basis, this approach has had its problems. In the past few decades there have been a number of catastrophic failures involving the total collapse of a mine structure. In addition there have been many lesser failures that have created hazards and loss of production.

Although safety is the overriding factor there are other reasons for employing good design. The Bureau of Mines is charged with conserving the Nation's mineral resources, and in mining conservation implies that the maximum amount of mineral is recovered from a deposit compatible with safety and restoration of the environment (more on this in a moment). Unfortunately, conservation and safety generally work at cross purposes; that is, the higher the recovery from a deposit, the less stable the structure. Seen in this light good design becomes increasingly important.

Another factor that is of concern to not only the Bureau of Mines but the public at large is the preservation and utilization of the surface over mined areas. Surface mining can be effected safely with an almost complete recovery of the deposit, and in most instances the surface can and should be restored so that a minimum scar remains. However, many deposits are too deep to mine by surface methods. If the surface is to be reasonably preserved there are two acceptable possibilities--a total extraction of the mineral deposit, which will result in an almost immediate and complete subsidence of the surface over the mined area, or the use of a mining method that will create a structure which will remain stable indefinitely and with only a minimum disturbance to the surface. The worst and unacceptable situation is to create an underground mine that is not stable in time, so that subsidence continues over many years until the structure finally stabilizes by filling its own voids. Both total

[1] Research physicist, Office of Science Advisor--Mining, Bureau of Mines, Denver, Colo.

mining or mining in such a manner that subsidence is indefinitely limited are problems that require good design.

In the early 1940's the Bureau of Mines initiated a pioneering program to develop a rationale for designing or evaluating the stability of mines or other underground excavations. This program included the development of standardized tests for measuring the mechanical properties of mine rock and theories for analyzing the stability of mine structures. Most of the early studies related to hard-rock mining, and these investigations have been reported in Bureau of Mines publications. Since the passage of the 1969 Coal Mine Health and Safety Act an accelerated program concerned with coal mining has been in progress.

The problem of designing a mine is very different from that of designing a conventional structure such as a building or a bridge. In the latter case the designer can specify materials with specific properties, such as steel or concrete, and he can expect that the materials used in the structure will meet this specification.

There is no equivalent procedure for mine design. Before underground access is possible the only source of information regarding the properties of the rock is from observations and tests on exploration cores. However, exploration cores provide only a very small sample of a mineral deposit, and large bodies of geologic materials can be expected to contain many anomalies. Hence, a design made on the basis of core studies can only be considered preliminary.

After underground access is possible and especially during the development phase of a mine, information regarding not only the rock properties but the state of stress in the rock as well can be obtained. As this information becomes available the design can be modified accordingly--and thus the rationale is to design-as-you-go. This approach is not new to engineers working with geological materials. Terzaghi advocated this procedure in the treatment of soil problems and, to some degree, in the treatment of rock tunneling problems.

Finally and most important, a design rationale cannot be developed unless the factors entering into the problem can be reduced to numbers. That is why instrumentation is so important to this problem. With proper instruments and procedures, the mechanical properties of the rock, the state of stress in the rock, and strains of deformation in components of the mine structure can be measured. Moreover, today measurements can be made not only on the surface of mine openings, but in the surrounding rock as well.

In this seminar a number of the instruments and procedures used for making such measurements will be described, and a forthcoming seminar, scheduled for the 5th and 6th of December, will consider the utilization of such data for designing or evaluating the stability of mines.

BASIC REQUIREMENTS FOR THE DESIGN OF MINE OPENINGS

by

Brian T. Brady [1]

ABSTRACT

Knowledge of the basic data related to rock properties is required for the design of openings in rock. Suitable theoretical and/or numerical procedures that will enable the design engineer to incorporate the basic data into a working model of the proposed opening, such as the finite element method, must be available. Ongoing field measurements during the mining operation will provide a check and, when necessary, will provide for a modification of the working model.

Applications of these basic data by the Bureau of Mines to the Galena silver mine, Wallace, Idaho, and the Colony oil shale mine, Rifle, Colo., are discussed.

INTRODUCTION

The main objective of mine design is to obtain the maximum extraction ratios under safe conditions. With the present status of rock mechanics, exact predictions as to the behavior of a mine cannot be made and approximate idealizations must be used. Realistic design of excavating in rock can be usually attained only after the in situ measurements in conjunction with laboratory studies and are used to predict the behavior of the rock mass. Certainly the state of the art of rock mechanics has now reached the point where the tools and techniques necessary to solve many problems regarding mine stability have been developed and are now available (3).[2]

BASIC CONSIDERATIONS REQUIREMENTS FOR THE DESIGN OF OPENINGS IN ROCK

To design an underground structure in rock or to evaluate the stability and safety of an existing structure, we must first determine (1) the deformation and/or stresses in the structure resulting from the external loads applied to the structure; (2) the physical properties of the rock in the vicinity of the opening; (3) the ability of the structure to withstand the applied

[1] Physicist, Denver Mining Research Center, Bureau of Mines, Denver, Colo.
[2] Underlined numbers in parentheses refer to items in the list of references at the end of this paper.

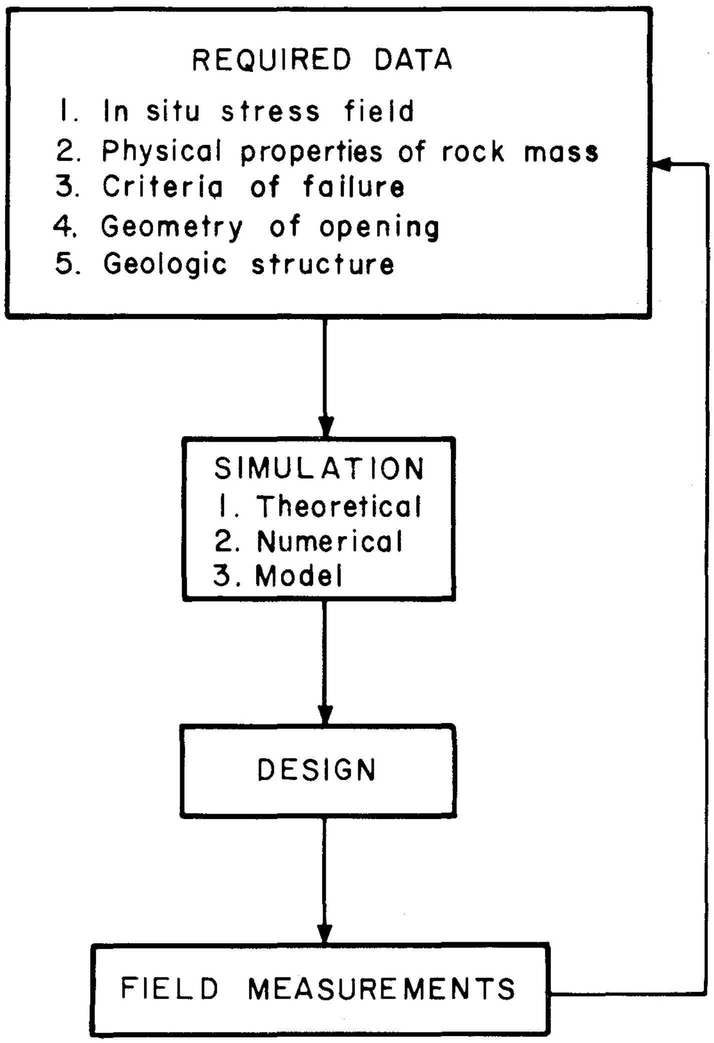

FIGURE 1. - Basic Requirements for Mine Design.

stresses or deformations; that is, reliable criteria of failure for the rock must be available; (4) the geometry of the opening; and (5) the regional geology and its influence on the stress and displacement distribution in the vicinity of the opening. The application of these basic data requires the use of suitable analytical methods, model studies, and/or numerical procedures, such as the finite element method, to incorporate this information into the initial design. It then becomes necessary to evaluate this design by full-scale field testing, and then correlating the field data with predicted results. Only by repeated feedback of field data into the input data and the design process can reliable design techniques be developed. Fortunately, the development of a mine is a slow process, so that there is usually an opportunity for experimentation, and design modifications can be effected as the results of experiment and experience warrant. Figure 1 illustrates the principals of this procedure.

Knowledge of the stress field existing prior to mining is essential in numerically simulating the mine design. Measurements of in situ stresses have shown that in many cases the measured stresses are anomalous, in the sense that the horizontal stresses cannot be attributed to gravity loading, even when allowances are made for variations in surface level.[3] This anomaly is believed to be a result of tectonic stresses and/or structural stresses.

Structural stresses are caused by inhomogenities of the region under consideration. Since mines commonly involve rock of different physical properties than the country rock, such effects are to be expected. Tectonic

[3]Under conditions of gravity loading, the vertical stress is calculated by knowing the depth of the structure from the surface and the density of the rock. The value of the horizontal stresses is determined once Poisson's ratio of the rock mass has been evaluated (fig. 2). This calculation assumes the rock mass behaves linearly elastic.

stresses can be due to a variety of causes such as mountainous terrain in the vicinity of the mine structure. Generally, tectonic stresses affect both the vertical and horizontal components of stress and the principal direction of the stresses will be in some direction other than the horizontal or vertical. Field measurements (fig. 2) of the in situ stress field indicate that, in general, the stress fields are triaxial--that is, there are three unequal principal stresses--and that the principal stresses are not oriented in the vertical and horizontal directions.

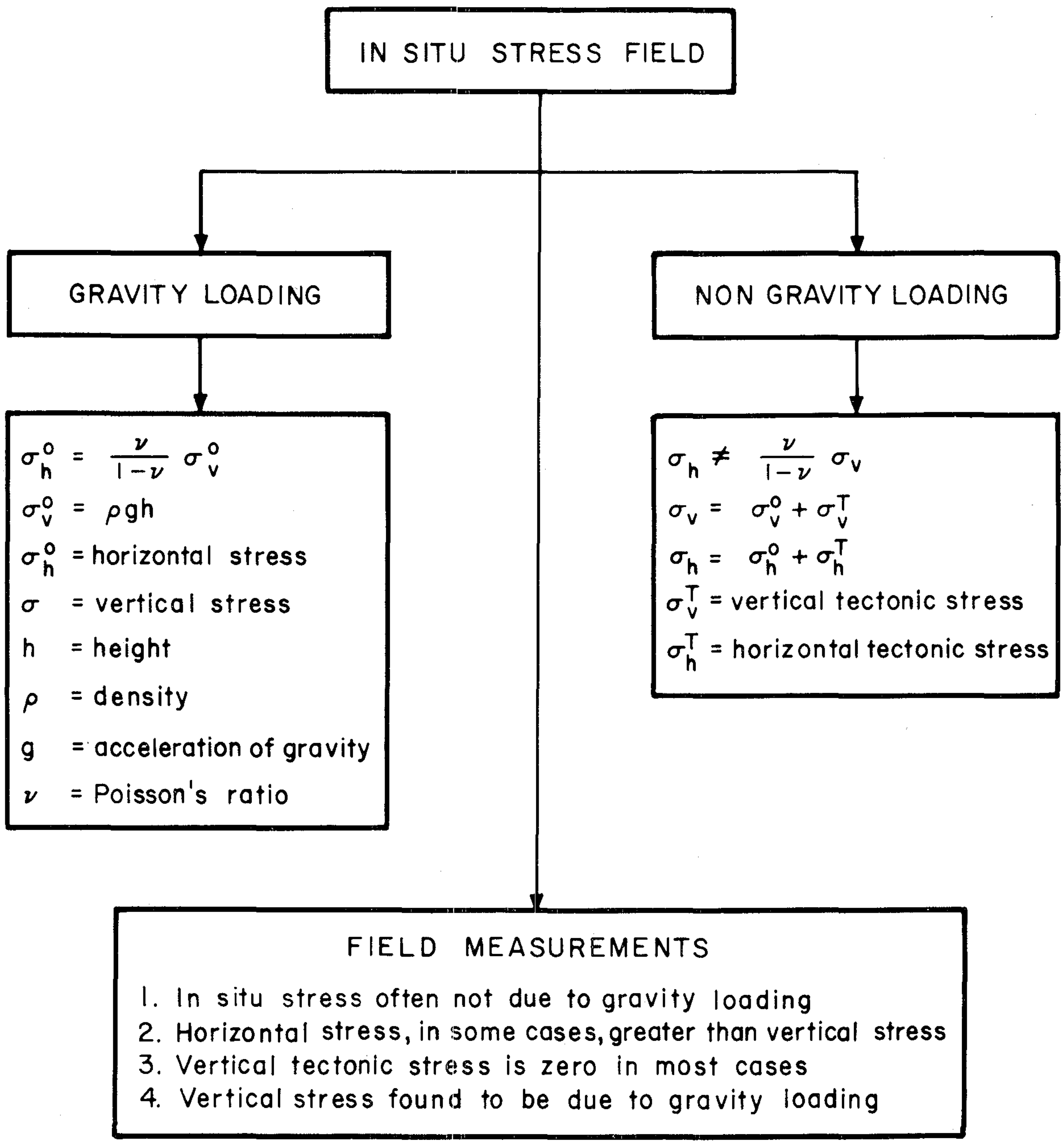

FIGURE 2. - In Situ Stress Field Measurements.

The determination of both the magnitude and direction of the in situ stress field plays a key role in design of mine openings and, in particular, in the geometry and placement of the opening with respect to the applied stress field. An underground opening will produce a stress concentration in the surrounding rock and if the stress in the surrounding rock exceeds its strength, the opening will fail either by fracturing or by deforming more than some tolerable limit. Thus, the problem generally reduces to not only determining the shape or geometry of the opening that will minimize the critical stresses or closure rate, but also to developing a criterion of failure for the rock in the vicinity of the opening. The effective design of engineering structures in rock thus points out the need for reliable measurements of the in situ stress field. The Bureau of Mines has long recognized this problem and has developed reliable tools providing these measurements. These methods are discussed at length in these proceedings.

Realistic design of structures in rock requires knowledge of the in situ physical properties. While some knowledge of the physical properties of the rock can be gained from measurements on laboratory specimens of rock, it is usually necessary to determine the properties of the whole rock mass affected by the structure. These properties are markedly dependent upon large-scale structural features such as bedding and jointing. Pore and joint water pressures also have a marked influence on the properties of a rock mass. These geologic factors must be evaluated by field investigations such as geologic mappings, drill cores, and seismic surveys. In some instances, it may be necessary to make in situ measurements of the physical properties using plate-bearing tests and large-scale compression and shear tests. Deformation moduli of rock in the field can be measured in a number of ways, including flat-jacks, used to find the Young's modulus of the rock mass near any free surface, and plate-bearing tests, used to determine both strength and Young's modulus. Other in situ tests, such as large-scale compression and shear tests, have been developed by the Bureau of Reclamation to measure physical properties of the in situ rock mass. However, such tests are both difficult to perform and are quite expensive. The Colorado School of Mines borehole deformation gage, discussed at length in these proceedings, provides a partial solution to determining the physical properties of the rock mass in that it is both simple to perform and inexpensive to operate. This gage provides an estimate of the shear modulus of the rock mass in the vicinity of the structure. Once the value of Poisson's ratio is available, the Young's modulus can be calculated. Since the Poisson's ratio of rock masses usually lies between 0.10 and 0.25, the Young's modulus of the rock can be calculated accurately.

Thus, in the engineering evaluation of rock structure design problems, the procedures discussed above form the basis for assigning numerical values to at least a part of the problem variables, and in this respect, structural rock mechanics is no different from any other phase of engineering. However, because of uncertainties and unknowns in these variables, and in particular, because of the effect of time on the physical properties of the rock mass, a safety factor should be employed in the initial design of the structure. The safety factor should be consistent with past experience. Data from field tests as the mining advances provides the necessary feedback information for modifying these variables and possibly, for revising the initial safety factor value.

APPLICATIONS OF MINE STRUCTURE DESIGN

The primary objective of the Bureau of Mines ground control research program is not just to develop a design capability within the group itself, but to demonstrate that these design techniques are a logical and reliable technology that can be transferred to the mining industry by means of training and education in the mining schools, and by demonstrating to consultants and mine engineers that the various techniques and instrumentation provide reliable information that can be used by mining companies to solve their many, varied rock mechanics problems.

We present in this section two examples of this research philosophy. These examples clearly demonstrate the basic requirements for mine design outlined earlier.

Rock Burst Mechanics at the Galena Mine, Wallace, Idaho

The Galena mine is located in the Coeur d'Alene mining district in northern Idaho. The total reported production of the district since the late 1880's is in excess of $2 billion. Silver, lead, and zinc are the predominate metals mined from veins in the Belt formation series of Precambrian quartzites and argillites. These formations have been highly folded and faulted and most of the bedding is steeply dipping. Veins generally are oriented at an acute angle to the bedding. A horizontal cut-and-fill mining system is predominantly used, with the mill tailings being pumped back into the mine to become the floor for the succeeding cut as stoping proceeds from a lower level to a level 300 feet above. Figure 4 shows a cross section of a typical stope in this mining system.

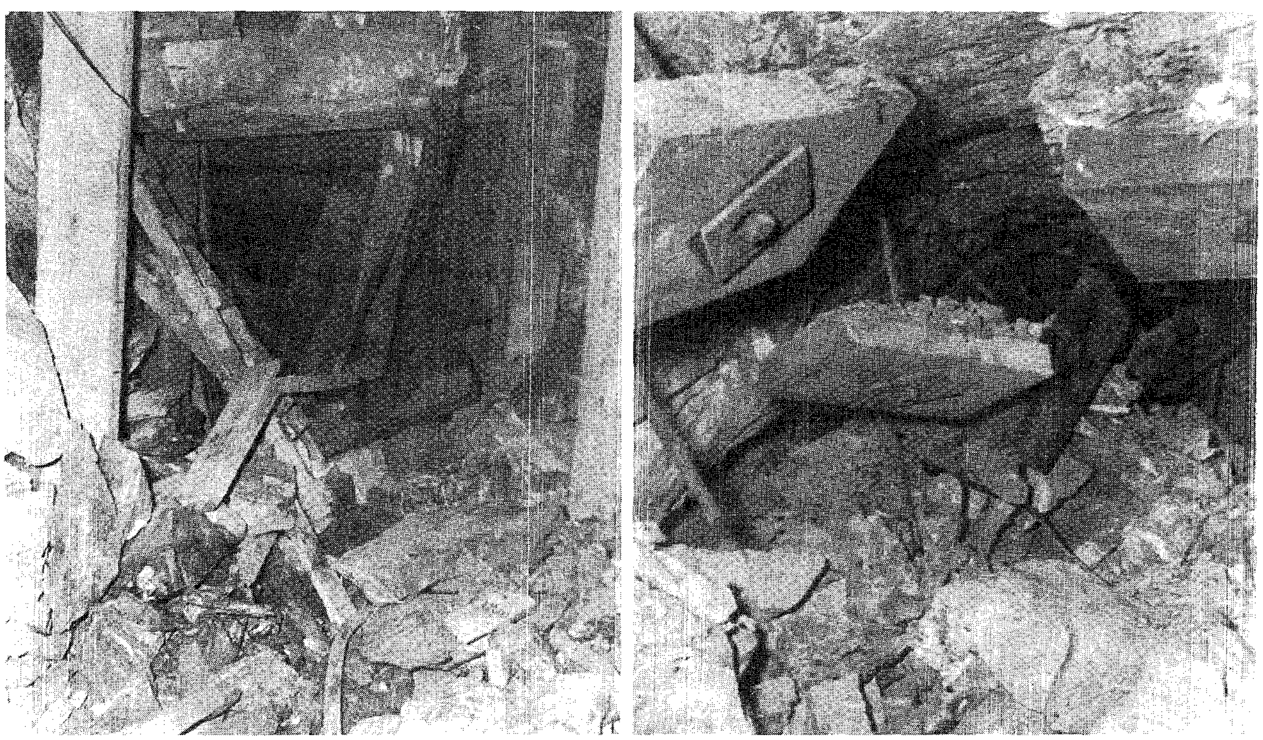

FIGURE 3. - Typical Results of a Rock Burst in Stope and Along Sill Drift in the Galena Mine.

Rock bursts in the Galena mine have occurred from depths of 2,000 to 4,300 feet below the surface and are the hard crystalline quartzite. Figure 3 shows typical results of a rock burst in the stope and along the sill drift over the pillar. This damage is typical of that resulting from a pillar burst and usually occurs when the pillar attains a critical size. To gain insight into the stress distribution around a stope pillar, finite-element stress-analysis computer model studies were made of the mining method (1). Figure 4A shows the cross section of a typical stope and 4B, the computer model idealization. Material properties and local geology for incorporation into the models were based on laboratory studies (1) and on field measurements. The finite element models were loaded by applying uniform displacements to their models, calculated to produce a 7,500-psi hydrostatic stress field. This stress field value was taken to be the state of stress in the vicinity of the silver vein on the 3,700-foot level based on in situ stress relief measurements by the Bureau of Mines both within and away from the influence of mining. Because of the geometry of the silver vein, a plane section across the vein was taken to be representative of the way in which an actual stope is subjected to load. To simulate the mining method, the model stope was mined in a series of 15 10-foot cuts. Each simulated mining cut was backfilled with compacted sand, and blasting fractures were modeled in the adjacent walls and back prior to beginning the next cut.

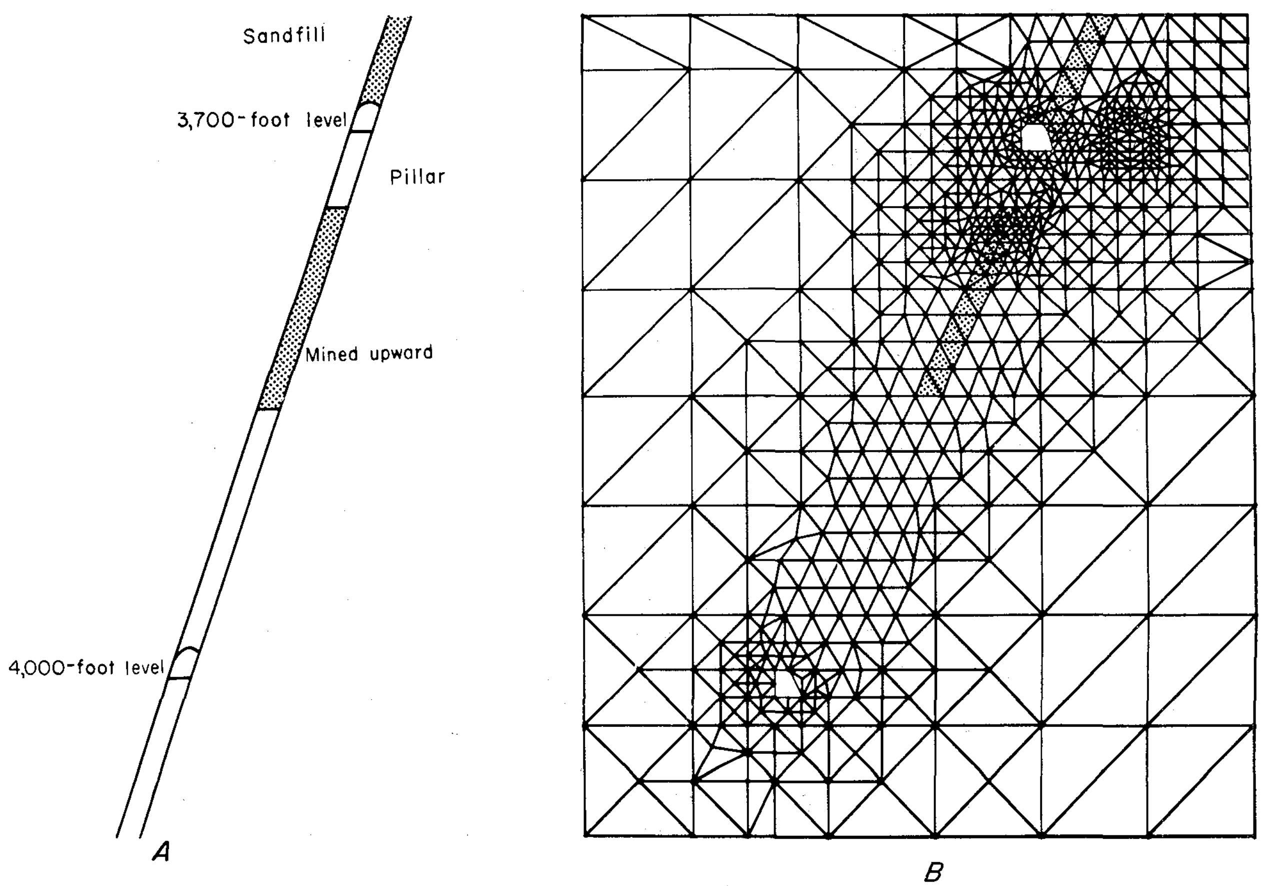

FIGURE 4. - *A*, Cross Section of Typical Stope and *B*, Computer Model Idealization of the Stope.

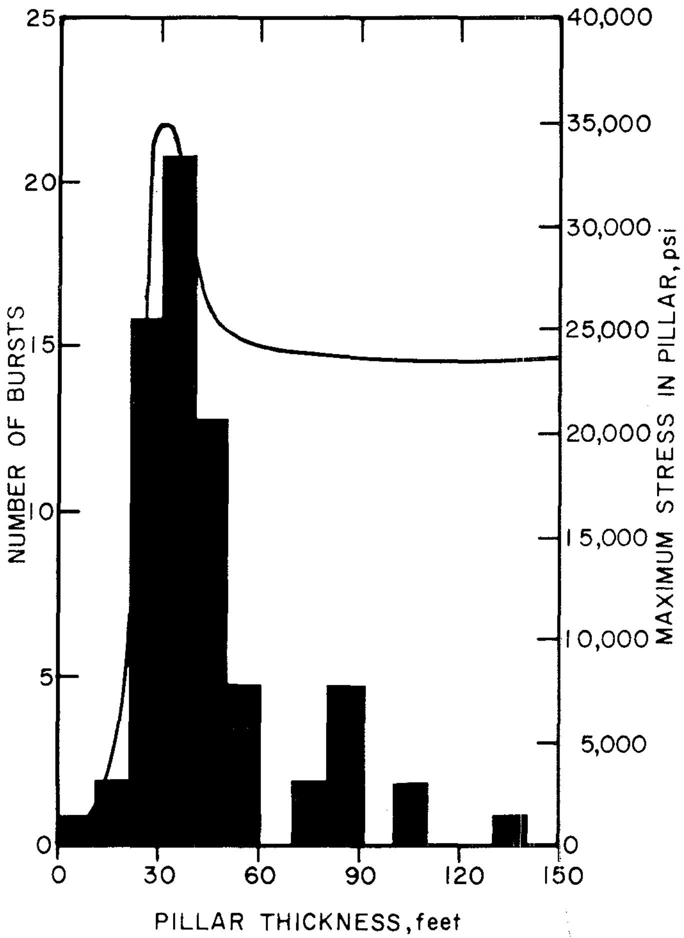

FIGURE 5. - Number of Bursts and Maximum Pillar Stress Versus Pillar Thickness.

Results from this simulated mining model are shown in figure 5, where the maximum principal stress in the pillar is plotted against the pillar thickness; for comparison, the number of damaging rock bursts at the Galena mine since 1958 is also plotted against pillar thickness. Note that the stress in the pillar does not significantly begin to increase until the pillar thickness is reduced to some 50 feet, coinciding with the onset of pillar bursting. The magnitude of the maximum principal stress for the actual pillar thickness of 30 feet is 35,400 psi (compression), a value which exceeds the average unconfined compressive strength (25,300 psi) of both vein and wall rocks. Clearly, failure of this pillar is to be expected. Also, the magnitude of the maximum stress in the backfill was found to be only 850 psi, from which it can be inferred that sand filling does not carry enough load to reduce the critical high stresses created in the pillar, and hence has little effect on reducing the evidence or severity of bursting.

This information suggested that destressing or softening of a pillar by blast-induced fractures would be effective in controlling pillar blasting, since softening the pillar will decrease its ability to maintain load, and accordingly the stress in the pillar will be reduced. Finite-element model studies were made of this process. Destressing was simulated in the model by reducing the elastic modulus of the vein by a factor of five. Model results for the critical 30-foot pillar indicated that this modulus reduction would result in approximately a fourfold reduction in the maximum stress in the pillar. Thus, fracturing of a pillar by destressing allowed the pillar to yield, releasing energy, resulting in a lowering of the stress within the pillar; that is, destressing results in more load being applied to the surrounding structure. Based on these computer model results, the destressing of rock-burst-prone pillars was judged to be an effective means of controlling or eliminating pillar bursting. Destressing of a burst-prone pillar at the Galena mine was undertaken based on these results.

The pillar was to be destressed by blasting a series of long holes, 1-7/8 inches in diameter, drilled in the footwall of the vein on 5-foot centers.

These holes were loaded with ammonium nitrate prill to within 5 feet of the sill and 3 feet from the collar, and fired with millisecond delays.

To determine the effectiveness of the destressing test, microseismic monitoring and seismic velocity surveys of the pillar were carried out. The source locations of rock noises generated following destressing would indicate if the pillar were still under a critical stress and where the stress had shifted if destressing were successful. Since seismic velocity is related to the elastic modulus of the rock, any decrease in seismic velocity would indicate a decrease in elastic modulus, a softening of the pillar, and hence a reduction in stress.

Prior to destressing, a seismic velocity survey was carried out through the stope pillar. Seismic velocity survey shotholes were loaded with a half stick of 45-percent dynamite and a shot-time target, and detonated. These blasts were recorded, the seismic velocities from each shot location to each geophone were computed, and a seismic velocity contour map of the pillar was compiled (1). Since destressing, the pillar has been mined in three 10-foot cuts by the normal breasting-down method. No difficulties were encountered in the form of crushed or slabby rock. Continued seismic velocity surveys through the pillar showed that a small increase in seismic velocity occurred when the pillar was reduced to 20 feet thick, indicating that the stress in the pillar had increased owing to its reduced area. When the pillar was reduced to 10 feet, seismic velocities decreased significantly, indicating that the pillar was now too fractured to support any additional load. Further closure of the sill drift and the lack of rock noises generated following blasting confirmed this observation. The stope was mined to completion without incident.

Strength and Stability of Rock Pillars

The analysis of the strength and stability of mine pillars is a problem that has long interested the Bureau of Mines. Several recent significant

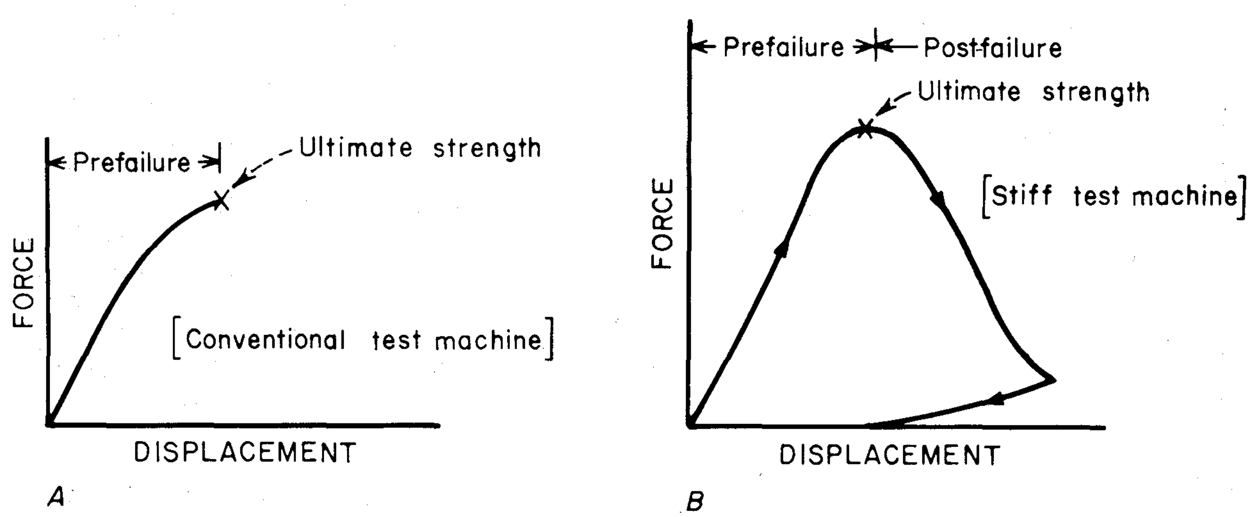

FIGURE 6. - Force-Displacement Behavior of Rock in *A*, Conventional and *B*, Stiff Loading Machines.

concepts related to pillar stability and failure have resulted from tests conducted on model rock pillars in a stiff loading machine. Earlier investigations of the strength and deformational behavior of rock pillars have centered on the prefailure characteristics of solid rock (fig. 6). These studies were confined to the prefailure region owing to testing machine limitations. For example, most rock pillars characterized by brittle fracture fail violently when deformed to ultimate strength in a conventional testing machine, since a fixed displacement rate applied to the rock cannot be maintained at ultimate strength. The use of stiff testing machines where a fixed displacement rate can be applied to the rock pillar has shown that brittle rock can be deformed in a controlled manner beyond ultimate strength into the post-failure region (fig. 6). Because a stiff loading machine is known to model loading conditions found in most underground room and pillar mines, it is logical to study the behavior of rock under these conditions (2).

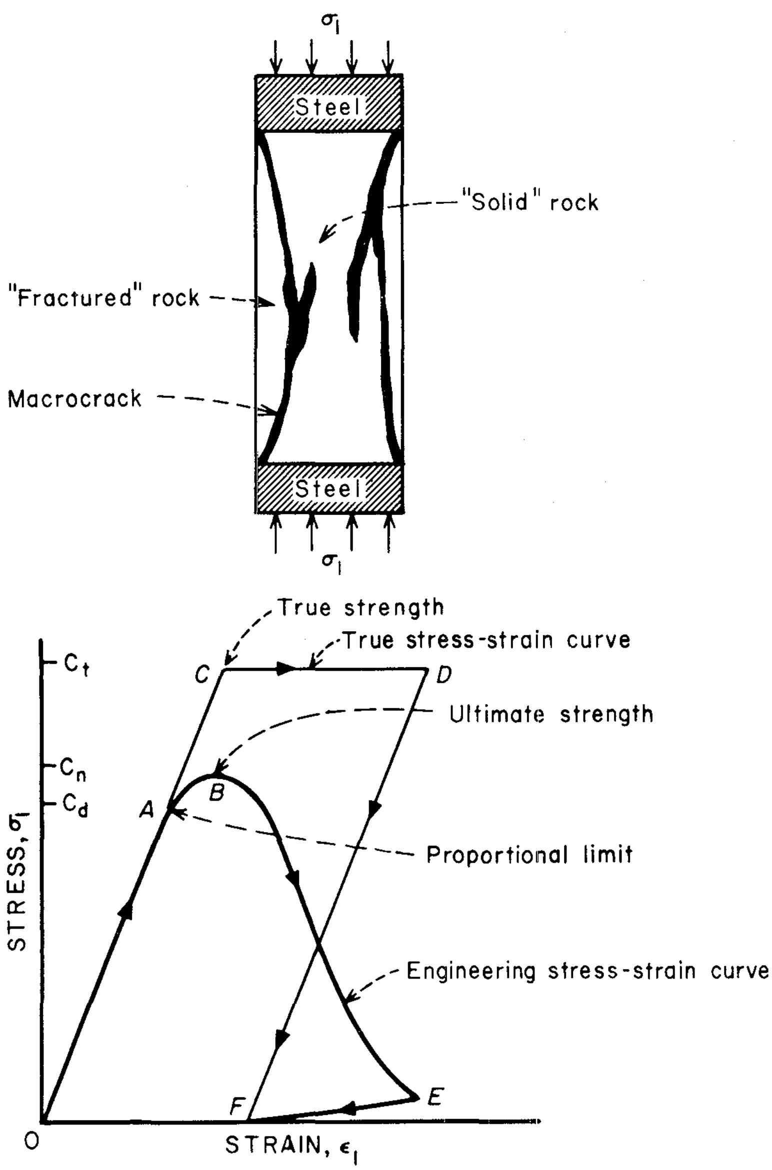

FIGURE 7. - Engineering and True Stress-Strain Behavior of Brittle Rock.

Laboratory tests conducted by the Bureau of Mines on model rock pillars have shown that the decrease in load on a fractured rock pillar in the post-failure region results mainly because of loss of load bearing area in the pillar and that the maximum stress on the area of solid rock within the pillar remains a constant (fig. 7) (2). These laboratory studies have shown that the fractured rock within the pillar under uniaxial loading carries little or none of the total load applied to the pillar. However, tests have also shown that the addition of only 1- to 4-psi radial confining pressure such as that resulting from supporting rock with rock bolts and/or wire ropes (fig. 8) can increase the failure strength of fractured rock pillars by as much as 10 percent. Thus, any simple means of applying an average

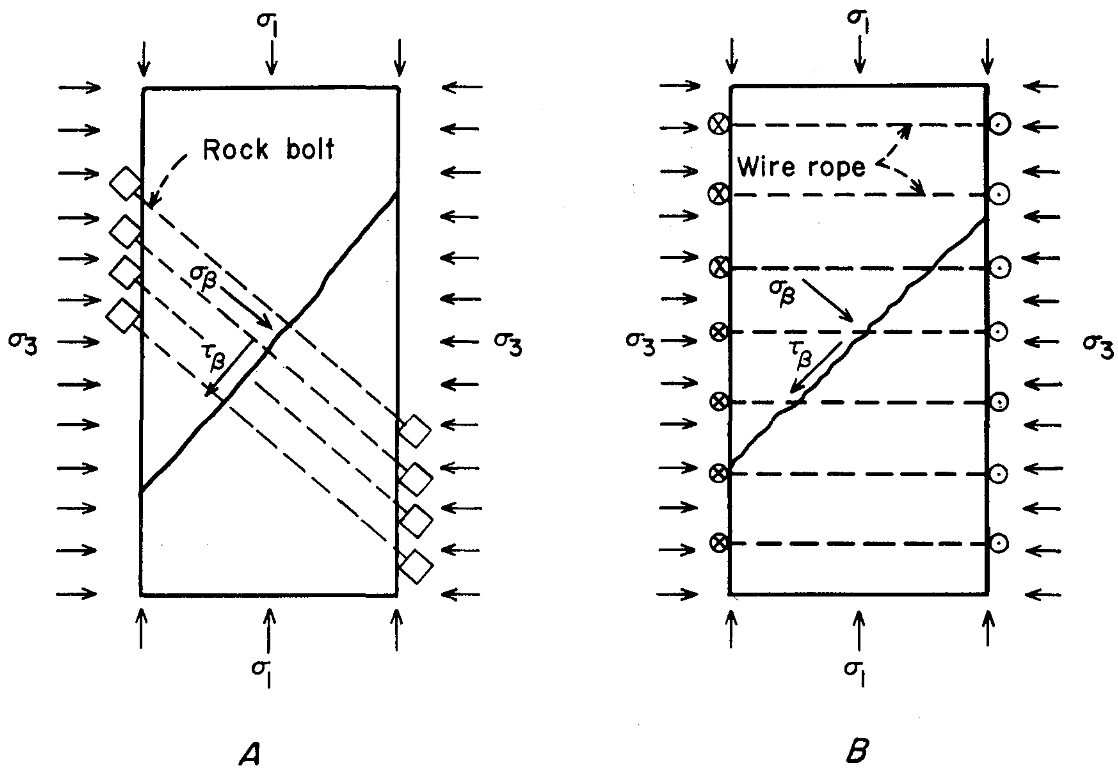

FIGURE 8. - Model Rock Pillars With *A*, Rock Bolts and *B*, Wire Ropes.

radial stress of a few psi should prove effective for strengthening and stabilizing fractured rock pillars.

These laboratory test results indicated a need to field-test these concepts. Accordingly, a cooperative agreement between the Bureau of Mines and the Colony Development Operation of Atlantic Richfield Co. was reached in 1971, enabling the Bureau of Mines to determine the mechanical properties of oil shale and to perform in situ stress determinations in the Colony oil shale mine. A major objective of this research project was to test the pillar stability and strength concepts discussed above.

A plan view of the underground workings at the time of stress relief measurements is shown in figure 9. The area from which core samples were obtained for mechanical property testing is also indicated.

Pillar dimensions in the Colony mine are 60 feet in height and 60 feet in diameter. The pillars are highly fractured and are transgressed by natural planes of weakness. The strike and dip of the weakness planes are approximately N-S and 60° S, respectively. Laboratory model studies have shown that pillar strengths are reduced considerably when they contain planes of weakness oriented at these angles. From uniaxial compression tests on drill core samples of oil shale, some of which contain planes of weakness and some of which are solid, we can estimate the strength of in situ pillars containing planes of weakness at various angles to the horizontal. The in situ situation of the

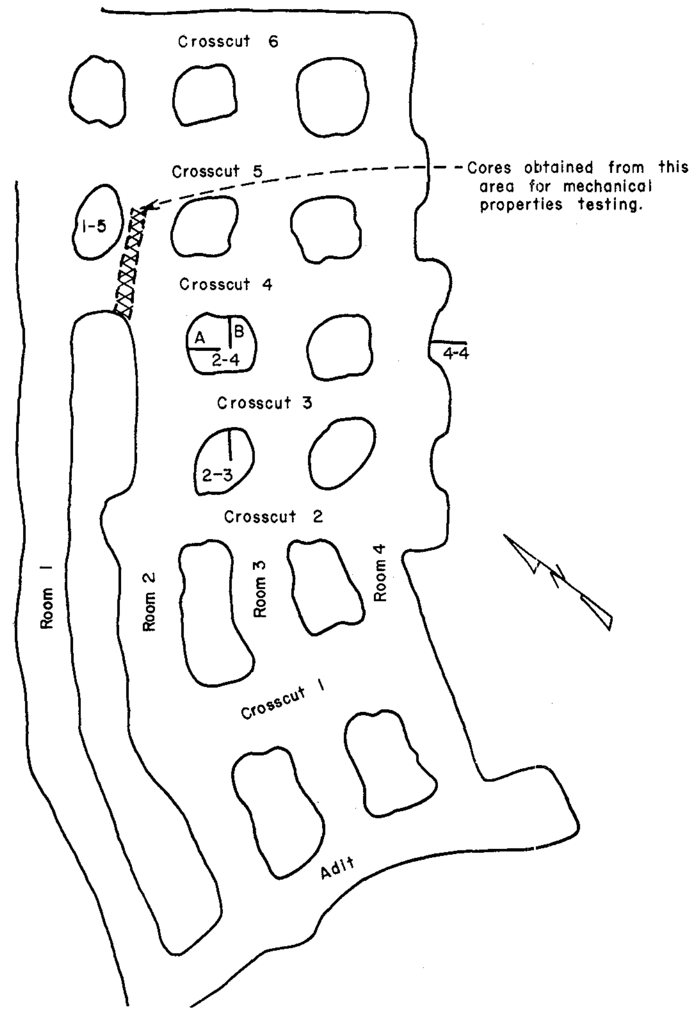

FIGURE 9. - Plan View of Underground Workings at Colony Oil Shale Mine.

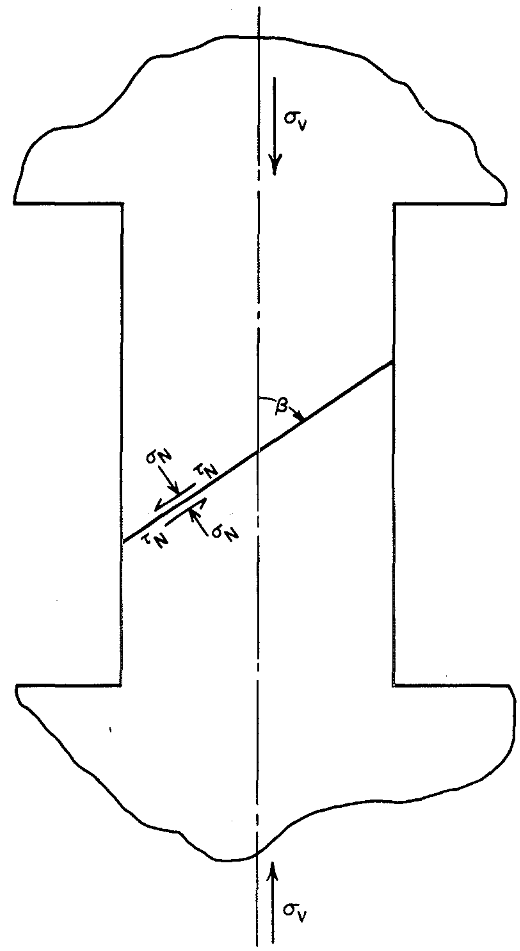

FIGURE 10. Cross Section of Rock Pillar Containing Plane of Weakness.

oil shale pillars with a length-to-diameter value of 1.0 was approximated by fractured cores with a length-to-diameter value of 1.0 and a β angle of approximately 45° (fig. 10). Laboratory tests results on these pillars give an ultimate strength of 3,400 psi for the 45° pillar and a solid core pillar strength equal to 12,700 psi. The fractured pillar strength of 3,400 psi is conservative for the real fractured pillar owing to both the interlocking effect of pillar fragments and to the triaxial stress state within the pillar due to its small length-to-diameter value.

The in situ stress field was obtained using the three-component Bureau of Mines borehole gage. A typical pillar stress profile is shown for pillar 2-4 in figure 11. The stress profile in the rib wall at proposed pillar 4-4 is shown in figure 12. In both figures the maximum and minimum stresses are denoted by P and Q, respectively. The orientation of these stresses with respect to the vertical is shown in each figure.

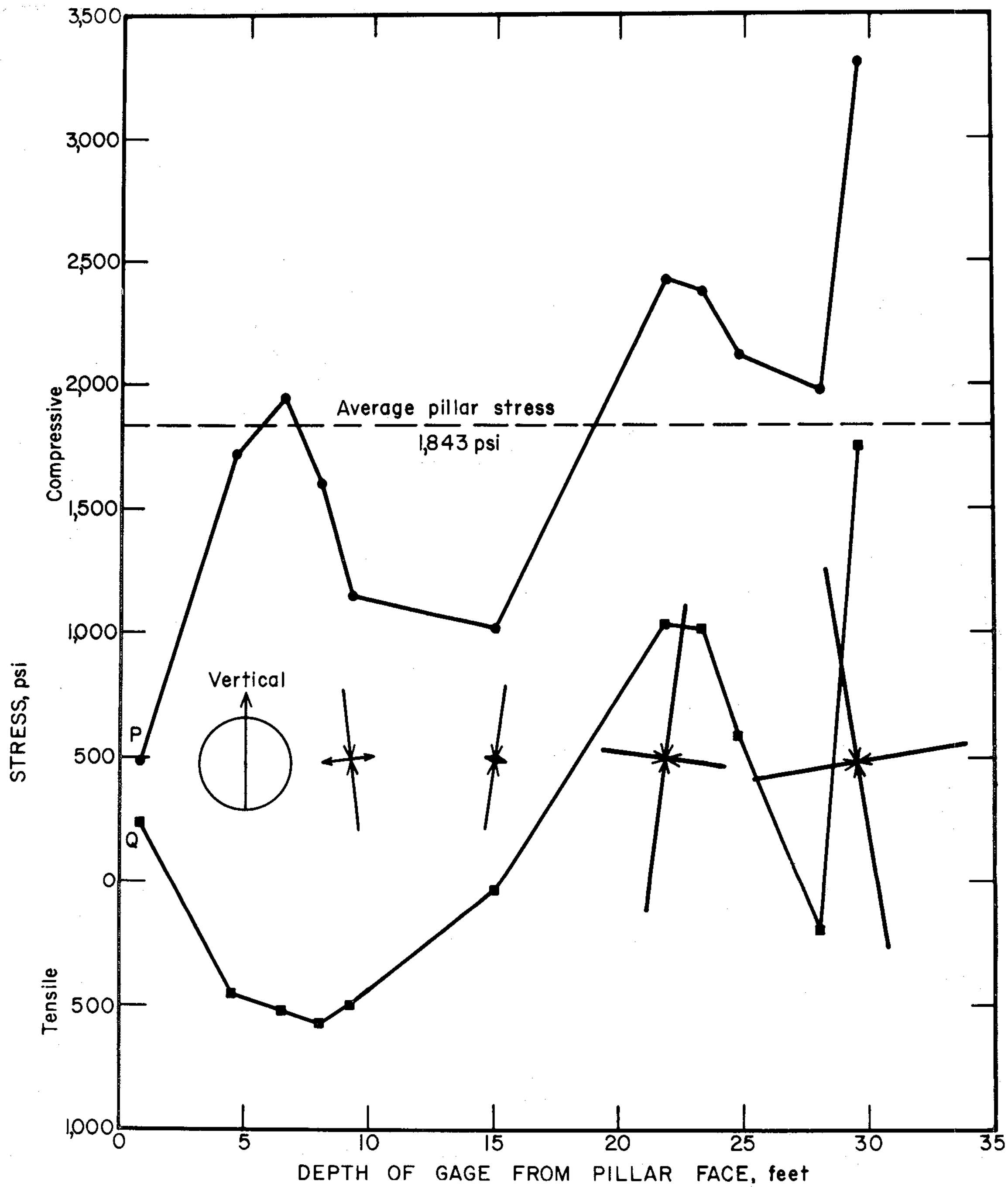

FIGURE 11. - In Situ Stress Field for Pillar 2-4, Colony Oil Shale Mine.

The overburden depths for pillar 2-4 and proposed pillar 4-4 are 690 and 807 feet, respectively. Using a weight density for the overburden of 147 lb/ft^3, the calculated overburden stresses are 704 and 824 psi, respectively. Using the cross-sectional area of pillar 2-4 and this overburden stress indicates this pillar should be bearing nearly 2,800 psi. The

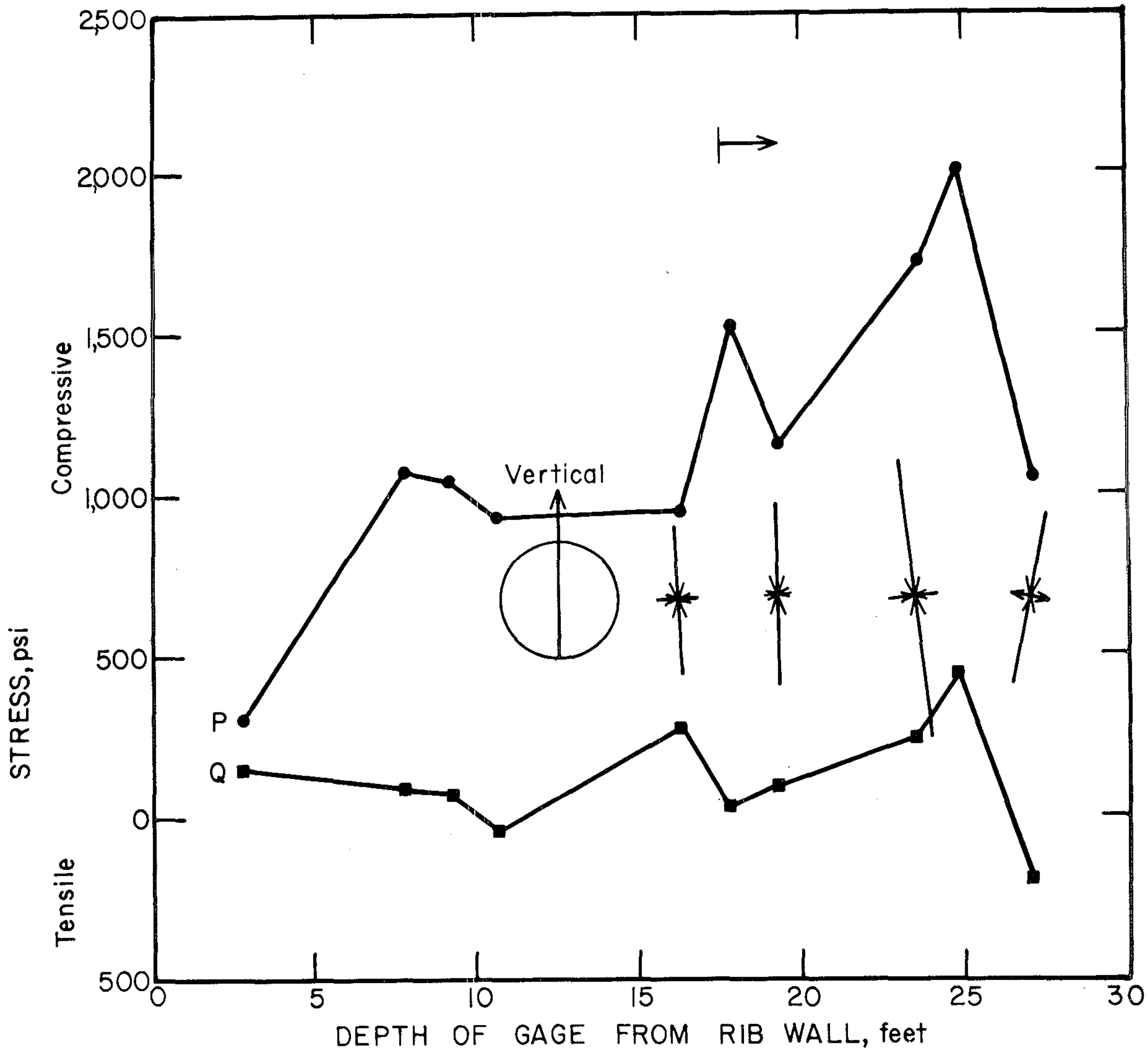

FIGURE 12. - In Situ Stress Field in Rib Wall for Proposed Pillar 4-4, Colony Oil Shale Mine.

calculated average vertical stress component is only 1,800 psi. Note that stress relaxation has occurred in the outer zone of this pillar. Recovered drill core showed this portion of the pillar to be in a highly fractured condition. The horizontal stress component is tension to a depth of 15 feet, where small compressive horizontal components begin to occur. The vertical stress in the central portions of the pillar begin to reach values on the order of 3,500 psi, indicating this portion of the pillar is more stable. Recovered core from this portion confirmed this observation. In the rib wall (4-4), the vertical component of stress again shows relaxation in the first 7-1/2 to 15 feet. The average vertical component of stress (fig. 12) is 1,500 psi, a value 1.8 times the expected value calculated from overburden. This is probably due to the effects of a pressure arch resulting from low pillar stresses. A Bureau of Mines two-component borehole gage was placed in 4-4 at the end of overcoring to determine stress changes with time as mining progressed.

Calculated stresses in this pillar after the upper 30-foot level had been mined were 2,768 psi average compressive stress in the vertical direction and 540 psi compressive stress in the horizontal direction. Continued mining on this full 60-foot pillar height eventually damaged the gage cable and the gage had to be removed, repaired, and replaced. Several additional readings were obtained after the 60-foot height was mined, before the gage was lost due to rock failure. Results of these readings indicate that the vertical component of stress in this pillar may have been only as much as 3,780 psi in the vertical direction with the horizontal component changing from compressive to 208 psi in tension at the time of failure. These results are significant in that they indicate that measured horizontal components of stress in tension in pillars indicate an unstable or failed condition of the pillar. These conditions also exist in pillars 2-3 and 2-4, but to a lesser extent.

Pillar 1-5 (fig. 10) was found to be in an unstable state--the pillar was breaking up. To stabilize this pillar, 19 rock bolts were installed; these had the effect of preventing further collapse. Theoretical analysis indicated that the main effect of these bolts was only to hold it together; that is, they had the effect of holding the fractured outer portions in, allowing them to carry some of the applied load. This has the effect of taking off load in the central unfractured portions of the pillar.

The results of these field tests are encouraging. While further study is necessary, it appears that the pillar strength and stabilizing concepts developed by the Bureau of Mines are valid.

SUMMARY

Basic requirements have been discussed in this report necessary for efficient design of engineering structures in rock and their successful application to practical mining problems. These requirements form the basis of the discipline of rock mechanics. However, it is clear that further work is necessary before we can safely say that rock mechanics is on its way to becoming an exact engineering discipline. The achievement of this goal is one of the objectives of the ground control research carried out by the Bureau of Mines.

REFERENCES

1. Blake, Wilson. Rock Burst Research at the Galena Mine, Wallace, Idaho. BuMines Tech. Prog. Rept. 39, August 1971, 22 pp.

2. Brady, B. T., W. I. Duvall, and F. G. Horino. A Study of the Post-Failure Characteristics of Brittle Rock. Proc. Rock Fracture Symp., Nancy, France, 1971, 22 pp.

3. Obert, L., and W. I. Duvall. Rock Mechanics and the Design of Structures in Rock. John Wiley and Sons, New York, 1967, 650 pp.

SOLUTION OF MINE STRUCTURE PROBLEMS THROUGH FIELD MEASUREMENTS AND THEORETICAL ANALYSIS

by

Louis A. Panek [1]

ABSTRACT

Realistic values for the stresses, deformations, and displacements of a mine rock mass structure, the geometric and mechanical properties of the rock components, and the preexisting ground stresses all can be measured in the mine; hence we can formulate quantitative relationships among these factors in order to predict and control mine structure behavior. Theoretical structural analysis helps us to formulate and to interpret these relationships. Mine structure analysis includes the measurement and correlation of structural properties and behavior, as well as the simulation of structural behavior by mathematical procedures. The role of field measurement data in mine structure analysis is illustrated through discussion of our approaches to several ground control problems.

INTRODUCTION

When rock is subjected to a load change, a deformation occurs. Ordinarily, this is observed in a mine as the displacement of one point with respect to another--deflection or even caving of the roof, which may be observed as a convergence between roof and floor; or the extrusion of material from the rib, which may be observed as a decrease of the distance between the rib and a nearby post.

The effect of high rock stress is in some cases desirable and in others undesirable, but in any event the observable effect is a displacement, which depends on the following four factors:

(1) The geometry of the structure--the size and shape of openings, pillars, and linings, and the geologic bedding and jointing.

(2) The mechanical properties of the rock--principally the strength, deformation modulus, and flow characteristics.

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(3) The load or applied stress--primary sources are the weight of superincumbent rock; secondary sources are redistributed pressures caused by other nearby openings.

(4) Duration of load.

Control of rock displacement is exercised by control of one or more of these factors. To achieve control we must in general be able to measure all of these factors in some sense, perhaps only qualitatively by visual observation, but preferably by actual quantitative determination with a measuring instrument.

The monitoring of pressures and deformations in a mine rock structure can be used by the mine operator as a basis for initiating changes in his extraction procedure. Used in this way, measurement becomes part of a very practical design-as-you-go procedure. Alternatively, the measured rock pressures and deformations can be used in conjunction with theoretical structural analysis to study the comparative advantages of proposed changes in mining extraction sequences. The development of the finite-element method of structural analysis in the past 10 years has made possible the mathematical analysis of the most complicated structural problems imaginable, including openings of irregular shape, rock layers of varying thickness and properties, and even including the effects of natural rock jointing and of time-dependent deformation. To carry out the mathematical analysis, we must first quantify the four factors that govern rock deformation; namely, the geometry of the structure, the mechanical properties of the rock, the ground stress, and the load duration.

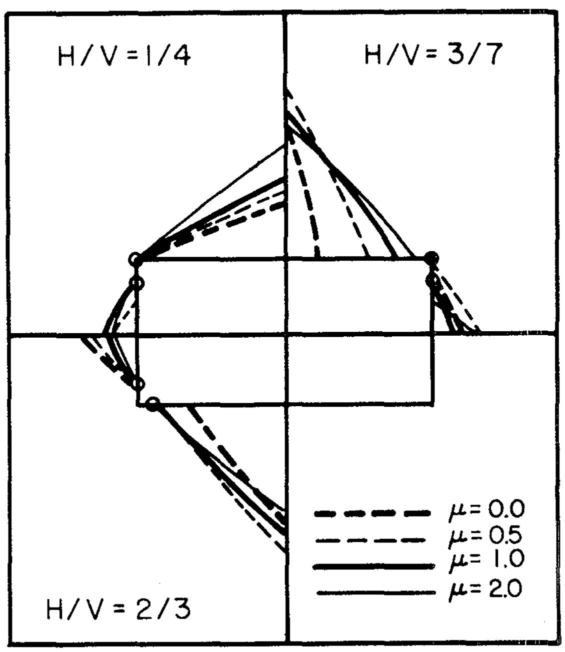

FIGURE 1. - Critical Fracture Surfaces. H/V = horizontal vertical ground stresses; μ = coefficient of internal friction.

Several ground control problems will be discussed to illustrate the need for specific kinds of measurement data and the field measurement techniques that we have developed to meet these requirements.

GROUND PRESSURES AND COEFFICIENT OF FRICTION HELP TO PREDICT MINE OPENING FAILURES

Recently we made a series of analyses of a single mine opening, incorporating the relatively simple Coulomb shear failure criterion (fig. 1). The failures predicted by this analysis consist of pillar slabbing and roof arching very similar in shape to rock burst failures in hard rocks and to roof arching failures above coal mine entries. A gothic-arch-shape failure above the opening is a common occurrence in coal mining. The Coulomb analysis indicates a dependence of the height of this arch on the ratio of horizontal to vertical earth pressure (H/V) and on the coefficient of internal friction (μ), two factors that must be determined in order to apply this analysis accurately to a specific mine.

We are developing schemes for determining the ratio of H/V in coal by observing the trends of pressure with time, in hydraulic pressure cells that are installed in 2-1/4-in drill holes. The second parameter, the coefficient of internal friction, is a property of the rock or coal, which is ordinarily determined by laboratory triaxial testing of specimens subjected to different ratios of horizontal to vertical stress. As an alternative to preparing and testing specimens, which is expensive and time-consuming, we hope to have operational soon a field test device to determine this coefficient by in-place tests. The device consists of an expandable plug that is dilated inside a borehole, then withdrawn, shearing out a portion of the rock or coal.

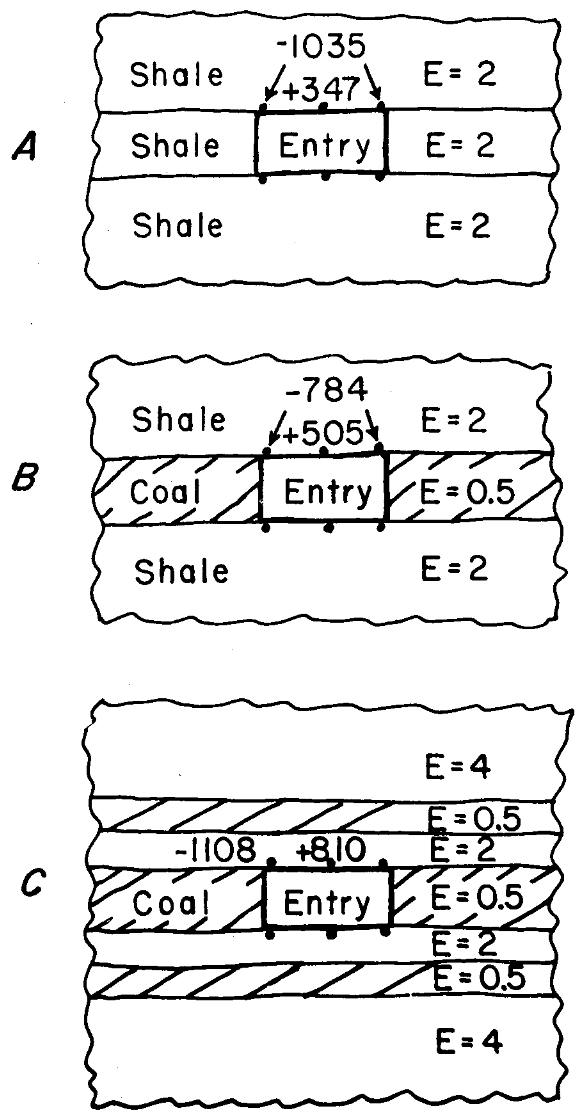

FIGURE 2.-Critical Horizontal Roof Stresses (psi). 8- x 16-ft entry at 1,000-ft depth, showing influence of strata stiffnesses (E x 10^6 psi).

SYSTEMATIC MEASUREMENTS ARE NEEDED TO UNRAVEL A COAL MINE ROOF FALL PROBLEM

A second structural analysis example is the behavior of the roof over an entry in a layered mineral deposit. Consider a simple rectangular opening, 8 ft high by 16 ft wide, at a depth of 1,000 ft (fig. 2). If the roof, floor, and "seam" are all shale, as shown in figure 2A, where E represents Young's modulus of the formations, a finite-element analysis shows that the critical stresses on the boundary of the opening are a horizontal tensile roof stress of 347 psi at midspan and a horizontal compressive roof stress of 1,035 psi near the corners of the opening. Now let us replace the shale seam by a coal seam, retaining the shale roof and floor (fig. 2B). Because the stiffness of the coal is only one-fourth that of the shale, more deformation occurs at the rib, which permits more roof bending, as compared with figure 2A, with the result that the maximum roof tensile stress increases to 505 psi and the maximum roof compressive stress decreases to 784 psi.

Extending this example to a coal mine, if the tensile stress exceeds the tensile strength, the lowest roof layer should crack at midspan, and so we may see the successive failure of thin roof layers. On the other hand, if the roof is strong enough in tension, the maximum compressive roof stress near the corner may initiate failure by Coulomb shear, leading to the arch-type failure. That is, the type of roof failure that occurs may depend on the relative stiffnesses of the coal and roof layers, not only on the rock strengths.

If either the roof or the coal has different stiffness in the north-south direction than in the east-west direction, then the roof failure in a given mine may tend to occur by one mode in face entries and by a different mode in butt entries. Of course, the rock strengths may be different in different directions, and so too may be the horizontal ground stress, which may increase or nullify the roof failure tendency, depending on direction.

Even in this apparently simple three-layer rock structure, segregating cause-and-effect relationships in order to devise a better approach to a mine roof-fall problem requires several kinds of measurements--of rock properties, of rock and seam deformations, and of earth pressures. Very few individuals seem to fully realize the significance of the interplay between these factors, the number of combinations of effects that are possible, and the need to establish valid proof of cause and effect by evidence that is both necessary and sufficient.

Figure 2C shows that a soft mudstone or a thin "rooster" coal layer just above the first roof layer weakens the roof even more.

These relatively simple models point up the need to determine routinely the stiffness of the coal seam and of the rock layers above and below, as well as the preexisting ground stresses. We are now able to determine all of these quantities by in-place tests with our hydraulic pressure cells. For example, we drill a 1.5-in-diameter hole up or down from the seam, insert a cylindrical pressure cell to the back of the hole, where we expand it, and measure the change of hole diameter produced by a given change of radial pressure, from which we calculate the stiffness of the rock layer. This test is repeated at successively shorter distances from the seam to determine the stiffness at selected strata horizons. No rock samples are needed, and furthermore, the test measures the stiffness of the rock as it exists in its natural state, including the effects of fractures and other irregularities.

LONGWALL MINING AS AN EXAMPLE OF MAJOR MINE STRUCTURE BEHAVIOR

For a third structural analysis situation, we consider longwall mining, a fundamental concept of which is to minimize the transfer of overburden weight to the mine workings. Until a substantial area is undermined and the mine roof is caved, the weight of undermined overburden, which increases as mining progresses, is steadily transferred to the perimeter of the undermined area, possibly creating a severe roof control problem in perimeter openings. The same phenomenon occurs on the perimeter of an area that is completely extracted by pillar mining. Measurements of surface subsidence over undermined areas show that arching and load transfer will cease to be possible once the short dimension of the undermined area substantially exceeds the depth, as indicated by the fact that this mining width is required in order to produce complete subsidence. Consequently, we measure subsidence whenever possible, not only because it helps us to predict subsidence itself, but because of its implications with respect to load distribution in the mine.

Figure 3 shows a simple scheme for measurement of subsidence over a longwall coal mining panel. There is one row of subsidence monuments along the length of the panel and a row across the panel. Additional markers in a second row across the panel are placed in a strain rosette configuration, strains being determined by taping measurements. There are also some tiltmeter stations for measuring inclination of the surface. What is unique is the instrumented drill hole for making subsurface measurements, which we can correlate with the underground measurements and the surface measurements and relate to the properties of the rock layers in the overburden.

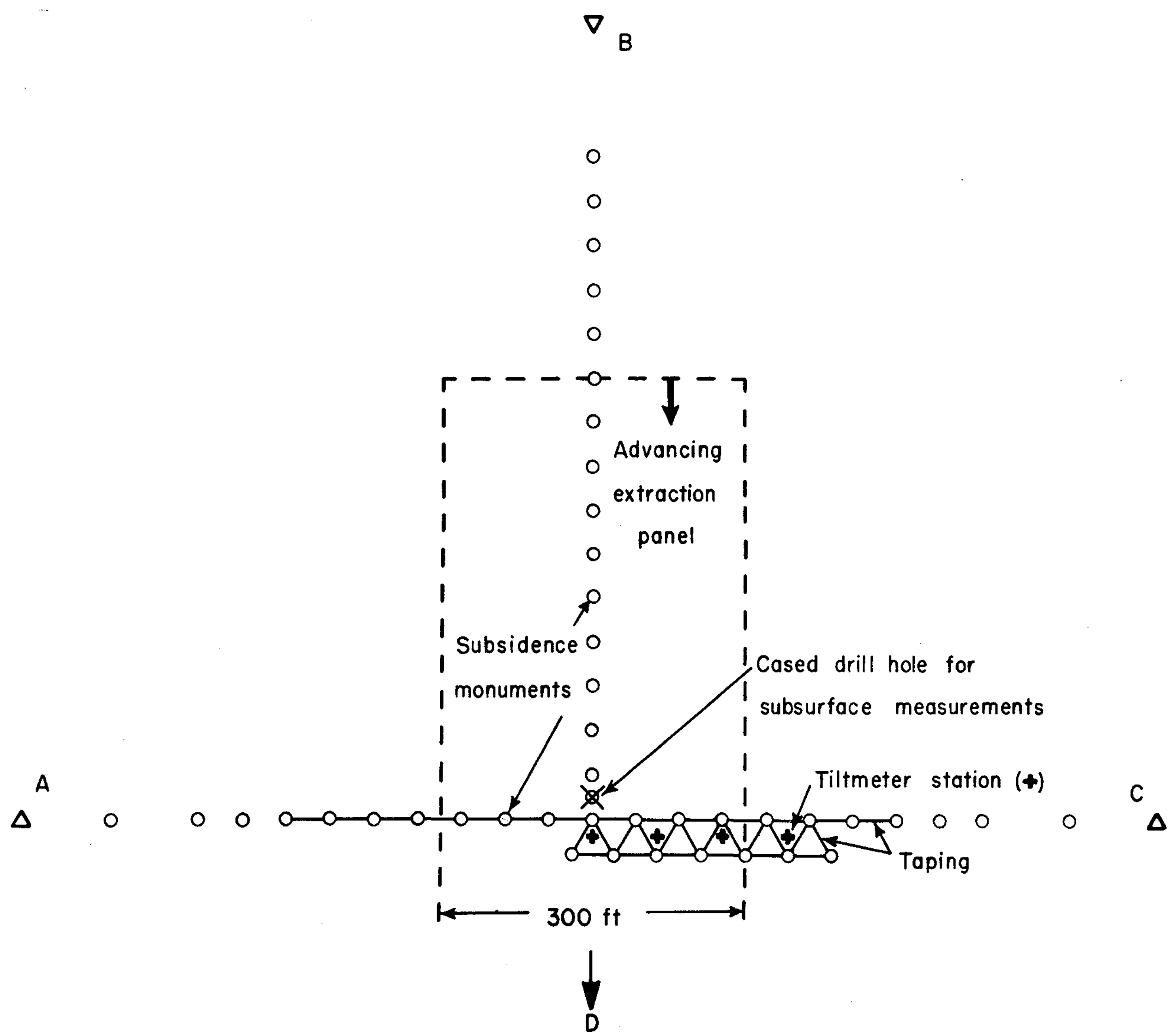

FIGURE 3. - Surface Instrumentation Over a Longwall Panel.

Figure 4 is a section through the drill hole. It contains hydraulic pressure cells to detect changes of horizontal pressure in the overburden as the longwall face is mined underneath. It contains a coaxial cable down which we send electrical pulses to detect zones where strata separations or horizons of shear displacement are developing. Finally, we grout into the hole an

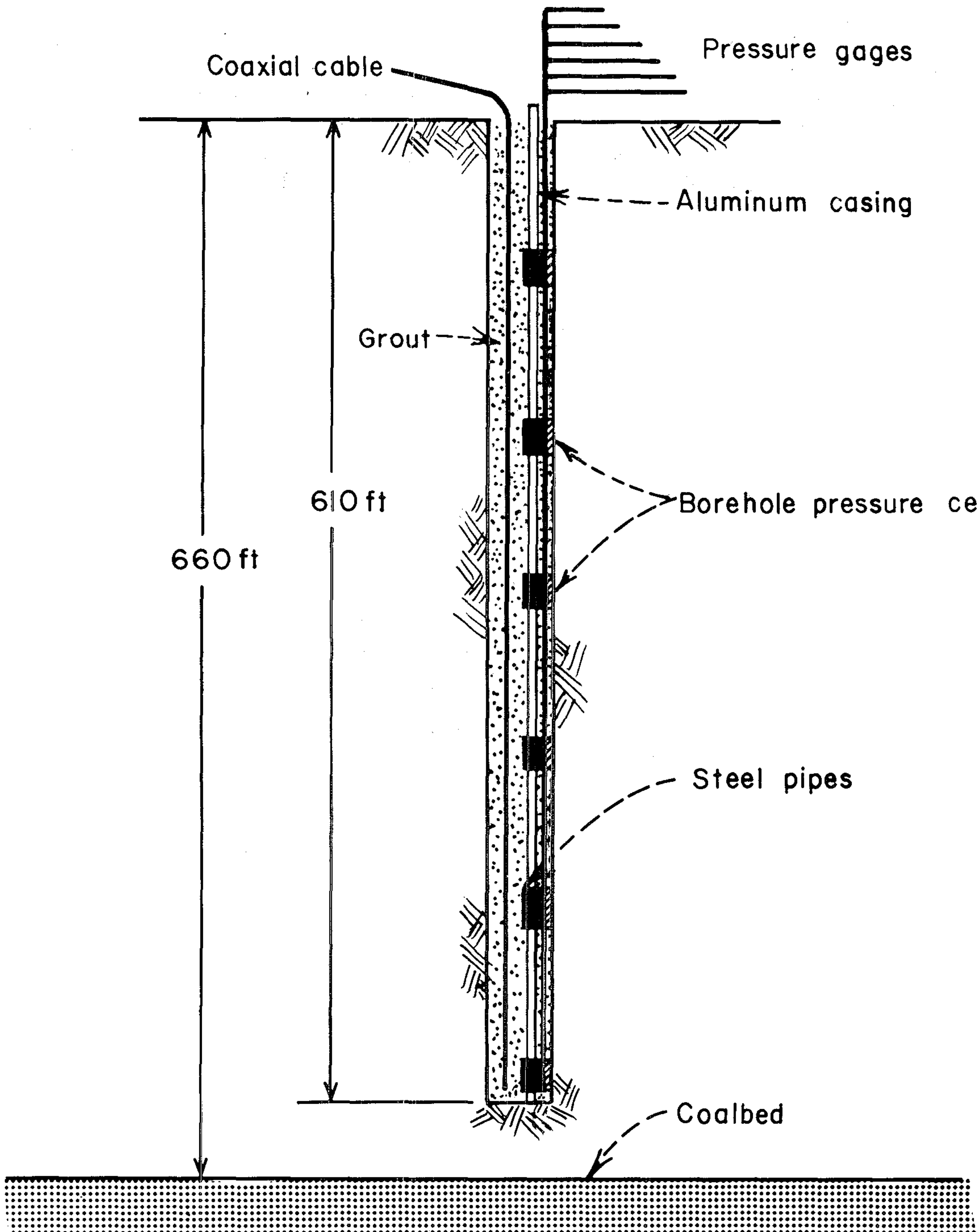

FIGURE 4. - Instrumented Surface Drill Hole in a Subsiding Rock Mass.

aluminum casing which serves as a reference datum for repeated runs with our inclinometer probe--from the inclinations of the various horizons we can calculate the changes of profile of the drill hole, which represent distortions of the rock mass.

In longwall mining, the critical area is along the face: If roof control is inadequate, then the immediate roof breaks up, roof falls occur along the face or between the supports, and production stops. A common reaction to this is to prescribe supports of higher capacity, in order to create greater resistance to convergence in the face area. However, this is not necessarily the answer. There is a minimum convergence dictated by the flexure of the mass of rock that comprises the overburden, which depends on the thickness and stiffness of the roof rocks and of the coal and the floor rock (the coal may tend to sink into a weak fireclay floor). Obviously, the structural properties of these rock layers are of great significance. The mechanized support system along the face cannot restrict this curvature of the main roof--artificial supports cannot resist a load of this magnitude, and in fact the attempt may aggravate a bad roof condition.

Experience shows that we need to provide support only for the immediate 10-, 20-, or 30-ft thickness of rock. The structural problem is to determine what thickness of rock must be supported and what amount of convergence must be permitted. Then we can provide a mechanized support system that yields at the appropriate level of load, thus preventing the development of excessive load.

Another consideration with respect to yield load of the supports is the bearing strength of the rock. Obviously, it will be ineffective to employ a 40-ton yield load prop if the props punch into the roof or floor at a 20-ton load. Consequently, one should determine the bearing strength of the roof and floor before purchasing and installing longwall equipment in order to avoid this difficulty. We have therefore developed a bearing test using a series of bearing plates of different sizes, so that we can scale up the results to the actual size of the foot plates of commercially available mechanical chocks.

Determination of the convergence and support load values is much more difficult. In England the National Coal Board has found that 1 ton of support per square foot of face area is required for good roof control, and that the roof will start to break up if the convergence exceeds 1/2 in per foot of face advance in a 3-ft seam. Since the values of these factors depend on the relative stiffnesses of the rock layers, we have little reason to suppose that the safe or required values of support load and convergence will be the same in the United States as in England. However, we can reasonably expect to determine appropriate values by field measurement coupled with structural analysis.

The Bureau would like to conduct such a series of measurements at one or more operating longwalls. As I mentioned earlier, we can easily determine the stiffness of the roof and floor rocks and of the coal seam by in-place tests performed underground. We also can prescribe instrumentation to make reliable measurements of prop loads, convergence, relative roof displacement, pressure in the coal, curvature of the main roof, and the heights at which relative displacements occur between strata, all of which we need to know in order to calculate the structural behavior of the roof and the roof thickness that must be supported by the props. Commercially available self-advancing support systems differ as to their support characteristics. Without preknowledge of what is required by a particular set of strata, selection of support equipment is largely guesswork. It is not surprising, therefore, that a significant proportion of longwall attempts are unsuccessful.

IN-PLACE MEASUREMENTS IMPORTANT IN CHARACTERIZING OF ROCK MASS PROPERTIES

A large part of major mine structure behavior has to do with the bending and subsiding of the overburden, which we delineate by measuring pressures and displacements. The objectives are to determine which strata groups are independent-acting units, to assign appropriate stiffness moduli to them, to adjust these parameters to make the calculated subsidence profile match the true profile, and to make the calculated ground pressures and closures in the mine match the measured values. We have attempted to determine these structural properties by sonic logging in the borehole. Although sonic determinations of modulus never produce the same result as load-deformation measurements, we hope to find that sonically determined relative stiffness values can be appropriately adjusted. So far, however, attempts to calculate the subsidence profile of the overburden from the material properties of the constituent rock layers have not succeeded, which indicates that our mathematical model is lacking in some essential characteristics.

Our lack of knowledge as to how to properly represent mathematically the behavior of the overburden has an underlying similarity to our inability to explain the uncontrollable caving that is experienced in entry-driving in certain Appalachian coalfields. Evidently, something more complex than brittle elastic behavior of a uniform rock mass is involved, because roof caving in half of the headings of a multiple-entry system cannot be explained by invoking conventional criteria of strength based on stress distributions that are calculated on the assumption of continuous elastic rock layers.

One factor that must be considered is the natural jointing or cleat in the coal and rocks. We have recently made some preliminary studies in an attempt to delineate the patterns of jointing in coal mine roof rocks, to transform measurements of joint spacing and attitude into quantitative parameters suitable for theoretical structural analyses, and to develop the necessary analytical procedures to incorporate these jointing parameters.

Another factor that may have substantial influence on the mode of structural behavior is the time-dependent deformation of coal and of rock strata. We are presently conducting mathematical analyses of coal mine entry and panel systems to account for nonlinear, time-dependent interaction of overburden, coal, and underlying strata.

To complement the computer analysis capability, we need to be able to measure inelastic mechanical properties in-place, in mines, and to measure ground pressures in inelastic rocks, in order to specify the properties of real structures and to delineate their structural behavior. We have recently developed techniques for determining existing ground pressure as well as pressure change in a viscoelastic medium, based on observing the time trends of fluid pressure in a set of hydraulic pressure cells installed in drill holes, and from measurements of the change of diameter of an open drill hole. We began these studies in salt and potash, which are uniform and continuous, and then extended the techniques to coal--the same principles seem to apply.

For underground measurements in coal mines, my group relies primarily on simple mechanical extensometers for making measurements of displacements along or across a drill hole or a mine opening, and on hydraulic pressure cells for determining the deformation characteristics and/or the ground pressure. All of this equipment is one-man portable and one-man operable. Usually we don't even use recorders--we simply take readings at hourly, daily, or weekly intervals, as needed according to objectives.

Figure 5 shows a typical layout in which hydraulic pressure cells are placed to reveal changes of ground pressures due to coal extraction by a longwall operation. Some pressure cells are in the chain pillars, others are in the panel. Each of the two sets of pressure cells in the panel will reveal the increase of ground pressure as the face approaches and will provide a profile of ground pressure across the panel. The ground pressure changes depend on the flexural behavior of the main roof and on the stiffness of the coal and floor. The pressure values are needed in order to interpret the structural behavior.

We use two kinds of hydraulic pressure cells in drill holes--cylindrical cells and flat cells--details of which will be discussed by Reed Smith (pp. 49-56). The flat cells are installed inside 2-1/4-in drill holes to indicate change of ground pressure. A sand-cement mortar fills all the space around the pressure cell, which is directional, responding only to changes of ground pressure at right angles to the plane of the cell. More recently we have experimented with pre-encapsulating the cell, which necessarily produces a softer cell, but which eliminates the delay underground for aging the mortar, after the cell is emplaced.

The cylindrical pressure cells (CPC) are installed in open 1-1/2-in drill holes; that is, without cement. Although we often use the CPC as a ground pressure sensing device, its primary function is to determine the modulus of the rock. The CPC exerts radial pressure against the wall of the drill hole; the stiffer the rock, the more pressure is required to dilate the hole by a given amount. With our equipment, we determine with high precision the pressure required for each increment of hole dilation, which ratio is proportional to the modulus. The deformation modulus determined in this way is the average value over the 7-in length of the CPC. Since the test specimen is not removed from the natural position, this is a test of the undisturbed rock, including the influence of the defects, fractures, natural rock jointing, etc. Figure 6 shows that the effective modulus of a rock mass in its natural position may be less than one-fourth the value indicated by laboratory testing of an intact core specimen, even considering, for example, a 10-ft-diameter mine opening and a 5-ft joint spacing, which is not close jointing. (In fig. 6, G_a is the modulus of rigidity determined by CPC test in a granite. The left-hand ordinate scale is expressed as a fraction of the average modulus $\overline{G}_o$ for unjointed rock. For the abscissa scale, the fracture spacing $\overline{s}$ is expressed as a fraction of the diameter d of the opening.)

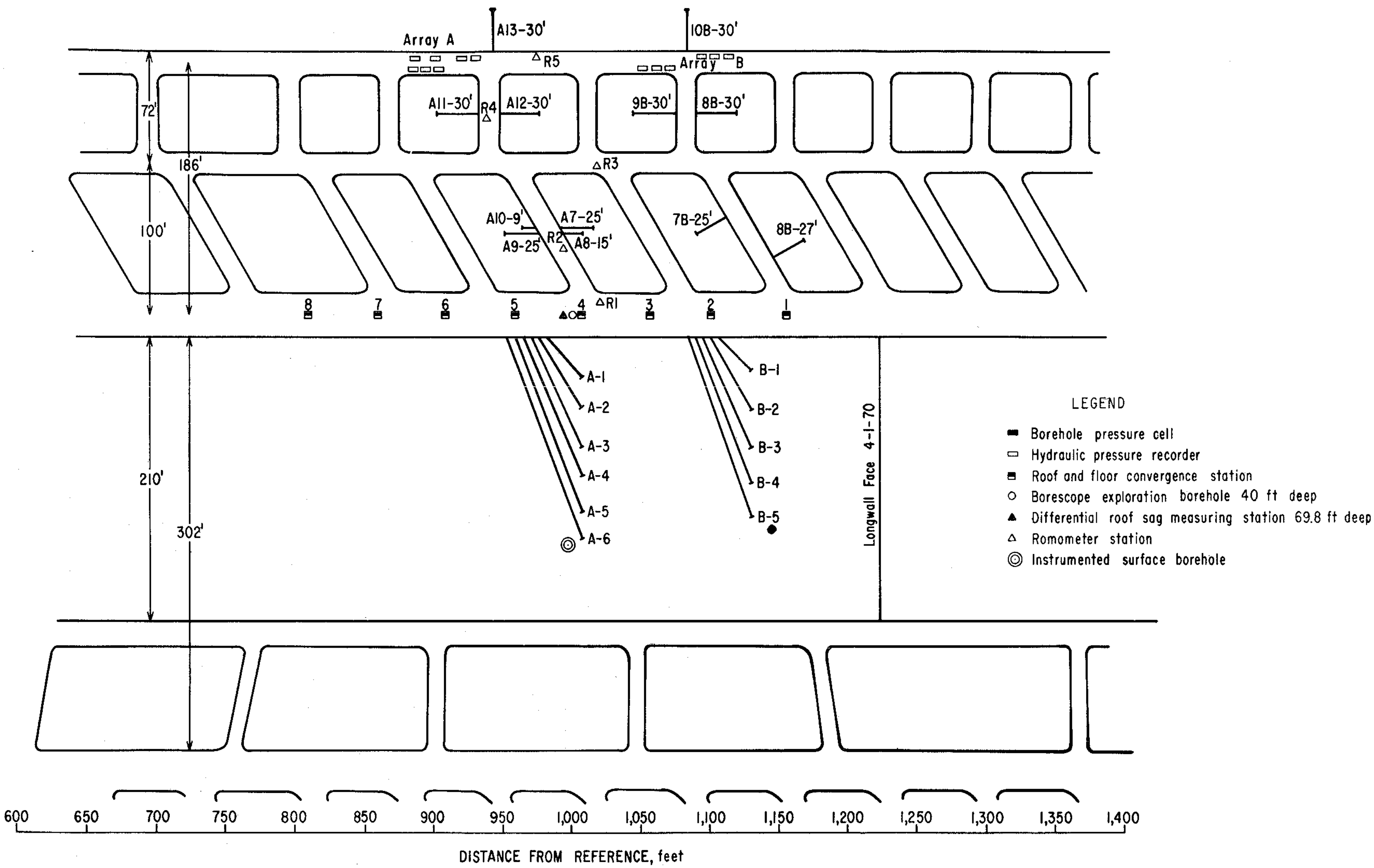

FIGURE 5. - Underground Instrumentation To Study Ground Behavior in the Vicinity of a Longwall Panel.

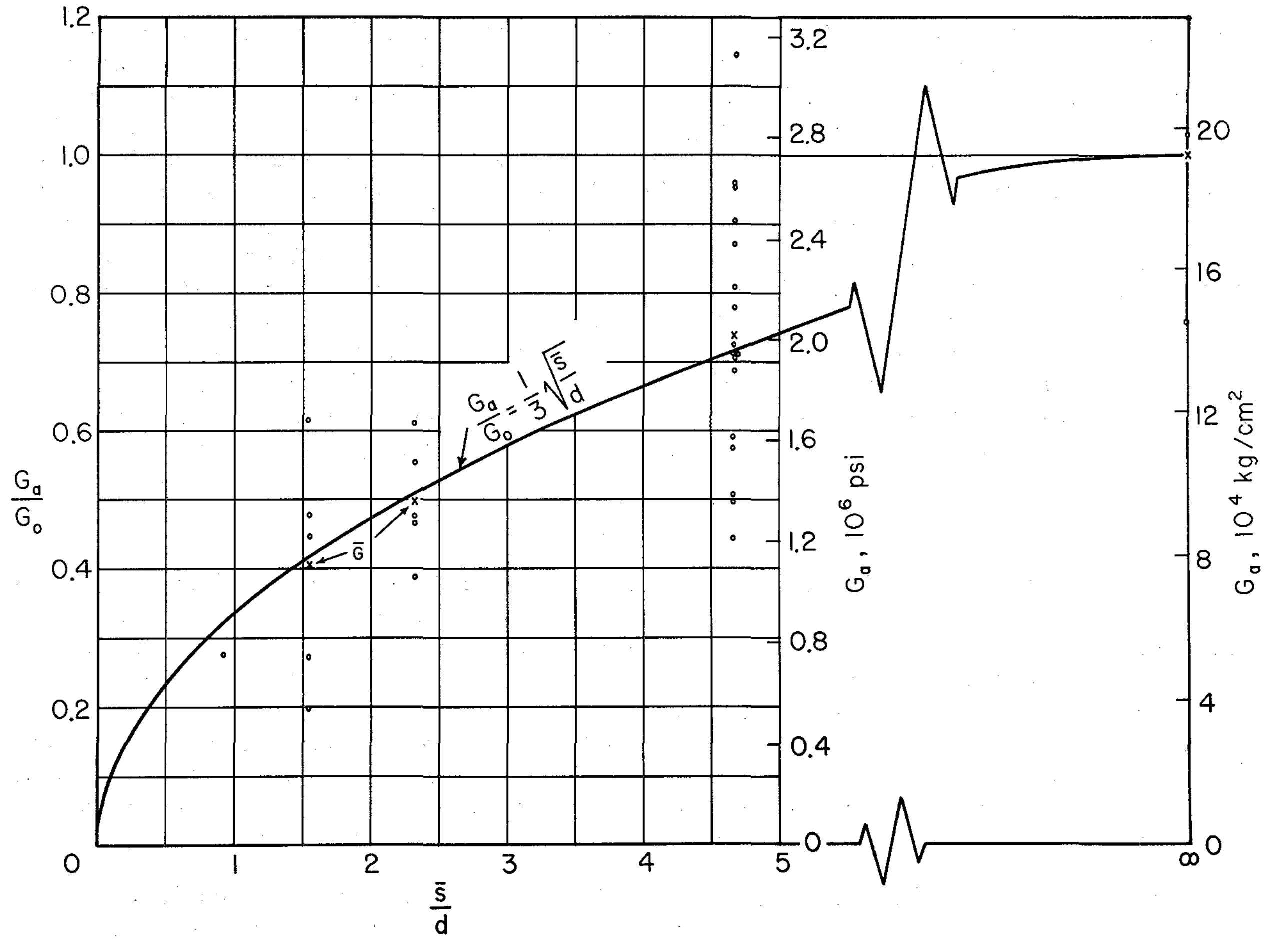

FIGURE 6. - Decrease of Rock Stiffness With Decreasing Joint Spacing.

If the effective modulus is influenced this much by jointing, then it is obvious that we must make much more use of in-place test data in our design calculations if we expect to be able to predict, for example, how a real coal mine pillar will behave under specified loads.

One of our major concerns is to develop in-place tests, the results of which can be incorporated into structural analyses so that we can do a better job of predicting the behavior of large rock masses. The largest rock mass that we must consider is the totality of all the rock overlying a mine. This rock may be allowed to set down on the mine floor if the seam is extracted 100 percent, or it may be supported on the pillars that are left in a partial extraction scheme.

The amount of load that will be transmitted to the pillars depends on the stiffness of this overlying mass of rock as well as on its total weight (thickness), and of course on the total pillar area and the pillar stiffness. In our studies of the behavior of the overburden, we have to delineate the geometry and properties, as already mentioned. However, in addition, to evaluate the analytically predicted behavior we need to measure the actual deformations of this large rock mass. Measurements of surface subsidence describe the deformation of one boundary surface of this rock mass. In order to better delineate the behavior of this large mass, we are increasing the use of our newly developed automatic-recording borehole inclinometer probe, the details of which will be discussed by Bill Tesch (pp. 68-74). To use the inclinometer probe we must drill a hole at least 4-1/2 inches in diameter from the surface down to the coal seam. In this hole we grout a special casing to serve as a fixed guide for the wheels of a special traveling probe that contains two inclinometers. From repeated measurements of the angle of inclination of the drill hole taken at regular intervals along the drill hole the profile of the drill hole can be constructed and the horizontal displacements of individual points can be calculated. We were led to develop this device because there was no commercially available probe that could be modified for automatic recording; considering the amount of data that we must generate, the manpower requirements for the usual manually operated system were prohibitive.

SUMMARY

These examples are illustrative of the range of structural behavior problems that we attack by a combination of field measurement and mathematical analysis. For all of these problems we must make available to the structural analyst information regarding the four major categories--the geometry, the mechanical properties, the ground stresses, and the time factor. In order to obtain realistic values to use in our mathematical analyses, we conduct a substantial program of field measurement. In fact, on the order of four times as much manpower must be engaged in the generation of measurement data as on theoretical analysis, so that the one activity keeps pace with the other.

Producing valid measurements of rock mass behavior is a demanding, time-consuming activity. We have developed and employ a relatively small number of proven and reliable techniques. For our work underground, my particular group emphasizes the use of inexpensive, simple, rugged equipment that is appropriate to the mine environment, so that use of the technique and equipment will be well within the capabilities of average mine personnel, who in many instances collect our data for us after we install the apparatus. We use virtually no electronic equipment underground, which has a lot to do with its reliability and which enables us to work with equal freedom in any mine, since there is no problem regarding electric power supply or permissibility. The stability requirement is especially severe. Changes of ground stress due to mining usually take place over a period of months; the same is true of rock relaxation (strain) under load.

We have the capability to monitor rock mass behavior adequately over a wide range of conditions, in rocks that possess relatively simple mechanical properties. Most commercial mineral deposits, however, involve rock that is fractured and/or exhibits time-dependent stress-strain characteristics, which is why the simpler approaches have not yet been able to provide the concepts that are needed to solve some longstanding problems of ground control. Developing the capability for determining rock mass properties and for measuring rock mass structural behavior will be a real breakthrough, which will unlock most of the problems of ground control. The obstacles are great, but the rewards for overcoming them will be much greater.

BUREAU OF MINES BOREHOLE DEFORMATION GAGE FOR DETERMINING IN SITU STRESS

by

James R. Aggson[1]

ABSTRACT

The Bureau of Mines has developed a three-component borehole deformation gage designed to measure diametral deformations of a borehole during the overcoring process of stress relief. The deformation measurements provide sufficient information to calculate the state of stress in the plane normal to the borehole. If borehole deformation measurements are obtained from three nonparallel boreholes, the three-dimensional average ground stress components can be determined.

INTRODUCTION

During the past three decades an increasing amount of consideration has been given to the problem of measuring and evaluating stress in rock around underground openings. Calculated stresses based on borehole deformation measurements have proven to be both accurate and reliable in recent years.

A Bureau of Mines report dealing with the measurement of the deformation of a borehole in rock subjected to a change in applied stress was published in 1961 (5).[2] A gage designed to measure this borehole deformation was constructed and reported on in 1962 (7). Since that time, numerous improvements in borehole gage design and refinements in the theory of elasticity associated with borehole deformation have been made. This paper describes the borehole deformation gage developed by the Bureau of Mines and some of the recent improvements that have been made in it, and the interpretation of borehole deformation measurements.

OVERCORING

The Bureau of Mines borehole deformation gage is used in conjunction with the overcoring method of producing stress relief. The overcoring method basically consists of (1) drilling a pilot borehole, (2) positioning the borehole deformation gage in the pilot borehole, and (3) diamond drilling a concentric borehole over the gage and recording the borehole deformation as a function of

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[2] Underlined numbers in parentheses refer to items in the list of references at the end of this paper.

drill bit position. Figure 1 shows the general configuration of the pilot borehole, deformation gage, and overcoring bit. Once the overcoring bit has been drilled far enough to relieve the stress on the core, the gage and the core are removed and the process is repeated along the length of the pilot borehole.

THE SINGLE-COMPONENT BOREHOLE DEFORMATION GAGE

The borehole deformation gage that was developed in 1962 was a single-component gage designed to fit in a 1-1/2-inch borehole. The single-component gage is shown in figure 2. The gage contacts the wall of the borehole at two

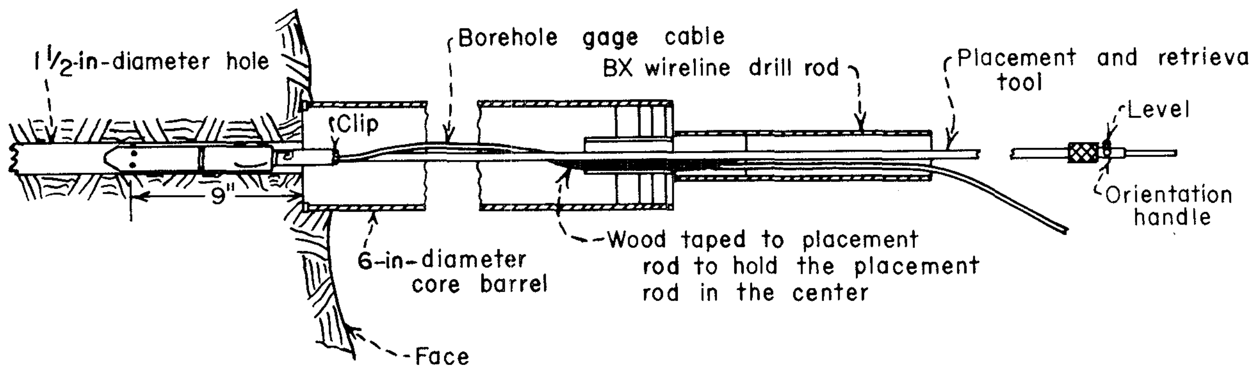

FIGURE 1. - Borehole Deformation Gage Being Positioned in Pilot Borehole.

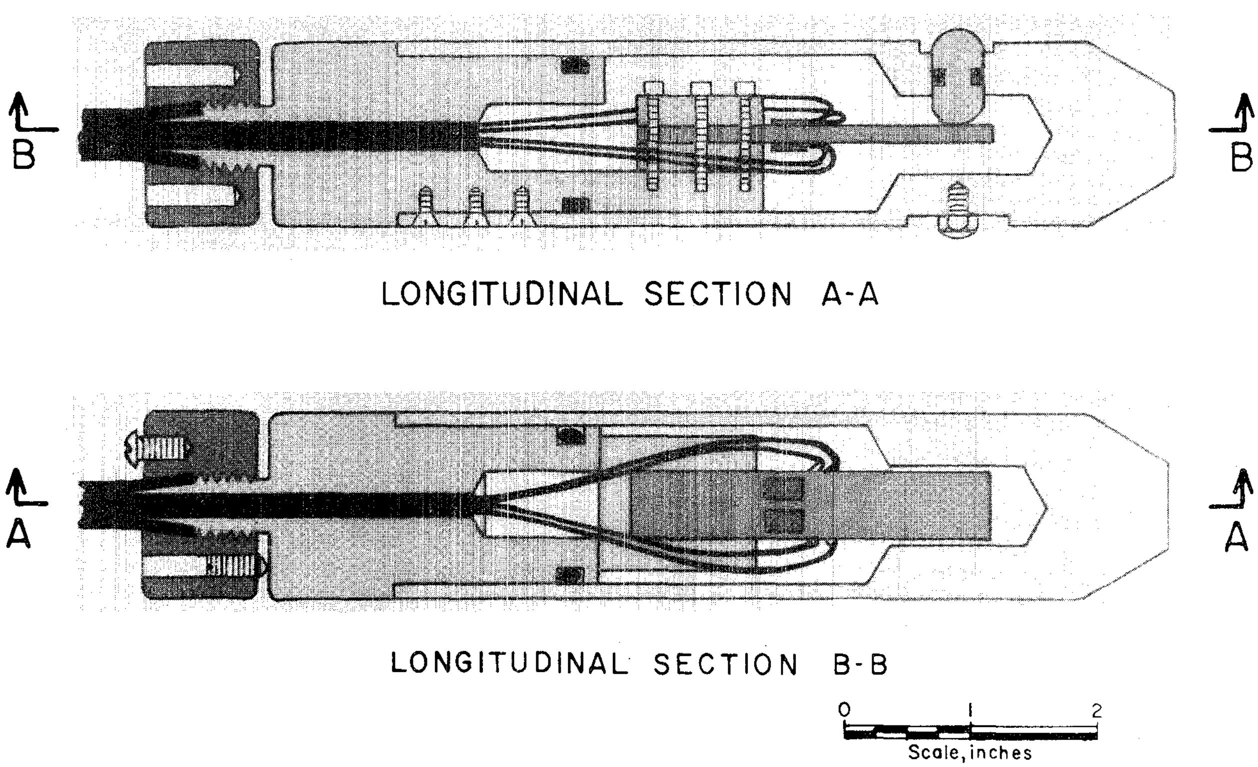

FIGURE 2. - Single-Component Borehole Deformation Gage.

points. One point of contact is a fixed stud and the other is a movable piston. As the borehole deforms during overcoring, the piston actuates the beryllium-copper cantilever on which four bonded resistance strain gages are mounted and connected to form a four-arm bridge. The bending strain produced in the cantilever is measured by the resistance strain gages and is read on a conventional resistance strain gage indicator. The gage is calibrated in such a way that a change in the measured strain, in the cantilever, corresponds to a known displacement of the piston. The cantilever also produces the contact force for the piston. Thus, the mechanical system has only two moving parts. The gage is constructed in such a way that it is both waterproof and dustproof. The single-component gage had an accuracy of 50 microinches and was 10 microinches per ° F temperature-sensitive.

In order to calculate the complete state of stress in the plane normal to the borehole, deformation measurements must be made across at least three different diameters. Therefore, the single-component gage must be positioned and overcored three times in order to calculate the stress in the plane normal to the borehole. This disadvantage led to the development of the three-component borehole deformation gage.

THE THREE-COMPONENT DEFORMATION GAGE

A 1967 report (4) described the construction and calibration of the three-component borehole deformation gage. This gage is shown in figures 3 and 4. The single cantilever in the original gage has been replaced by three pairs of beryllium-copper cantilevers. This arrangement allows three diametral deformation measurements, in the same plane, each time the gage is overcored.

To prevent wear, the piston assembly in this gage was modified to include a tungsten carbide insert. Variations in the diameter of the borehole are compensated for by the use of shim washers in each piston. Each of the

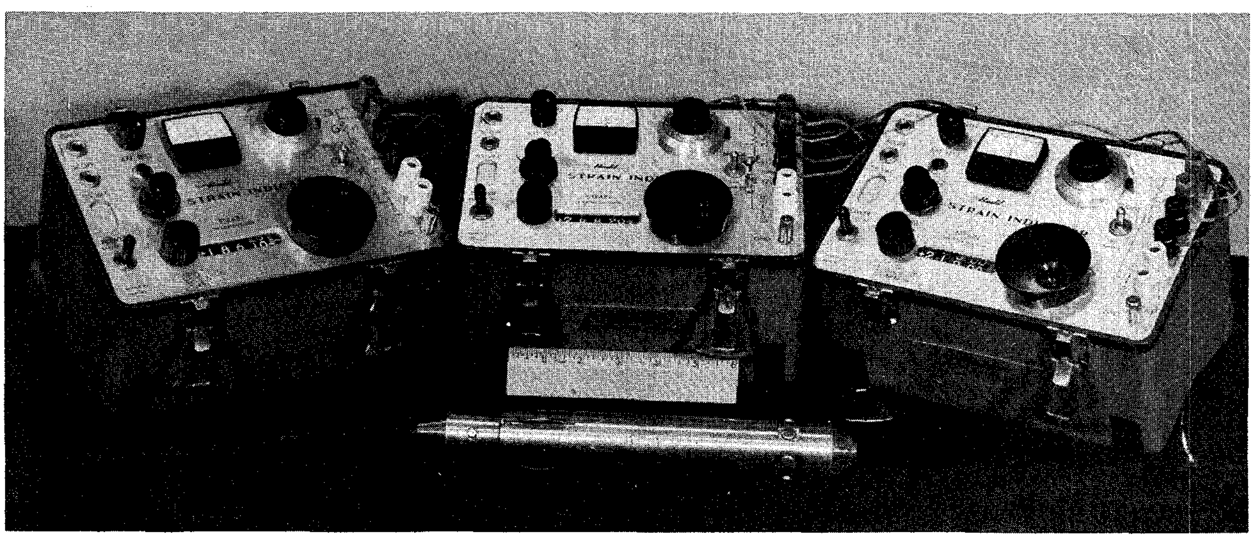

FIGURE 3. - Three-Component Borehole Deformation Gage and Strain Indicators for Each Component.

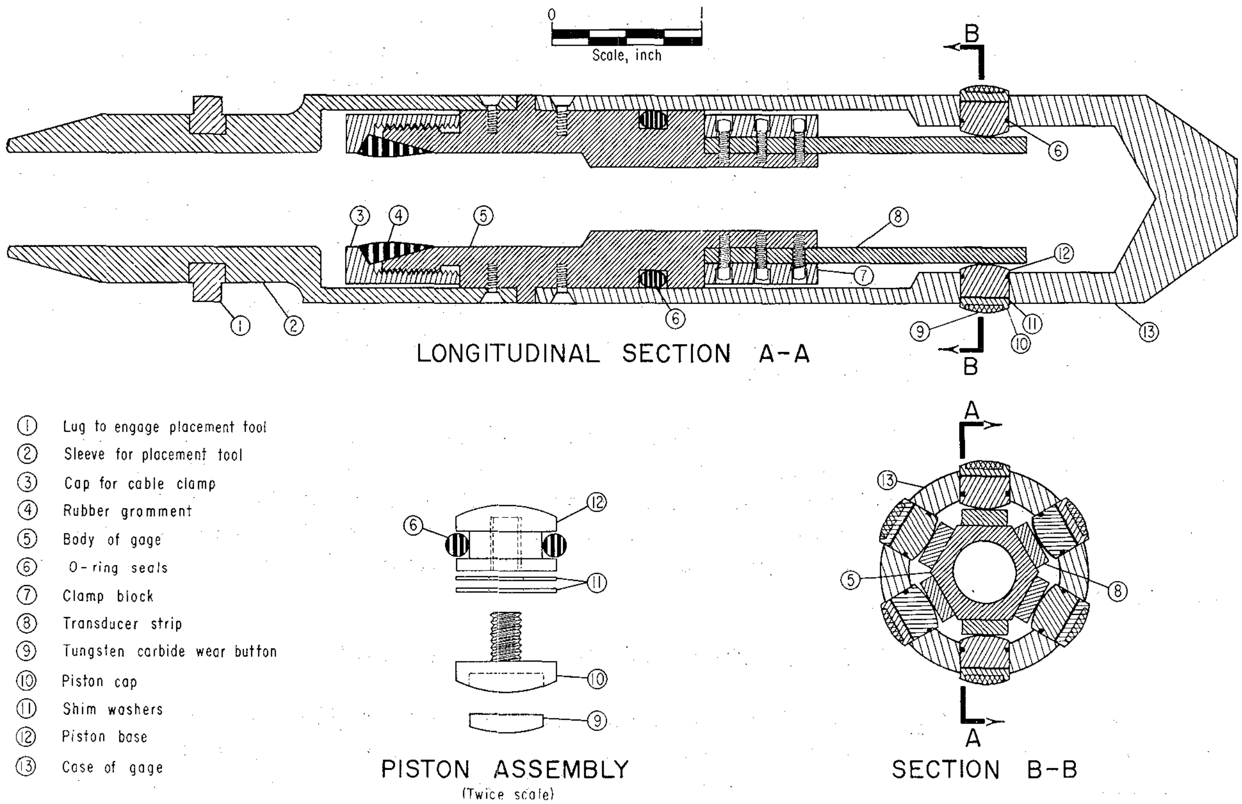

FIGURE 4. - Three-Component Borehole Deformation Gage.

six-cantilever strain transducers in this gage is bolted to the body of the gage with a clamp block and three cap screws. Two 1/8-inch strain gages are mounted on each cantilever strain transducer. The four strain gages on opposite transducers are connected to form a four-arm bridge. The accuracy of the three-component gage is 1 microinch across each diameter. The electrical resistance in each strain gage is accurately measured. The strain gages in opposite pairs of cantilevers are selected, based on the resistivity, such that the gage is less than 2 microinches per ° F temperature-sensitive.

As in many systems using strain gages, there is a certain amount of creep or drift in the deformation reading of each pair of cantilevers while under constant load. In an attempt to reduce drift, the back of the cantilever transducers and the transducer mount in the body of the gage were recently redesigned. This design is shown in figure 5. The cantilever strain transducer is still machined from a solid piece of beryllium-copper, but rather than bolted to the body of the gage, the back of the transducer is a machined, tapered pin which is drawn with a locknut into a tapered mount. Figure 6 is a photograph of a gage with the tapered transducer mount design.

Another recent modification that can be seen in figure 5 is the addition of a plug that extends between the cantilever strain transducers. This plug will not allow the cantilevers to be bent or permanently deformed.

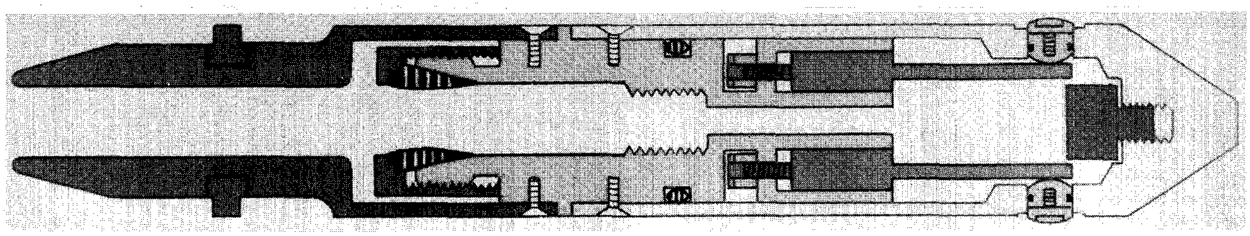

FIGURE 5. - Representation of the Three-Component Borehole Gage With Tapered Mounted Cantilevers.

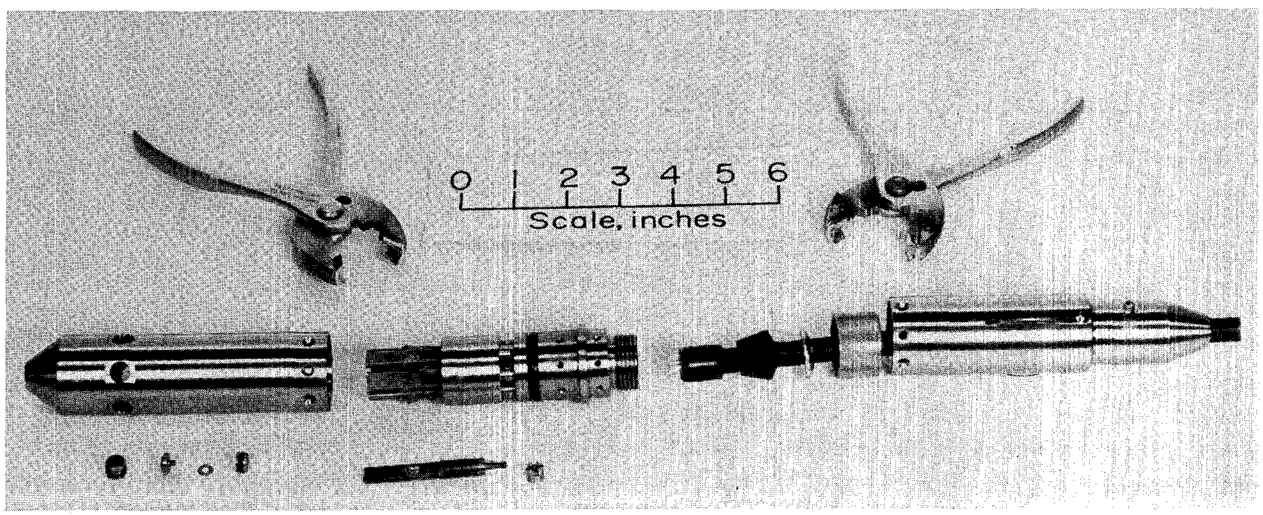

FIGURE 6. - Three-Component Gage With Tapered Mounted Cantilevers.

The drift, as a function of time, for a typical pair of bolted-down transducers is represented by the upper curve in figure 7. Although the majority of the drift has taken place within the first 10 minutes after the cantilevers are deflected, the curve does not level out within the 50-minute time interval. The lower curve in figure 7 is the plot of drift versus time for a pair of tapered-mounted cantilevers. If deformation measurements are taken after the gage has been in place for 10 minutes, all measurements can be used directly without regard to drift and time. With this degree of time stability, the gage can be left in a borehole and used to monitor stress changes over long periods of time.

BOREHOLE DEFORMATION DURING STRESS RELIEF

The process of borehole deformation during stress relief has been studied using finite-element methods. Figure 8 is a plot of the diametral deformation versus the position of the overcoring bit that has been reconstructed from a

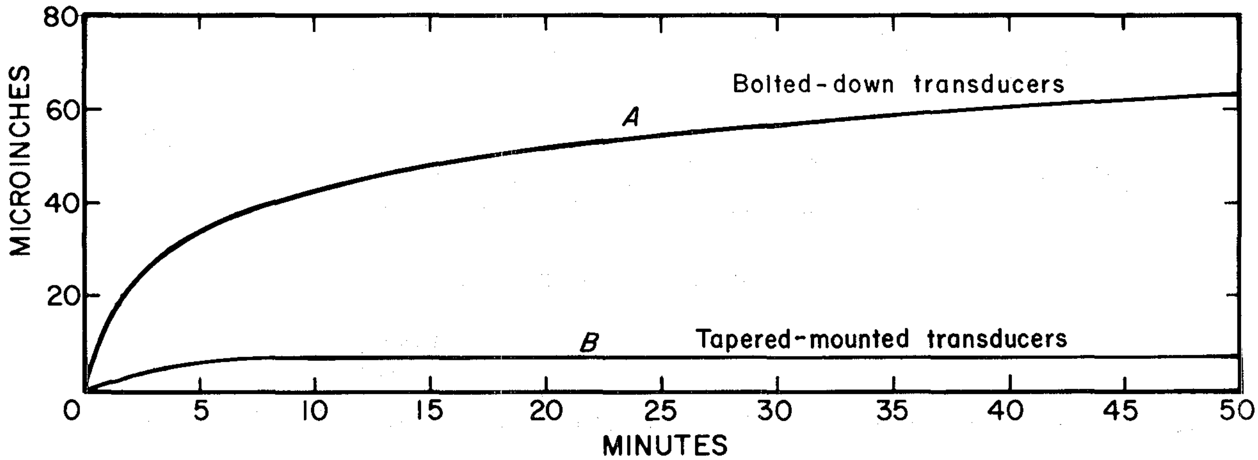

FIGURE 7. - Plot of Drift in Deformation Reading Versus Time for Different Types of Transducer Mount.

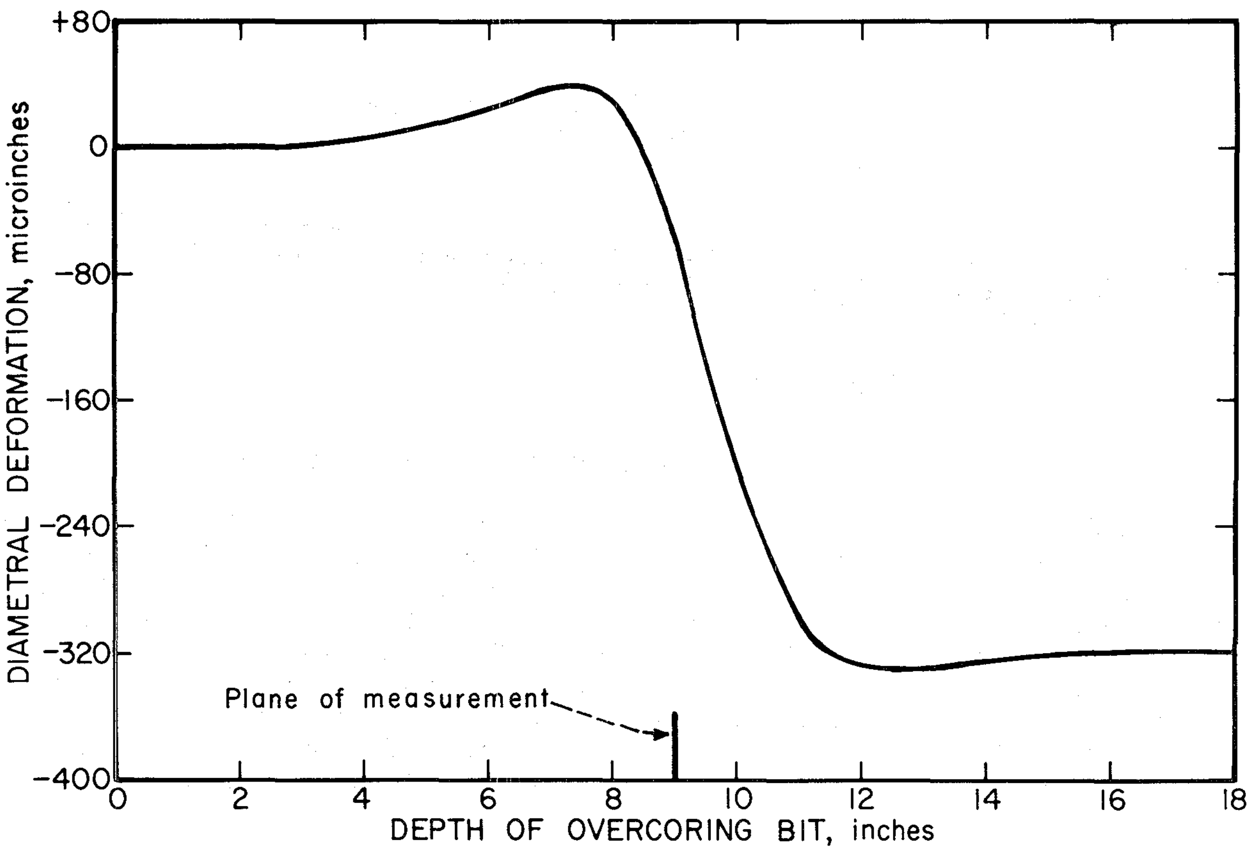

FIGURE 8. - Plot of Diametral Deformation Versus Depth of Overcoring Bit Taken From Finite Element Calculations With the Plane of Measurement at 9 Inches.

series of finite-element calculations. The plane of measurement, or the pistons of the borehole gage, was in this case 9 inches into the pilot borehole.

In figure 8 a negative deformation corresponds to an increase in the size of the diameter of the borehole. The deformation curve in figure 8 is due to axisymmetric compressive stress of 1,000 psi in a material where $E = 9.33 \times 10^6$ psi and $\nu = 0.2$.

The curves shown in figure 9 represent the three components of an in situ stress relief measurement. The plane of measurement for this set of curves is also at 9 inches.

The magnitude of the stress relief measurement across each diameter is simply the difference between the indicator reading before overcoring and the indicator reading taken once the overcoring bit is sufficiently past the gage (that is, once the curve has leveled out). In practice, however, it is advisable to continuously monitor the deformations during overcoring, for if the core breaks, the overcoring bit must be stopped to prevent damage to the gage or the readout cable.

RECENT MODIFICATIONS IN GAGE DESIGN

If the borehole deformation gage is being used in unfractured rock that will core well, the deformation gage is usually set such that the plane of measurement is 9 inches into the borehole. The overcoring bit is then drilled 9 inches past the plane of measurement. In fractured rock, or rock that is disking, this ideal 18-inch run may be impossible to obtain. The problem of obtaining stress relief deformation measurements under such conditions has led to the development of a reversed borehole deformation gage. The reversed gage, shown along with the regular borehole deformation gage in figure 10, uses the same cantilever transducer arrangement to measure borehole deformations. However, the cantilever transducers are reversed 180° with respect to the body of the gage. This type of gage construction allows the gage to be positioned in the borehole in such a way that the plane of measurement may be closer than 6 inches to the front of the pilot borehole. The shape of the borehole deformation versus drill depth curve is dependent on the position of the gage relative to the front of the pilot borehole.

The relief curve in figure 11 is constructed from the same set of finite-element calculations as is figure 8. The plane of measurement is at a depth of 1 inch in the pilot borehole. The finite-element model was designed to simulate overcoring at a free surface and overcoring at depth in a hole. Figure 11 represents overcoring at a surface. If overcoring is done at depth in a hole and the plane of measurement is closer than 6 inches to the front of the existing pilot borehole, the magnitude of the relief is a measure of both the applied stress and the stress concentration due to the influence of the 6-inch-diameter hole. The magnitude of the deformation is therefore dependent on the position of the gage. This position dependency has been determined using finite-element methods and verified during field tests of the gage.

Figure 12 is a plot of data taken during field tests of the reversed gage with the plane of measurement 1 inch from the front of the pilot borehole. With the gage in this position, a complete relief curve can be obtained in a much shorter length of overcore than was previously required. Since the

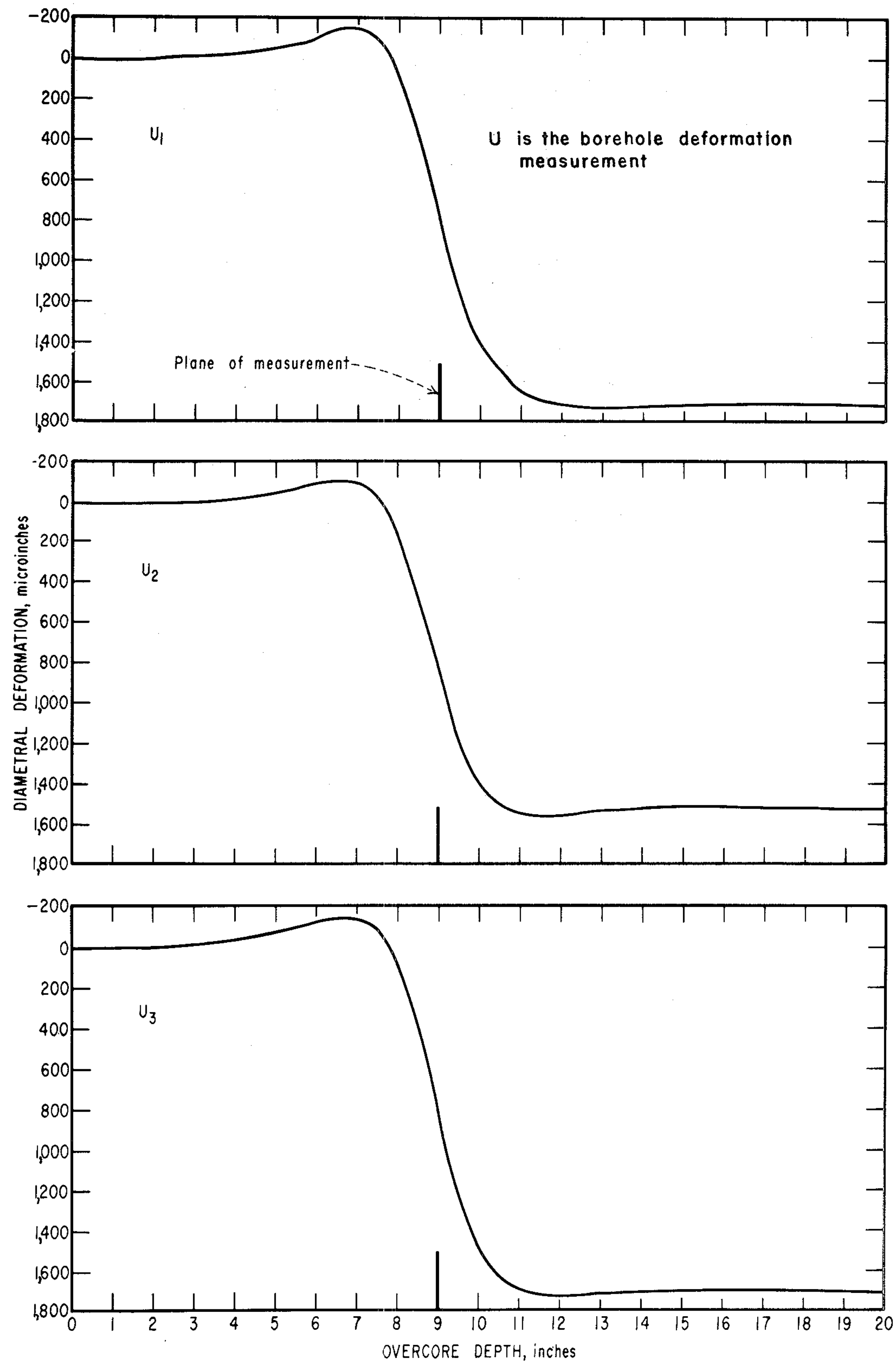

FIGURE 9. - The Three Components of a Stress Relief Measurement Taken From Field Data.

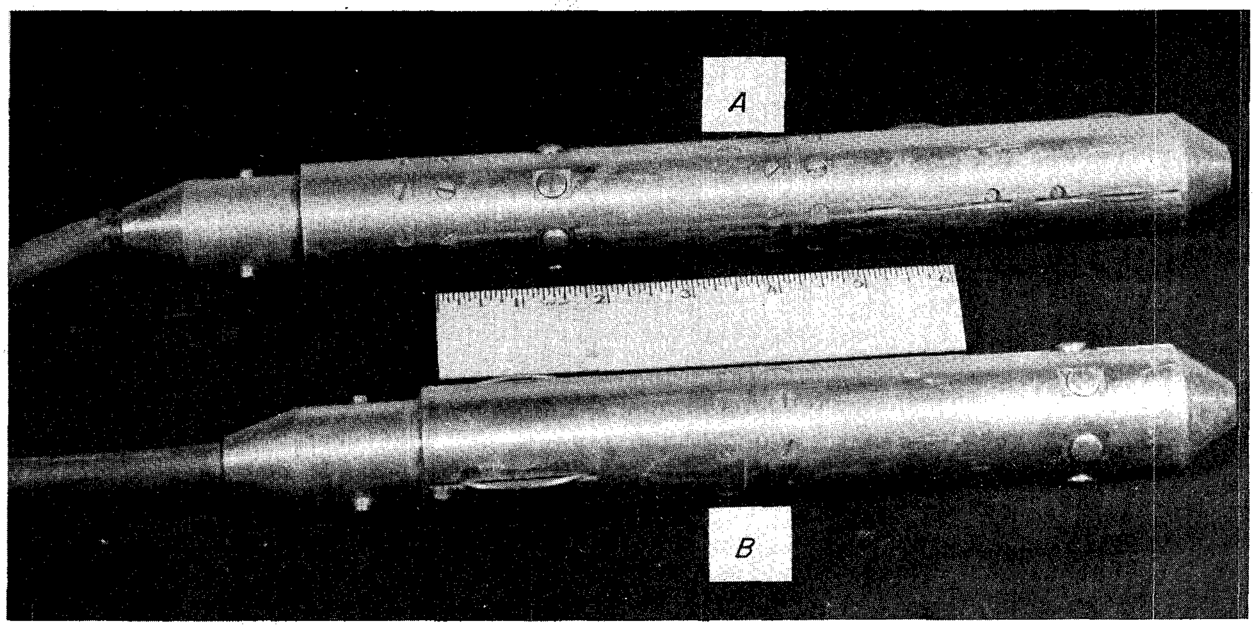

FIGURE 10. - *A,* The Reversed Borehole Gage. *B,* The Regular Gage.

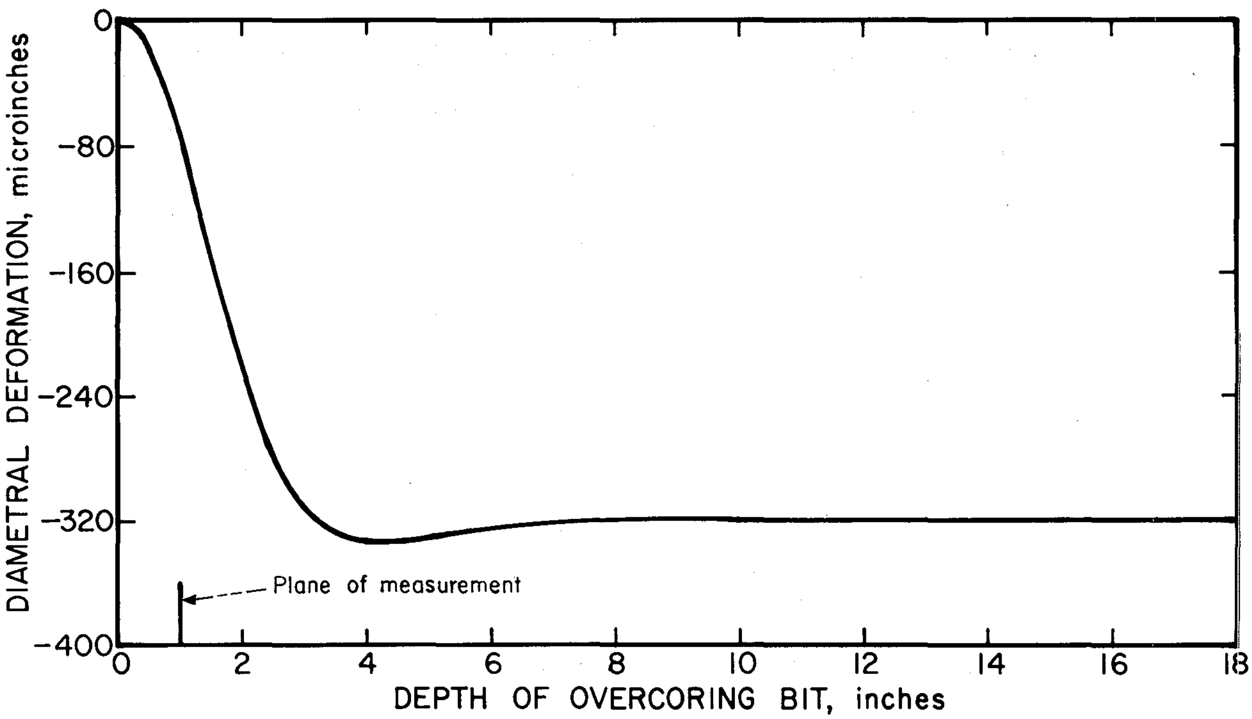

FIGURE 11. - Plot of Diametral Deformation Curve Reconstructed From Finite Element Calculations With the Plane of Measurement at 1 Inch.

reversed gage was field-tested, it has been used successfully to obtain stress relief measurements in an area in which core lengths over 10 inches were unobtainable.

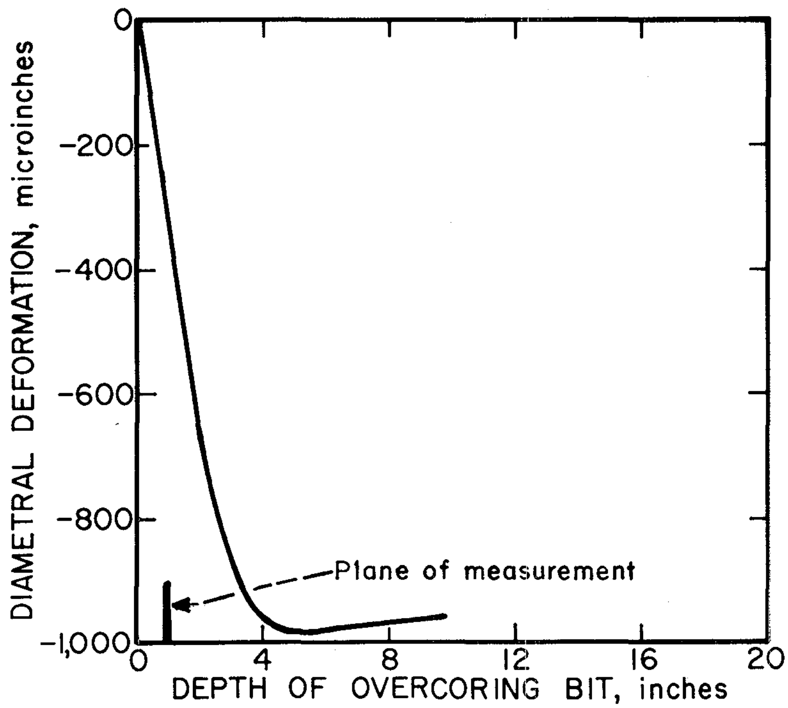

FIGURE 12. - Stress Relief Curve Taken From Field Tests of the Reversed Gage With the Plane of Measurement at 1 Inch.

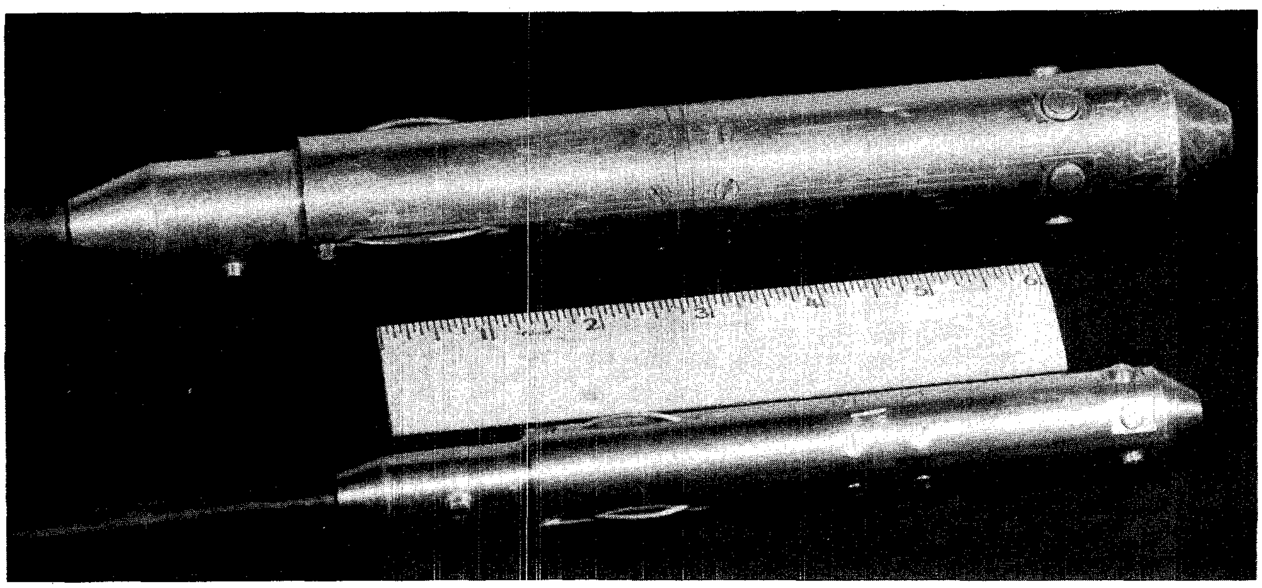

FIGURE 13. - The 1½-Inch-Diameter Borehole Gage and the 1-Inch-Diameter Borehole Gage.

A three-component borehole deformation gage designed to fit in a 1-inch-diameter borehole has been constructed and calibrated. This gage and the 1-1/2-inch-diameter gage are shown in figure 13. The use of this smaller gage will reduce drilling costs, for it will allow overcoring with a smaller bit and the lengths of the core will not need to be as long as those required when using the 1-1/2-inch-diameter borehole gage. The 1-inch gage will be used in medium- to fine-grained materials.

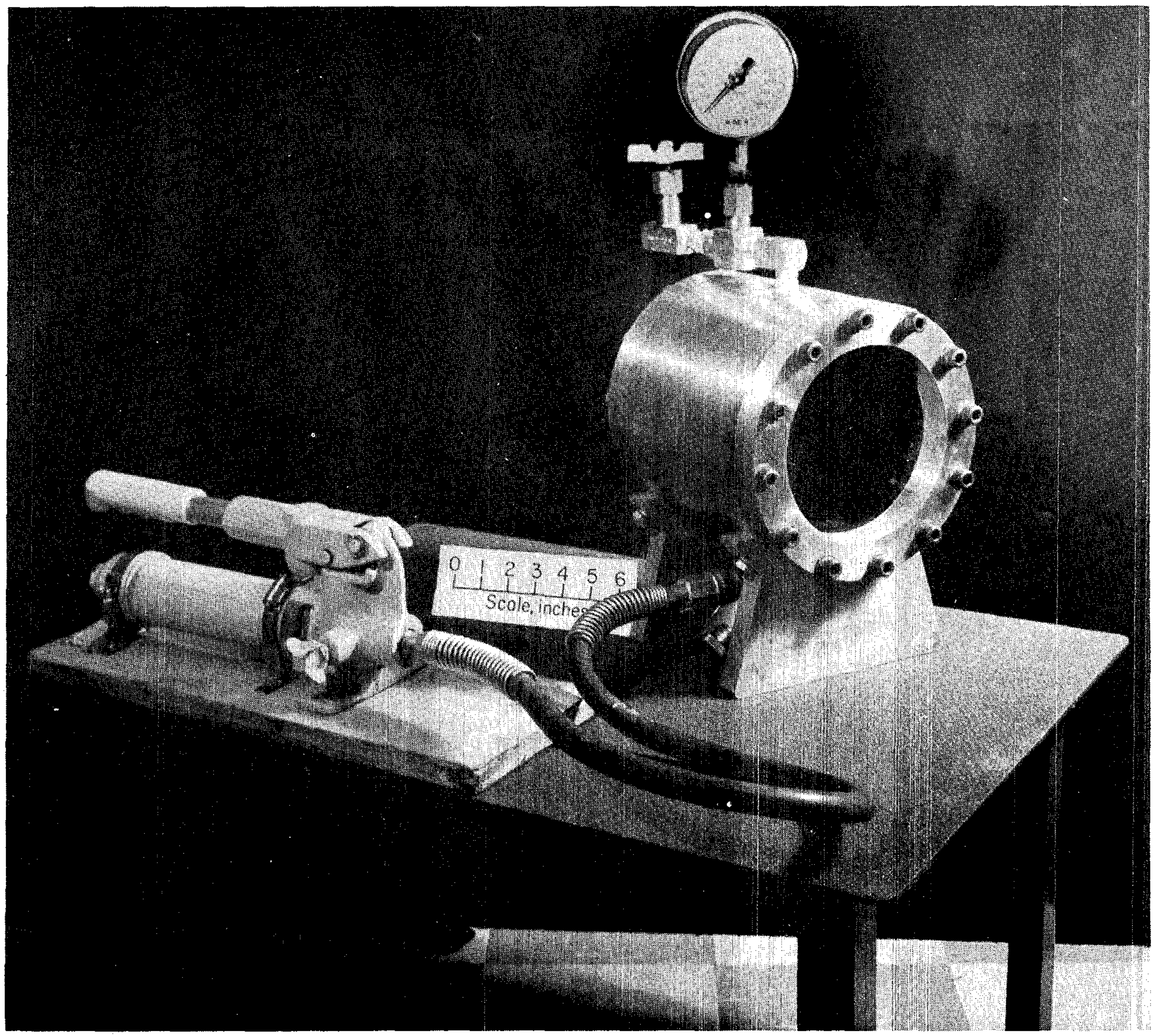

FIGURE 14. - Biaxial Pressure Cell.

STRESS CALCULATED FROM BOREHOLE DEFORMATION MEASUREMENTS

After in situ stress relief measurements have been obtained, the borehole deformation gage is used in conjunction with the biaxial pressure cell to determine the elastic modulus and the degree of anisotropy in the rock. The biaxial pressure cell is shown in figure 14. The biaxial cell has a neoprene sleeve that transmits a lateral load to the rock core. Resultant displacements are measured in the borehole with the deformation gage (6).

Once the borehole deformation gage has been used to obtain the three in situ diametral deformations and the elastic modulus of the rock, the stress in the plane normal to the borehole can be calculated using the following equations (1):

$$P + Q = \frac{E}{3d(1-\nu^2)} (U_1 + U_2 + U_3);$$

$$P - Q = \frac{\sqrt{2}E}{6d(1-\nu^2)} [(U_1-U_2)^2 + (U_2-U_3)^2 + (U_1-U_3)^2]^{1/2};$$

$$\tan 2\theta = - \frac{\sqrt{3}(U_2-U_3)}{2U_1-U_2-U_3}.$$

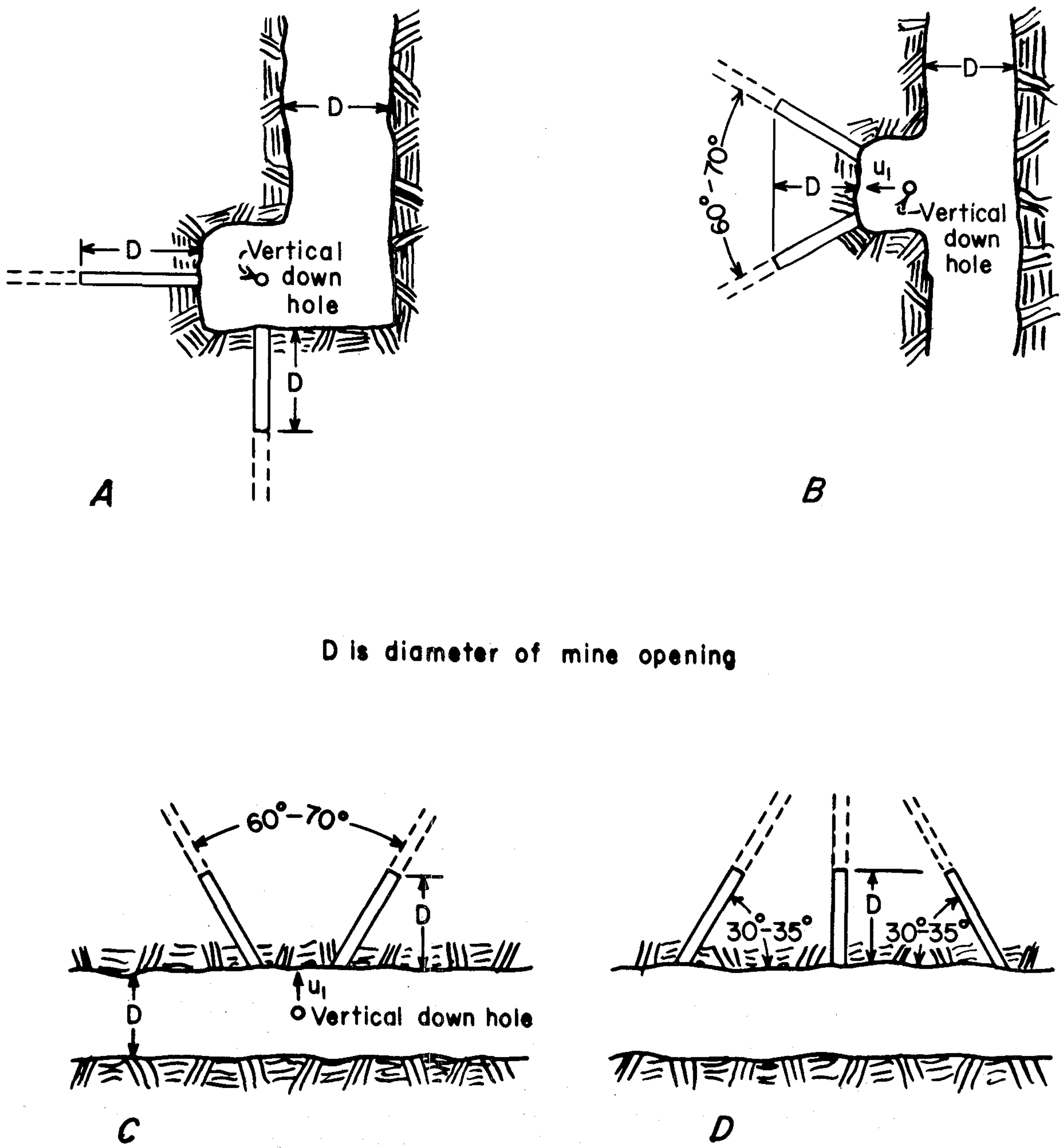

FIGURE 15. - Recommended Borehole Configurations.

In the equations given above, U_1, U_2, and U_3 are the three borehole deformation measurements, E is the elastic modulus, ν is Poisson's ratio, d is the diameter of the pilot borehole, and P and Q are the maximum and the minimum secondary principal stresses in the plane. The angle θ is the angle from the deformation U_1 to the maximum stress P. These equations can be used if the rock is relatively isotropic. If this is not the case, another system of equations must be used (1).

In order to calculate the complete three-dimensional state of stress, deformation measurements must be made in three nonparallel boreholes. Some of the recommended borehole configurations are shown in figure 15. If stress concentrations around an opening or in a pillar are of interest, overcoring and deformation measurements can be taken throughout the length of the hole. However, in order to determine the complete three-dimensional state of stress in an area, stress relief measurements must take plane outside the zone of influence of the opening. This distance is indicated by the distance D in figure 15.

After borehole deformation measurements have been made in three nonparallel boreholes, all components of the three-dimensional stress field can be calculated in any coordinate system (8). The three components of the principal stress system can also be calculated relative to any coordinate system.

SUMMARY

The Bureau of Mines borehole deformation gage as it now exists is an accurate, temperature-insensitive, time-stable device that can be used to measure in situ stress. Borehole deformation theory and stress measurement calculations have been advanced to include the effects of rock anisotropy (1), temperature variations (3), and the topography of the overburden (2). The borehole deformation gage can be used to measure stress in any rock or elastic material that can be cored with a diamond drill. The length of the core required to obtain a complete stress relief measurement has been reduced by recent modifications in gage design.

Accurate and reliable stress measurement methods and equipment are essential to all aspects of ground control, finite-element studies, and mine design in general.

REFERENCES

1. Becker, Robert M., and Verne E. Hooker. Some Anisotropic Considerations in Rock Stress Determinations. BuMines Rept. of Inv. 6965, 1967, 23 pp.

2. Hooker, Verne E., David L. Bickel, and James R. Aggson. In Situ Determination of Stresses in Mountainous Topography. BuMines Rept. of Inv. 7654, 1972, 19 pp.

3. Hooker, Verne E., and Wilbur I. Duvall. In Situ Rock Temperature Stress Investigations in Rock Quarries. BuMines Rept. of Inv. 7589, 1971, 11 pp.

4. Merrill, Robert H. Three-Component Borehole Deformation Gage for Determining the Stress in Rock. BuMines Rept. of Inv. 7015, 1967, 38 pp.

5. Merrill, Robert H., and Jon R. Peterson. Deformation of a Borehole in Rock. BuMines Rept. of Inv. 5881, 1961, 32 pp.

6. Obert, Leonard. Triaxial Method for Determining the Elastic Constants of Stress Relief Cores. BuMines Rept. of Inv. 6490, 1964, 22 pp.

7. Obert, Leonard, Robert H. Merrill, and Thomas A. Morgan. Borehole Deformation Gage for Determining the Stress in Mine Rock. BuMines Rept. of Inv. 5978, 1962, 11 pp.

8. Panek, Louis A. Calculation of the Average Ground Stress Components From Measurements of the Diametral Deformation of a Drill Hole. BuMines Rept. of Inv. 6732, 1966, 41 pp.

MINE INSTALLATION OF TWO BUREAU OF MINES HYDRAULIC PRESSURE CELLS AND A BOREHOLE DEFORMATION GAGE

by

Reed L. Smith[1]

ABSTRACT

The installation of two Bureau of Mines hydraulic pressure cells and a borehole deformation gage in mine drill holes is described.

One hydraulic pressure cell, a cylindrical pressure cell, is used to determine the modulus of rigidity of rock in situ. The other cell, a flat borehole pressure cell, detects changes of rock pressure in the direction normal to the plane of the pressure cell.

The borehole deformation gage measures deformation of a borehole across three diameters at intervals of 120°.

INTRODUCTION

The borehole deformation gage consists of three dial indicators mounted in a steel frame in such a way as to measure deformation of a borehole across three diameters at intervals of 120°. The gage is designed to be installed in a 6-inch-diameter hole. The readings are taken by sighting through a telescope.

The cylindrical pressure cell (CPC), 8 inches long, operates in a 1.50-inch-diameter borehole. (No cement is used.) The primary use for the CPC is to determine the modulus of rigidity of rock in situ, over the pressure range 1,000-7,000 psi. Correction for the compressibility of the system is determined by dilating the CPC in two metal calibration cylinders just prior to the tests in the borehole.

The flat borehole pressure cell (BPC), 8 inches long, operates in a 2.25-inch-diameter borehole. The space around the BPC is completely filled with mortar (no voids). The primary use for the BPC is to detect changes of rock pressure in the direction normal to the plane of the pressure cell. The BPC is not recovered after use.

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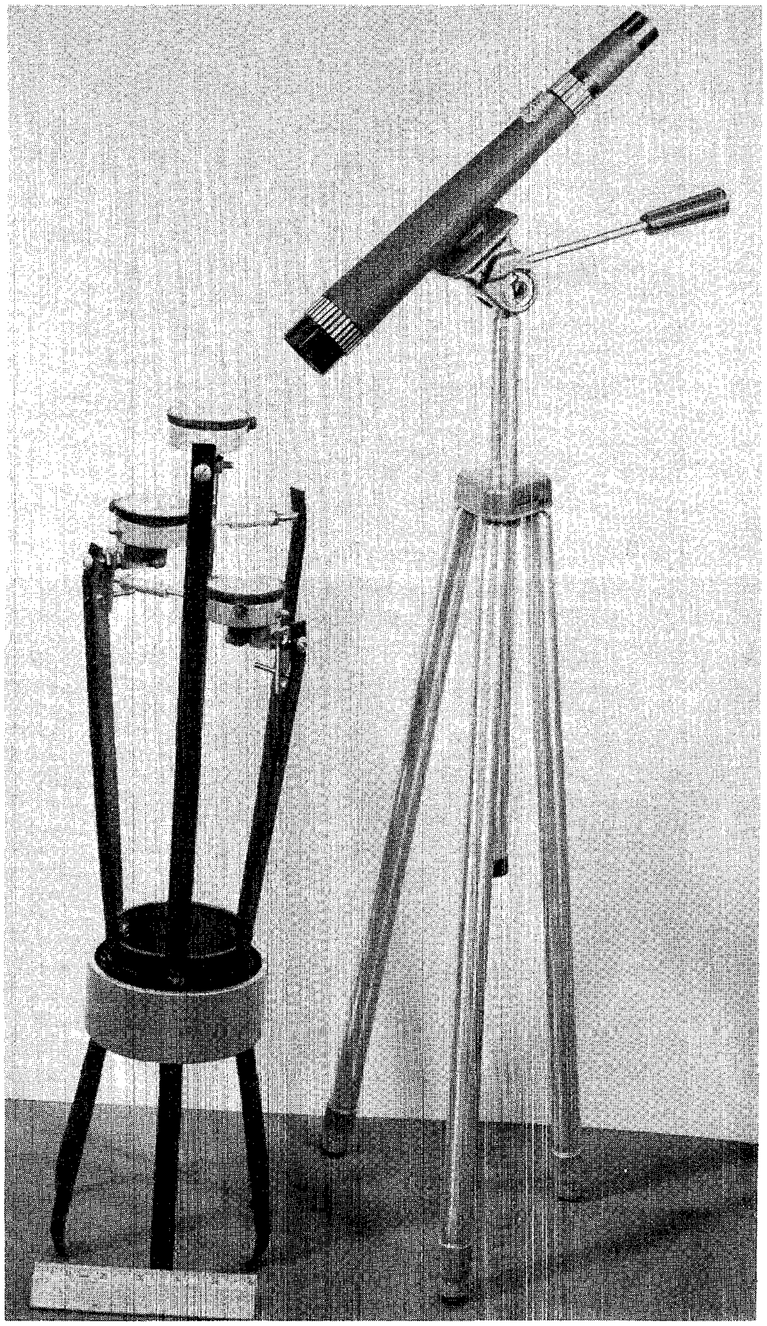

FIGURE 1. - Mechanical Borehole Deformation Gage and Telescope.

A compact type of fluid meter/pump is used to inject fluid (usually glycerin) into either pressure cell to obtain the pressure/volume relationship for the in-place pressure cell.

The installation of these instruments requires very little technical knowledge and becomes routine after the first few are installed. These instruments are very simply constructed and yet rugged enough to withstand the conditions encountered in a mine; they have been used in both metal and nonmetal mines.

MECHANICAL BOREHOLE DEFORMATION GAGE

This borehole deformation gage (fig. 1) is a mechanical gage as opposed to the gage described this morning by Mr. Aggson. Although both gages have the same function, to measure diameter change in a borehole, the mechanical gage was devised primarily to study creep properties of rock; that is, diameter changes in a hole 6 inches in diameter (or larger) over a period of a few months or years without having to be attended, except to take the readings. The gage consists of three dial indicators mounted in a steel frame and oriented to measure change of diameter in a borehole in three directions 120° apart. The dial indicators used in our most recent tests had a range of 1/2 inch and could be read to 0.0005 inch without interpolation between the divisions. The metal ring around the outside of the frame is used to compress the contact points of the dial indicators by pulling the ring toward the indicators. This facilitates easy insertion or extraction of the gage from a rough hole such as in coal.

Before being transported to the test site, the gage is completely assembled and preset in the lab as much as possible for the size hole in which it will be installed. The gage is installed by pushing it into the hole with a metal rod that is also used to orient the gage within the hole. Since the gage is usually oriented so that one dial indicator is measuring in the vertical direction, the gage can be oriented by alining the vertical element with a plumb bob string which is suspended across the collar of the hole. The readings are then taken by sighting through a telescope, using a cap lamp as a light source. Some adjustment of the gage may be necessary to obtain the desired initial reading. After the initial reading is obtained, the installation of the borehole deformation gage is complete.

HYDRAULIC BOREHOLE PRESSURE CELLS

We use two types of hydraulic borehole pressure cells, a cylindrical cell and a flat cell. Both types of borehole pressure cells are designed to be inserted into a hole drilled in rock and then dilated by forcing fluid into the cell.

The cylindrical cell, which is installed in a 1.5-inch-diameter hole, without grout, consists of a steel core and a copper jacket 8 inches long. The cell and its ancillary apparatus is shown to the left in figure 2. For comparison, the flat borehole pressure cell is shown in the upper right-hand corner. The primary use of this cell is to determine the modulus of rigidity of rock in situ over a pressure range of 1,000-7,000 psi. Correction for the compressibility of the system is determined by dilating the cell in two metal calibration cylinders just prior to dilating it in the borehole.

The flat pressure cell is installed in a 2-1/4-inch-diameter hole and either grouted in place at the test site or encapsulated with a sand-cement mortar and allowed to cure in the lab prior to being taken to the test site. Figure 3 shows the flat pressure cell before and after being encapsulated in the lab. The flat borehole pressure cell is also 8 inches long and is primarily used to detect changes of rock pressure in the direction normal to the plane of the pressure cell. To convert an observed change of fluid pressure in the flat borehole pressure cell to the corresponding change of ground pressure, it is necessary to determine the response of the pressure cell in that particular rock medium. This can be done by several methods. If the rock cores easily, the most convenient procedure may be to install a flat pressure cell inside a 6-inch-diameter core, which is then subjected to varying loads in a laboratory triaxial loading device. If coring is difficult or inconvenient, a large (16 to 24 inches square) hydraulic pressure cell or flat jack may be installed in a slot to simulate a known change of ground pressure within about 6 inches from an installed flat borehole pressure cell. Either method determines the ratio of the change of fluid pressure in the cell to the corresponding change of pressure in the rock. At the time of installation in rock, the pressure cycling of the borehole pressure cell (part of the normal installation procedure, described below) indicates the quality of the installation. For example, in the event of a poor grouting job, the slope of the pressure versus volume curve for a particular cell will be abnormally low compared with other tests in the same material.

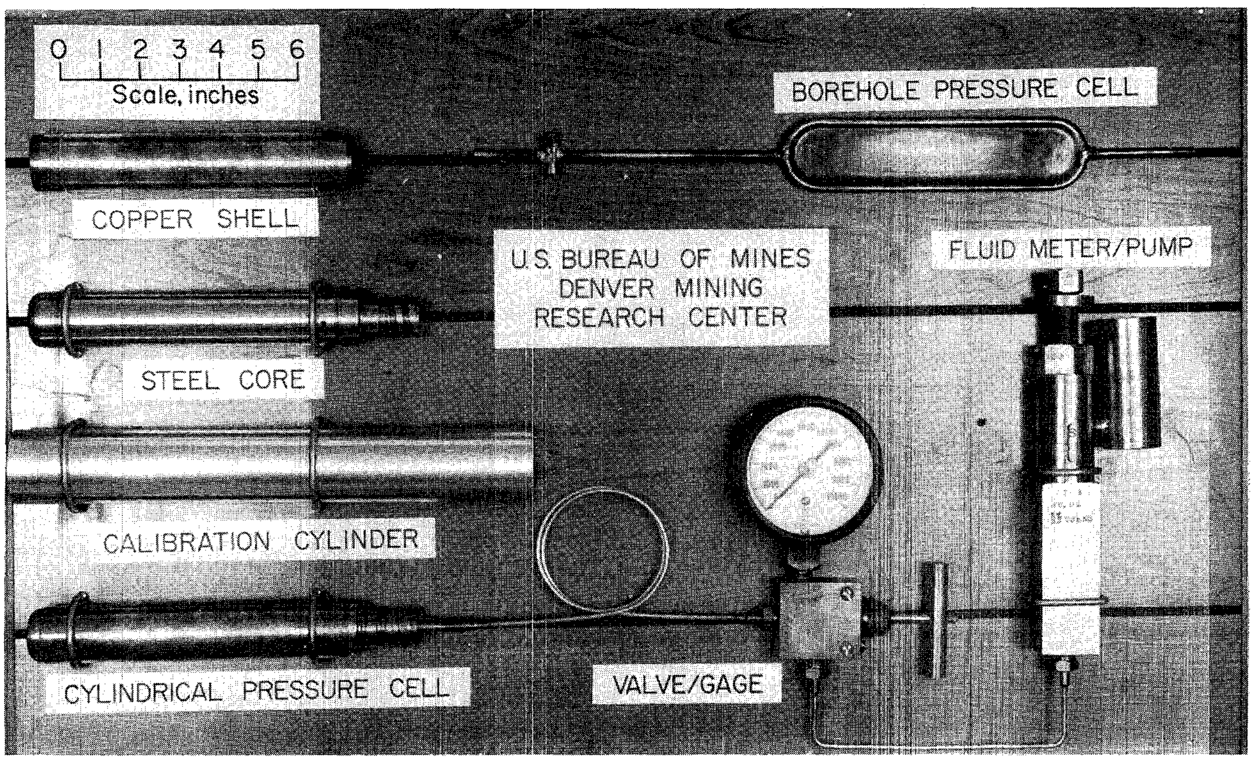

FIGURE 2. - Hydraulic Borehole Pressure Cells and Ancillary Apparatus.

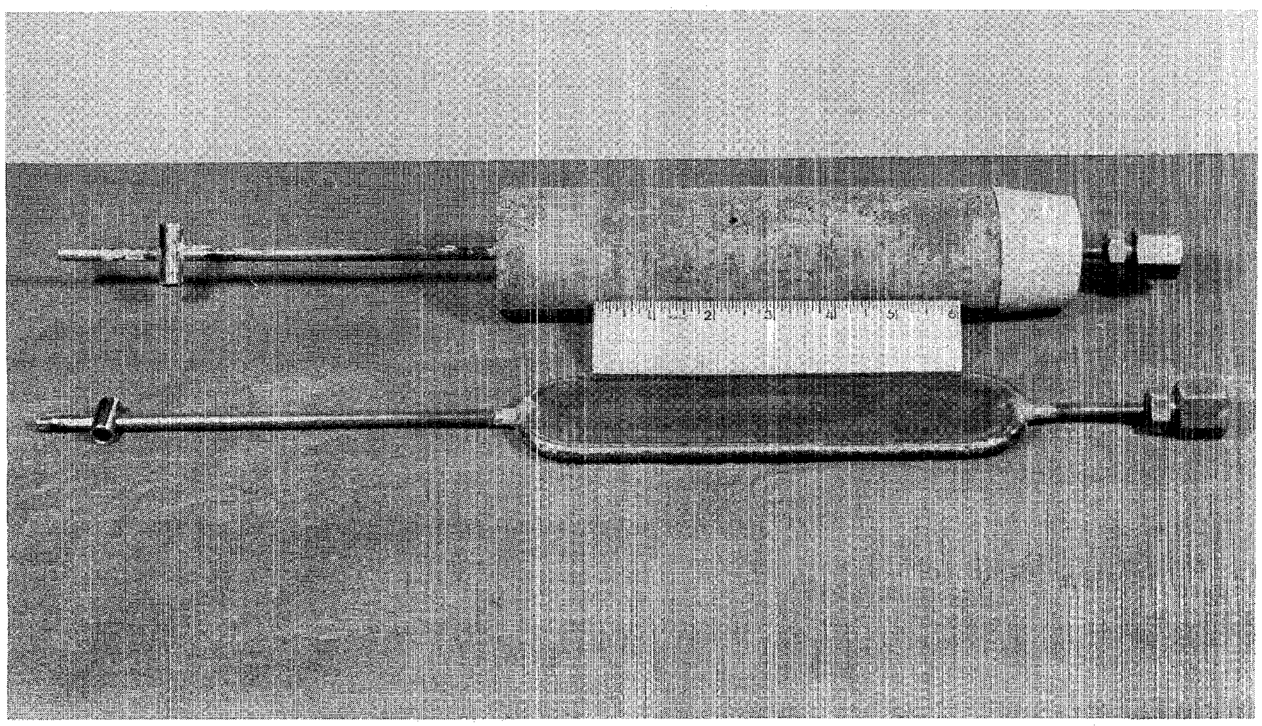

FIGURE 3. - Encapsulated and Plain Version of Flat Borehole Pressure Cell.

The same hydraulic system is used to pressurize both the cylindrical and the flat cell. The pressurizing system consists of the tubing, the valve/gage unit, and the fluid meter/pump. (See fig. 2.) The fluid meter/pump is a very compact unit which is used to inject the fluid into the pressure cell by turning the threaded piston into the body of the pump. On the piston is mounted a turns counter which enables one to count the number of revolutions of the piston to the nearest 100th turn, which represents approximately 0.002 in^3 of fluid.

Before taking the cells and equipment to the field, the correct lengths of tubing (as determined by the instrumentation plan) are silver-soldered to each of the pressure cells. The cells, complete with tubing, and the pressure gages, which will eventually be connected to the cells, are filled with fluid in the lab. The Bourdon-tube pressure gages are then calibrated against a pressure transducer to improve the accuracy of the gage readings. These gages are relatively inexpensive, and they are recovered and reused after each installation.

MINE INSTALLATIONS

The pressure cells, gages, and fluid meter/pump are packed separately for transportation to and assembly at the test site. At the test site it is usually necessary to scale the roof and ribs in the immediate working area and remove all the loose rock from the area. Once this is done, the test holes are marked for the drillers and an area is set up for assembling and calibrating the instruments. Usually the first hole is a practice hole for the drillers, because we have found that the hole size for a given bit and drill speed will vary considerably from one material to another and even from one area to another in the same material. It sometimes takes several tries to come up with a combination of a proper drill bit, drill speed, and possibly a reamer that will yield the proper size hole. However, once a technique is developed, the drillers (usually one experienced driller and his helper) can easily keep ahead of two instrument installers (for test holes of 20 feet depth or less). The diameters of the holes are measured with a special set of calipers to ±0.005 inch.

Figure 4 shows a cross section of the holes that are used for both the flat and cylindrical pressure cells. A flat cell that is to be grouted in place requires only a rough hole 2-1/4 inches in diameter (hole A). The end of the hole is completely filled (no voids) with a cement-sand mortar and the cell is then forced into the mortar. Since the mortar requires a 28-day curing period, one may find it necessary to use the pre-encapsulated cell, which can be pressurized immediately on installation. However, the pre-encapsulated cell does require a more precise hole (2.240 inches ±0.060 in diameter) in the portion of the hole where the cell will be located. The first portion of the hole can still be drilled rough as shown for hole B. The hole for the cylindrical cell (hole C) is drilled in very nearly the same manner as the hole for the encapsulated cell, except that the test portion of the hole is drilled to a diameter of 1.500 inches ±0.015 and the hole must not have open joints or fractures so wide that the copper shell extrudes into them.

In a recent test series, two encapsulated flat cells and one cylindrical cell were installed in the same hole (hole D). One of the two encapsulated cells was oriented to measure change in vertical stress and the other, change

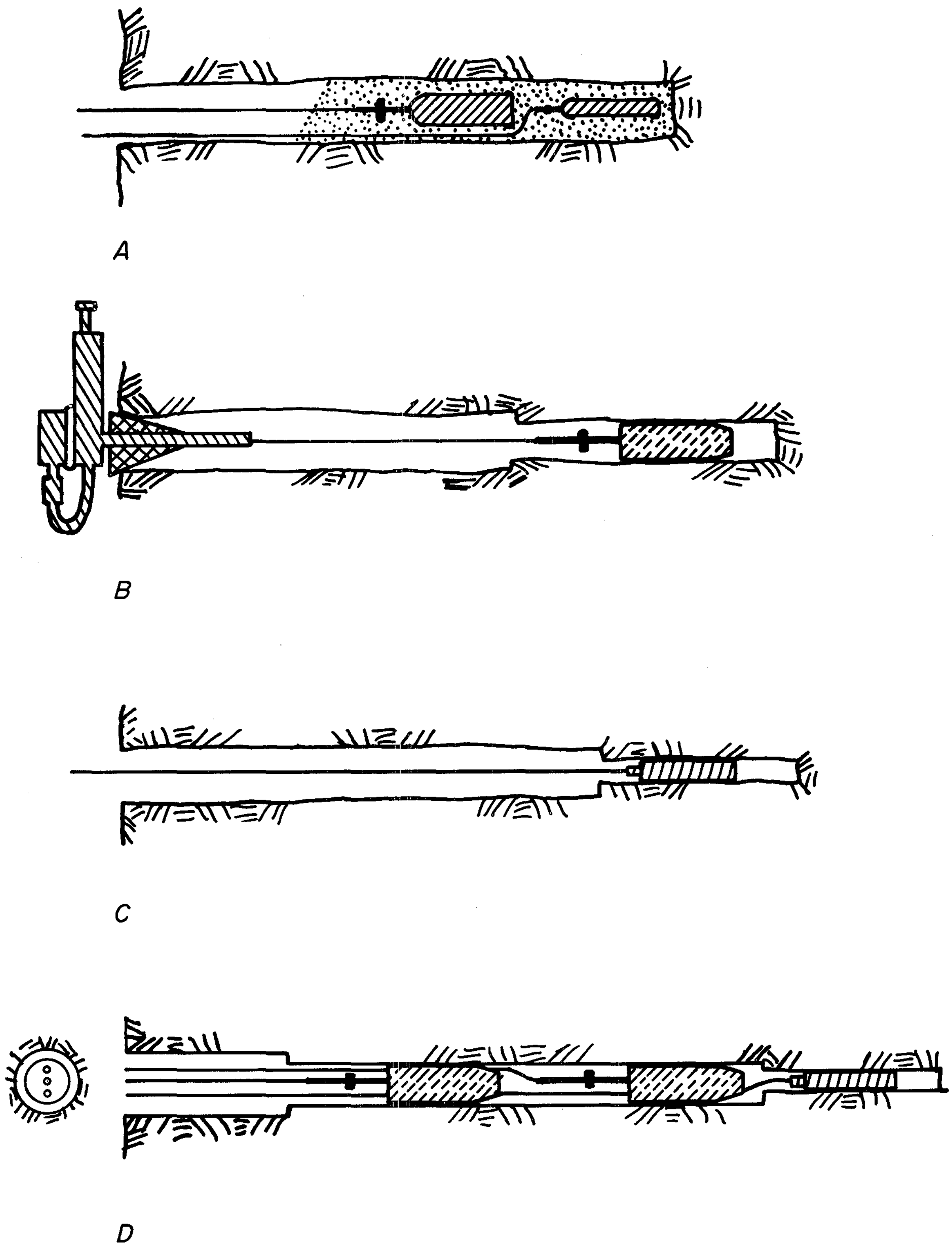

FIGURE 4. - Drill Hole Cross Sections for Several Pressure Cell Installations.

in horizontal stress. Although this procedure requires a more precise hole in the portion containing the encapsulated cell, since that portion of the hole is longer, the method reduces the total drilling required.

After the holes have been drilled to the proper size and checked with the calipers, the borehole pressure cells are inserted and oriented within the hole (usually to measure either horizontal or vertical stress). The flat borehole pressure cell is oriented by means of a rigid rod which attaches to an orientation block immediately behind the cell and which extends out beyond the collar of the hole. The cell is alined by observing a mark on the rod outside the hole. No orientation is required for the cylindrical cell, since it is equally sensitive to stress change in all directions perpendicular to its axis.

At this point, the pressure gages and fluid pump are connected to the cell and the pressure-cycling is begun. The mounting of the pressure gages and the procedure for pressure-cycling the cylindrical and flat cells are identical. The pressure gages and pump are fastened to the mounting bracket with bolts, and the bracket complete with gages is secured to the rib, roof, or floor by wedging the bracket into the collar of the test hole. (See fig. 4, hole B.) The tube from the pressure cell is then connected to the pressure gage and the system is completely filled with fluid from the pump reservoir. Air remaining in the system is then bled off and the system is ready to be pressure-cycled.

The pressure-cycling with either the cylindrical or the flat cell consists of a loading, unloading, and a second loading. The cycling is done at a rate of 200 psi per minute, with readings taken at each 1-minute interval. The readings consist of a pressure reading and a turns reading on the pump; this represents the volume of fluid injected into the cell. At the end of the second loading in rock, the modulus of rigidity test is complete, and the cylindrical cell could be removed or relocated in the same hole for another test. However, the cylindrical cell, like the flat cell, can be left in place to observe the trend of pressure versus time. In this case, the valve is closed to set the pressure and the fluid meter/pump is removed and used to pressurize another cell.

When the gages are to be left in place and the pressures have been set, readings are taken at varying intervals of time. For example, during the first hour after the pressures have been set, the readings are taken every few minutes; then as the pressure begins to stabilize, the readings are taken on a weekly or monthly basis by the mine company personnel. For all of our installations which are left in place over a long period of time, arrangements are made with the mine to have one of their employees record the gage readings, usually on a weekly basis. At the end of the installation about 1 hour is taken to go over the procedure for reading the gages with the person designated by the company to take the readings; this person is then accompanied while he takes one complete set of readings, to see that he knows where all the gages are and how to read them. After taking one set of readings he should have no difficulty taking the readings thereafter. The gages are left in place for whatever length of time is required to complete the test. In our tests, we have left the gages in for periods ranging from a few months to a few years.

Some use has been made of spring-wound, continuously recording pressure recorders, which will record continuously for a period of 7 days (fig. 5). The final step in the installation is to provide some sort of shield for the gages and/or recorders, to protect them from falling rock and traffic which may be moving through the test area. One method (shown in fig. 5) is to insert the gages into an 8-inch-diameter hole drilled approximately 8 inches into the rib, and then place aluminum plates over the holes. The valve handles are also removed to help prevent anyone from opening a valve out of curiosity. At this point, the installation is complete, with the exception of cleaning up the area and removing our equipment from the mine.

Many of the details have been left out of the presentation; however, you should have a general idea of how the instruments are installed. Once the technique of assembling the instruments is learned so that fluid leaks are not encountered, and once the driller comes up with the right combination for drilling the holes, the installation should go very smoothly.

FIGURE 5. - Completed Installation of Hydraulic Pressure Cells, With Pressure Gages Recessed in Covered Holes and Mechanical Chart Recorders for Continuous Readings of Selected Cells.

A BOREHOLE DEVICE TO DETERMINE IN SITU ELASTIC CONSTANTS OF COAL MEASURE ROCKS

by

Verne E. Hooker[1]

ABSTRACT

A simple borehole device to determine the modulus of rigidity (G) of rock has been developed and successfully tested in the laboratory and in the field. The mathematical analysis required to interpret the results has also been developed. In the laboratory tests, good agreement was observed between the known values of G and those determined experimentally for both metal and rock cylinders. The device was tested at the Colorado School of Mines Experimental Mine at Idaho Springs, Colo., in the No. 4 coal mine of the Mid-Continent Coal and Coke Corp. near Carbondale, Colo., and in the Colony Development Co. oil shale mine.

INTRODUCTION

The Bureau of Mines, in the sixth rock mechanics symposium at Rolla, Mo., in 1964, described a cylindrical pressure cell used in a drill hole to determine the modulus of rigidity of rock.[2] This cell, consisting of a steel core to which was brazed a copper shell with a connecting tube, was inserted into a drill hole. Glycerin was pumped between the core and copper shell, causing the shell to expand and contact the surrounding rock. The fluid pressure-volume curve was recorded and used together with elastic theory to compute the modulus of rigidity of the surrounding rock.

The CSM (Colorado School of Mines) cell developed under contract with the Bureau of Mines is similar in principle; however, an adiprene membrane now replaces the brazed copper shell. This has simplified the use of the cell and has (1) reduced greatly the possibility of damaging the cell in either inserting or removing the cell from the hole, and (2) eliminated requirements of two special calibration cylinders. Only one cylinder of known elastic properties is required for this system.

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[1]Geophysicist, Denver Mining Research Center, Bureau of Mines, Denver, Colo.
[2]Panek, L. A., E. Hornsey, and R. L. Lappi. Determination of the Modulus of Rigidity of Rock by Expanding a Cylindrical Pressure Cell in a Drill Hole. Proc. 6th Symp. on Rock Mechanics, Rolla, Mo., 1964, pp. 427-449.

THE CSM CYLINDRICAL BOREHOLE PRESSURE CELL

The cell and equipment presently being used is shown in figure 1 and consists of the following items:

(1) A high-pressure generator with vernier indicator, rated at a pressure capacity of 10,000 psi and a fluid capacity of 30 cm^3.

(2) A 10,000-psi pressure gage that can be read to 5 psi with a bleeder for purging, flushing, and bleeding the Bourdon tube.

(3) Twenty feet of high-pressure stainless steel tubing 1/8 inch OD and 0.04 inch ID rated at 30,000 psi.

(4) Three-way high-pressure valve rated at 30,000 psi.

(5) Sectional insertion pipe.

(6) Thermometer.

Briefly, in operation, the fluid is forced into the cell in known quantities by the high-pressure cylinder and corresponding values of pressure are indicated on the pressure gage. These are the values needed for pressure-volume relationships.

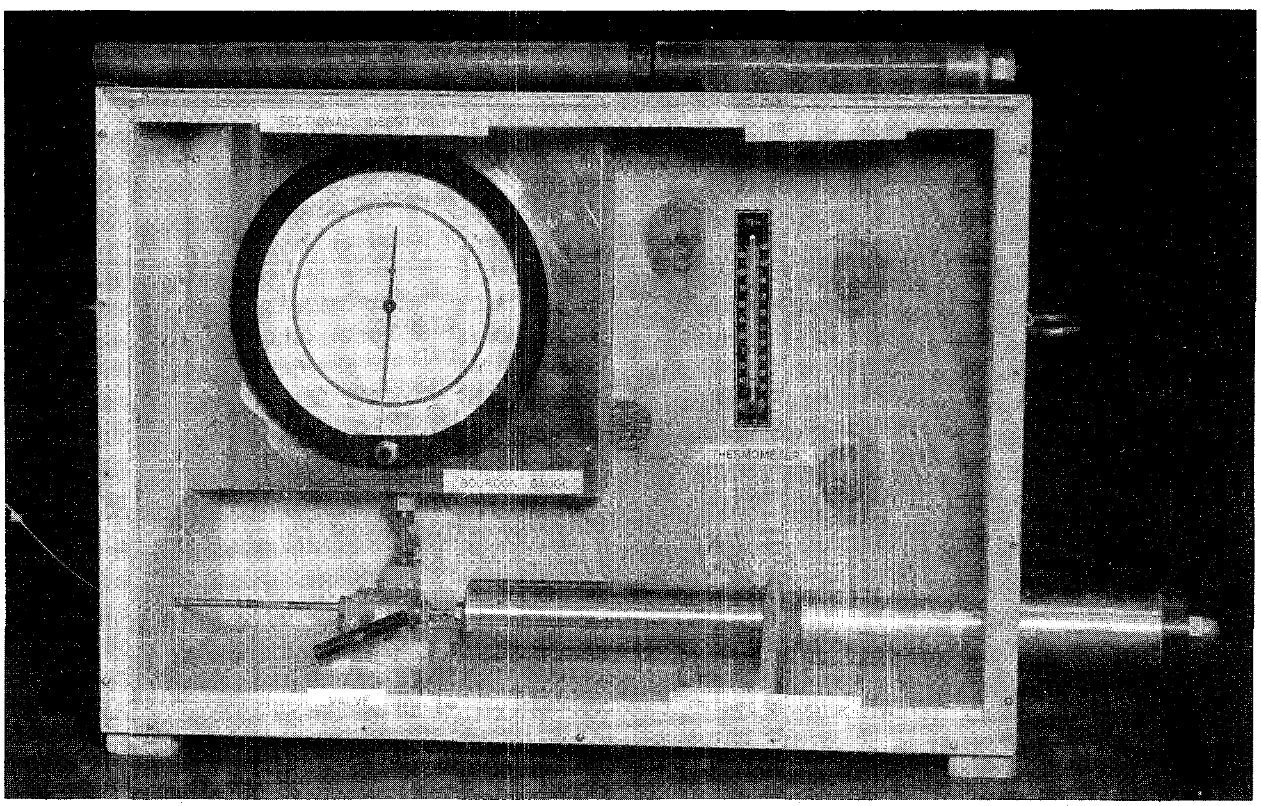

FIGURE 1. - Colorado School of Mines Borehole Device.

Design of Basic Cell

The basic CSM cell is shown in figure 2 and consists primarily of only two major units, the central shaft and the membrane.

The central shaft and one endpiece are made from a single piece of steel. High-strength steel is required for this shaft in order to obtain a system pressure of 10,000 psi without difficulty. Normal-strength steels were tried and tested for the shaft but failed at about 7,000 psi.

The adiprene membrane is constructed in a two-step process. First, a central core is made using Ross wax (melting point of 160° C). This is then placed into a second mold, and the adiprene-moca compound is poured. After the membrane has cured, the central wax core is easily machined out and removed, leaving the desired membrane.

The membrane is placed part way over the central steel shaft. The annular space is filled with glass spheres 4 mm in diameter in order to reduce the total volume of fluid in the system. The goal is to assemble the system so that there is a minimum of trapped air or other gasses and so that the system is free from leaks. As the fluid enters the membrane cavity, the applied pressure seals the flange against the central shaft and seals the ends against the steel end caps, thus preventing the fluid from escaping.

The fluid used to fill the system consists of a mixture of one part permanent antifreeze (ethylene-glycol base) and five parts water by volume. This mixture has proved to be a satisfactory fluid and permits operation down to 20° F. After filling and purging the system satisfactorily, the cell is placed in a calibration cylinder like those shown in figure 3 and pressurized to 3,000 psi to test for leaks. After a short time, the pressure should remain constant, thus indicating a nonleaking system ready for calibration and use.

Calibration and Pertinent Theory

It is most convenient to measure and calculate pressure-volume changes in terms of psi/turn of the pressure generator. One first calculates M_c for the calibration cylinder of known elastic properties using

$$M_c = \frac{\gamma G}{\pi L r_i^2 \left[\frac{1+\beta-2\nu\beta}{1-\beta} \right]} ,$$

where $\gamma = 0.0220$ in^3/turn, volume per turn of pressure generator,

$L = 6.5$ inches, effective length of membrane,

and $\beta = \left(\frac{r_i}{r_o} \right)^2$, where r_i, r_o are inner and outer radii, respectively, of cylinder.

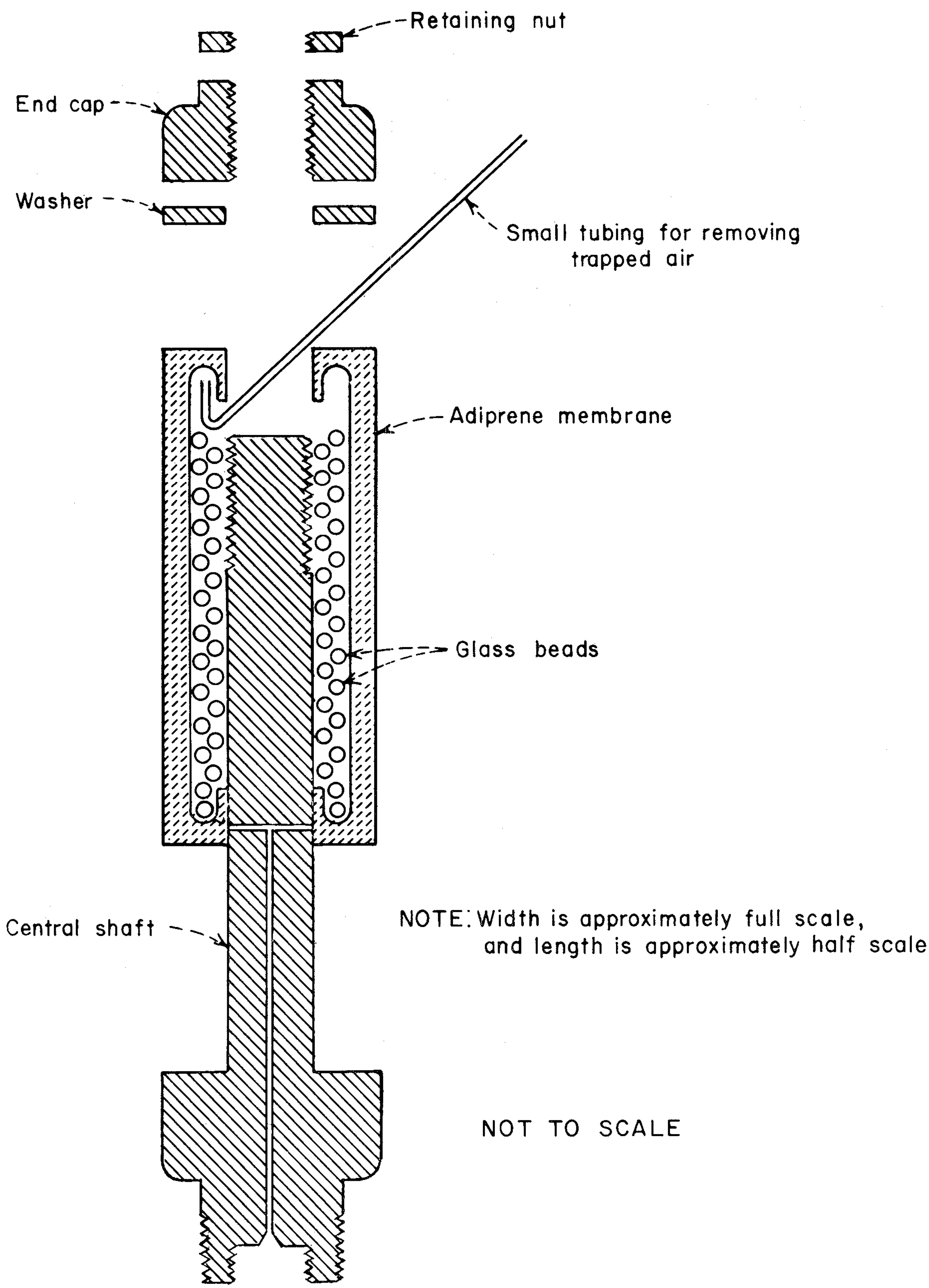

FIGURE 2. - Cross Section of Basic Cell.

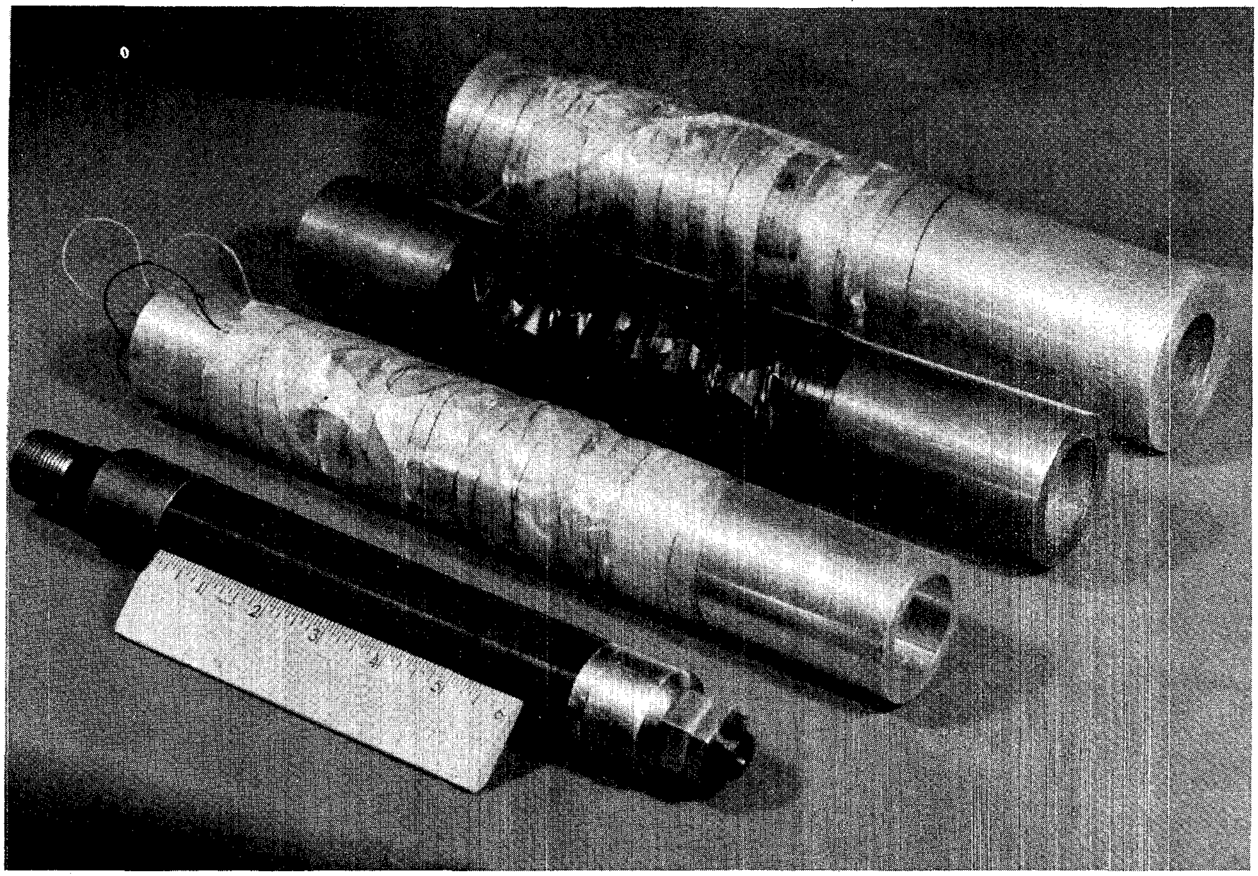

FIGURE 3. - Typical Calibration Cylinders.

Next, the CSM cell is inserted into the calibration cylinder, and the slope of the experimental pressure-volume curve (M_m) is determined. A plot of the data obtained in a steel cylinder is shown in figure 4. The system stiffness (M_s) is then calculated from

$$M_s = \frac{M_c M_m}{M_c - M_m} .$$

The CSM cell is then inserted into the cylinder or borehole of unknown properties, and the pressure-volume curve (M_t) is measured over the same range of pressure as the calibration. A set of such data obtained from oil shale is shown in figure 5. The pressure-volume relationship (M_r) for the test data can then be calculated using

$$M_r = \frac{M_t M_s}{M_t - M_s} .$$

The modulus of rigidity for this test cylinder is then calculated from

$$G_r = \frac{M_r \pi L r_{i\,r}^2}{\gamma} \left[\frac{1+\beta_r - 2\nu_r \beta_r}{1-\beta_r} \right],$$

or for a borehole,

$$G_r = \frac{M_r \pi L r_i^2}{\gamma}.$$

In these equations, the value of γ can be determined quite accurately, whereas, the value of L depends on the applied pressure and hole diameter. As long as the pressures and hole diameters used in calibration and in the field tests are about the same, the introduction of any error is small. The CSM system will provide reliable results for rock material having a modulus of rigidity of up to 3×10^6 psi. Most coal measure rocks would be in this range of properties.

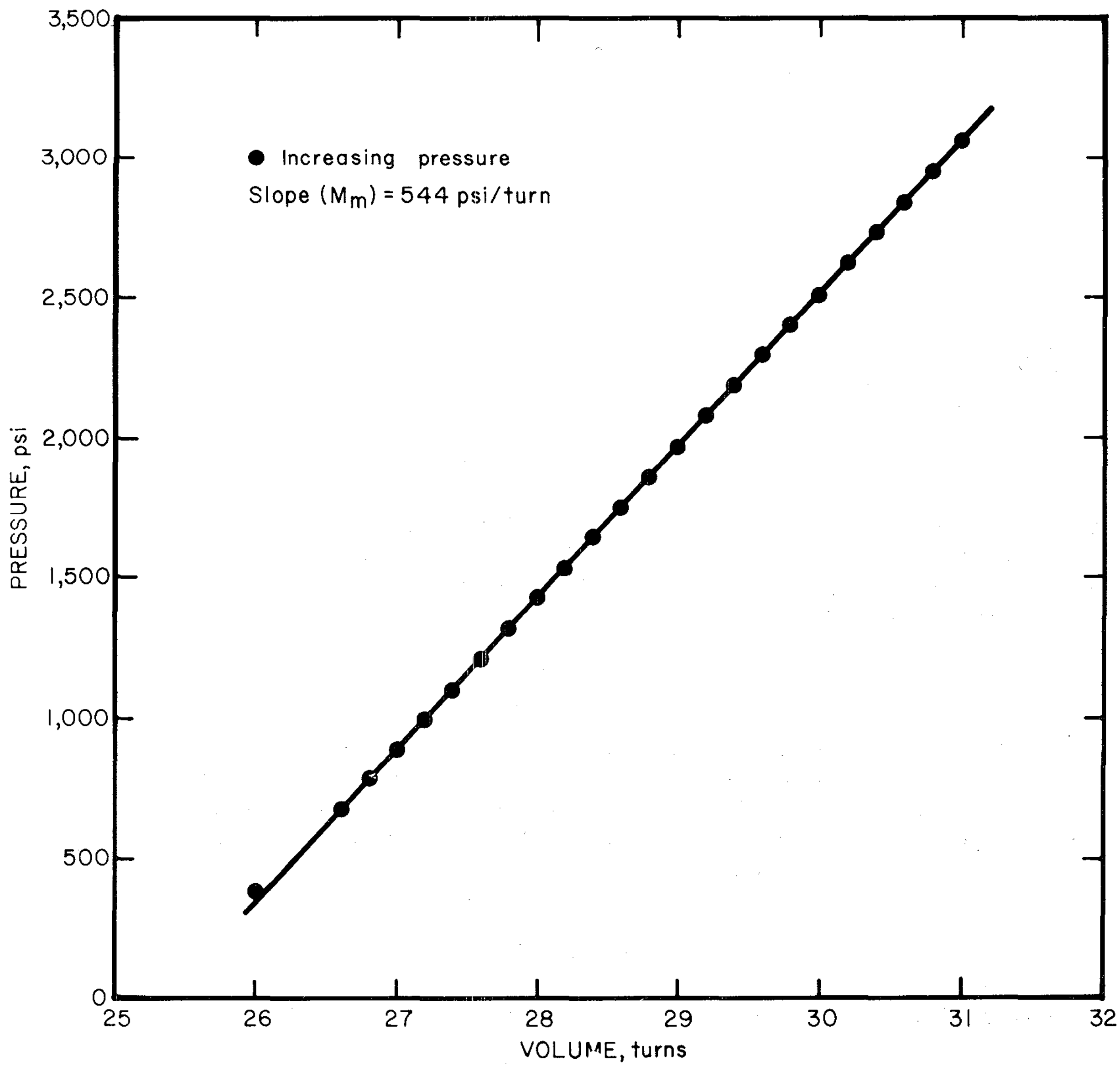

FIGURE 4. - Calibration Curve for Steel Cylinder.

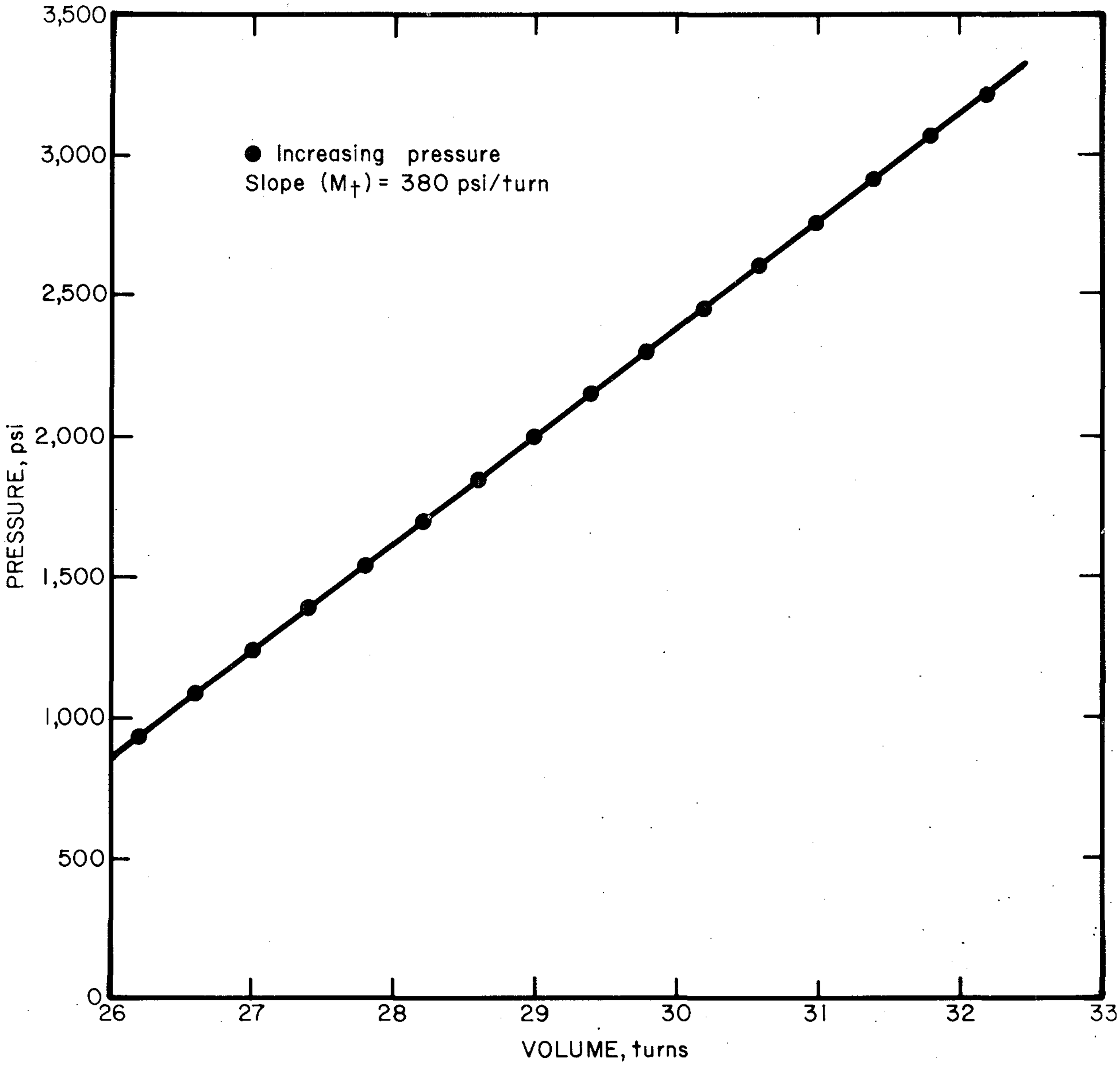

FIGURE 5. - Pressure-Volume Relationship for CSM Cell in Oil Shale.

A large amount of laboratory test data obtained from several rock types and metal cylinders were accumulated during the development program. These data essentially show good agreement between known values of G and those determined by the CSM device. The pressure-volume curves and calculated elastic constants for all laboratory tests and field tests in oil shale, coal, and Idaho Springs gneiss are given in the contractor's progress and final reports.

REPEATABILITY OF TESTS

An example of in situ test data obtained at various hole depths and at different times is shown in table 1. The reliability of these data are demonstrated by the fact that the slopes are repeatable within 2 percent. The entire time period for completing tests at six different hole depths was only a single morning.

TABLE 1. - Computed slopes of pressure-volume relationships at various depths in oil shale

Depth, feet	Initial readings		Final readings	
	Time	Slope, psi/turn	Time	Slope, psi/turn
2	9:25 a.m.	370.4 376.5	11:55 a.m.	374.2
4	9:45 a.m.	392.1 394.6	11:45 a.m.	393.6
6	10:05 a.m.	380.9 385.7	11:35 a.m.	380.4
8	10:20 a.m.	412.0 416.3	11:25 a.m.	411.7
10	10:35 a.m.	415.1 420.4	11:15 a.m.	414.0
11.4	10:50 a.m.	412.8 421.4 419.9	-	-

After transporting the equipment to the underground test location, sufficient time (30 to 45 minutes) was allowed for the system components to reach temperature equilibrium prior to calibration and in situ testing. Therefore, the measurements were carried out under constant temperature conditions. This is ideal and necessary if good results are to be obtained.

APPLICATIONS OF THE BOREHOLE PRESSURE CELL

The borehole device provides a simple means of obtaining in situ elastic constants of rock material. These constants are a basic requirement in mine design and provide reliable input for finite element mine model studies. The gage may be useful in determining a deformation modulus for large rock masses. The device is a basic tool that can be used to obtain the elastic constants of rock. These constants are necessary to convert measured displacements to stresses in other studies. For example, consider roof and floor control problems in coal mines or any rock surface where the knowledge of the existing stresses or changes in stress are desired. The Bureau has recently developed and tested a technique for obtaining information that is simple and economical. Only a small amount of equipment is needed, as shown in figure 6; the items shown are a templet for drilling a 60° rosette configuration, a 3/8-inch rock drill bit for drilling holes for pin installation, six stainless steel pins for use with a Whitemore-type gage, and a Whitemore gage.

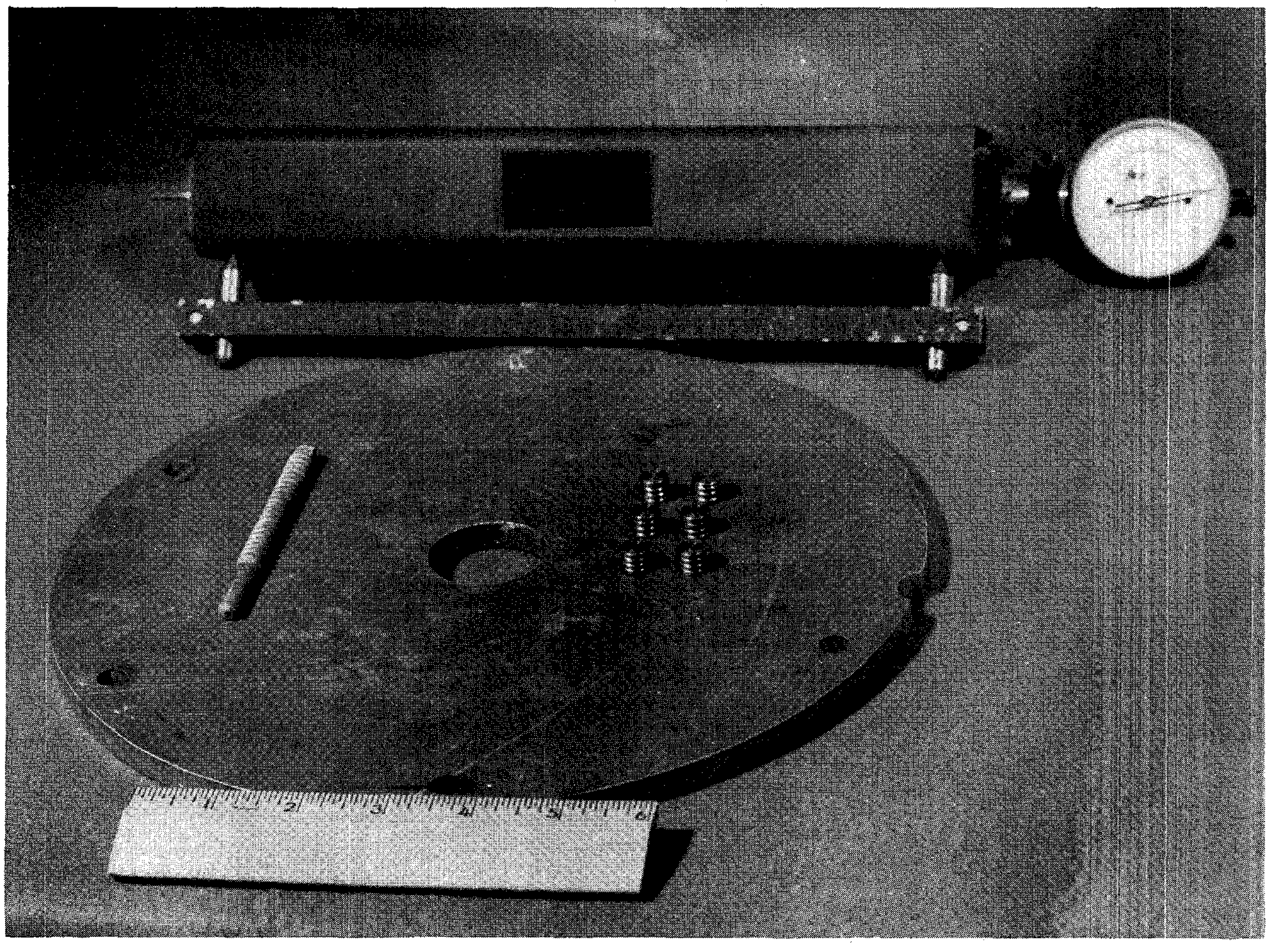

FIGURE 6. - Equipment for Surface Rosette Displacement Measurements.

For absolute stress determination, the technique consists of installing the stainless steel pins across three diameters of a proposed drill hole in the roof rock and then drilling this hole near the center of the six pins, as shown in figure 7. The configuration shown utilizes a Whitemore gage of 10-inch length. The 1-1/2-inch hole is drilled, and the in situ elastic constants of the rock obtained with the CSM borehole device. The 1-1/2-inch hole is then reamed or a new hole drilled to 6 inches in diameter to provide sensitivity to the system. Measurements of displacement across the three diameters provide enough information to solve for the stresses in that plane of measurement. Without interpolation between units on the gage, the sensitivity is 110 psi for absolute stress measurements. For measurement of stress changes, this can be reduced to 22 psi with interpolation.

Depending on sensitivity requirements, hole size and diameter of measurement can also be varied. Stress measurement data can thus be economically obtained in many surface locations in a mine with only a small amount of equipment. This system has been tested and used very satisfactorily at the Denver Mining Research Center for determining the absolute stress in rock.

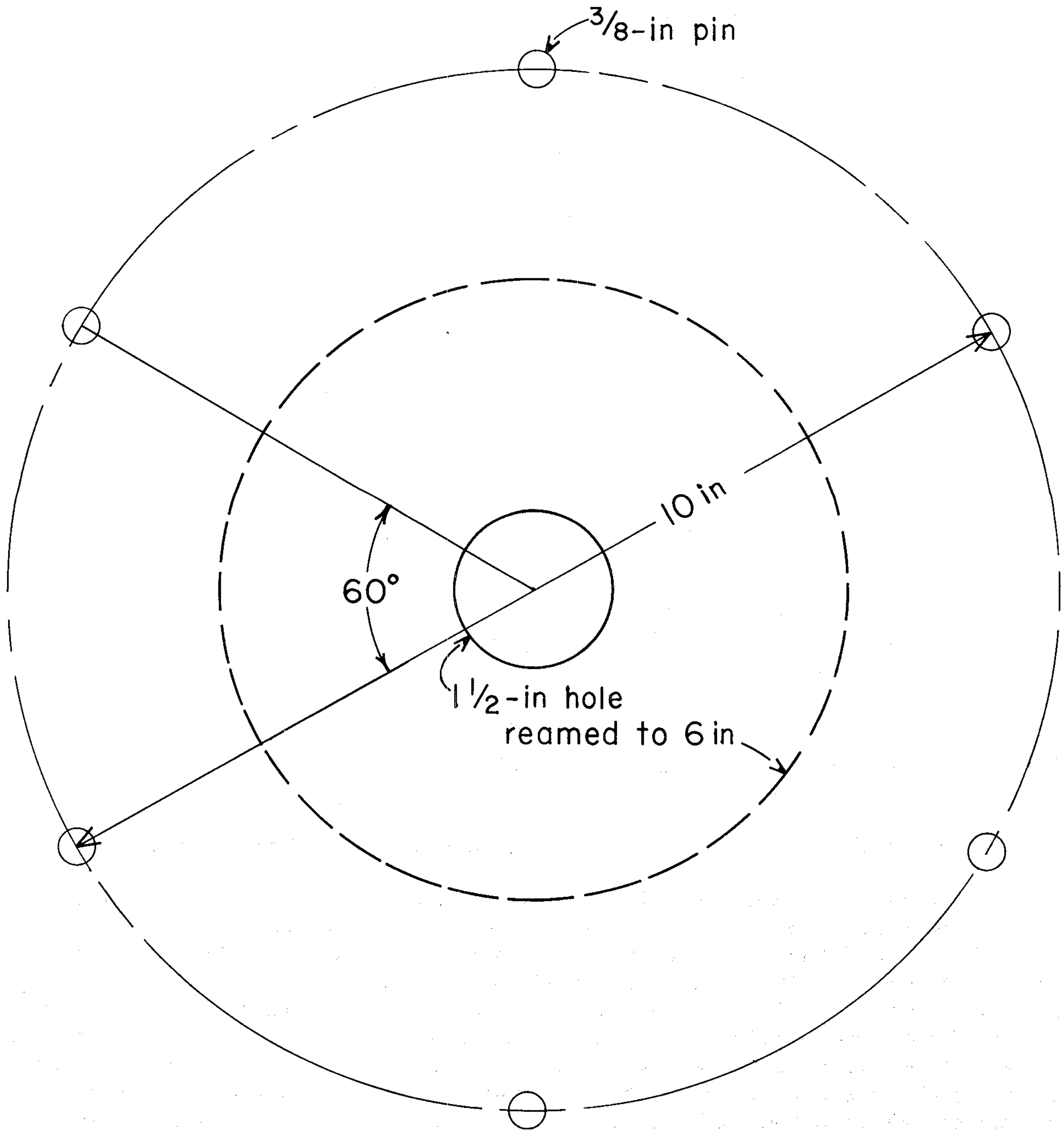

FIGURE 7. - Surface Rosette Configuration.

SUMMARY

A borehole device to determine the in situ elastic constants of rock material has been developed and successfully demonstrated to provide reliable results. A large amount of data can be obtained in a relatively short period of time. The device has been tested in coal, oil shale, and granite rock types.

The use of the tough, resilient, elastic, low-modulus membrane for the expansion shell may simplify the use of the system in irregular or oversize drill holes.

An economical technique for measuring displacements on a rock surface was described. With a knowledge of the in situ elastic constants obtained by a device such as the CSM cell, these displacements can be converted to stresses. Such a system can conceivably be used for determining absolute stresses or changes of stress.

More detailed information is available on the Colorado School of Mines borehole device in the final contract reports, which are on open file in all Bureau of Mines libraries.

FIELD OPERATION OF THE BUREAU OF MINES AUTOMATIC RECORDING BOREHOLE INCLINOMETER PROBE

by

Willard J. Tesch[1]

ABSTRACT

An inclinometer system is used to survey the attitude of cased drill holes, in order to determine ground movement caused by mining. Guide wheels on the inclinometer probe travel in grooves in the casing. The survey equipment consists of the inclinometer probe, winches and speed controller, an over-the-hole sheave device, an automatic data acquisition system, and a calculator. This equipment is operated from within a van truck.

The inclinometer probe is used to make 12 survey runs in the hole, three in each groove, to obtain a measure of precision. Three readings (two inclinations and the depth) are obtained at 1-foot intervals while the probe is being pulled up the hole at 30 feet per minute by the cable-drive winch. These readings are punched on paper tape and entered into the calculator, which calculates the average reading from each accelerometer axis, to detect spurious readings. About 4 hours are required to make these 12 runs in a 400-foot hole. The paper tapes are returned to Denver and converted to computer cards, which are processed by computer to obtain the average hole offsets in the north-south and east-west planes. The change in shape of the hole between sets of survey runs is then plotted.

INTRODUCTION

The inclinometer system is a field tool used to obtain readings of the inclination of a cased drill hole versus depth. The readings are obtained automatically as the probe is moving up the cased drill hole at a speed of 30 feet per minute. These readings, punched on paper tape, are then returned to Denver, where they are used as input to a computer that calculates the hole offsets versus depth, related to the hole collar. Changes in these offsets are a measure of the ground movement caused by mining, as mentioned earlier today by Dr. Panek.

The system consists of the inclinometer probe, a cable-drive winch and controller, an over-the-hole sheave device, an automatic data acquisition system, and a calculator. This equipment and a teletype is mounted in a four-wheel-drive van truck which contains a motor-generator set to supply power.

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The probe has four guide wheels which run in grooves in the casing. It is 1 foot long between the guide wheel axles and 1-3/8 inches in diameter. The data acquisition system contains a digital voltmeter, bidirectional counter, coupler-formatter, and an eight-channel paper tape punch. Figure 1 is a photograph of the data acquisition system and associated power conditioner.

For the purposes of this discussion, it is assumed that the hole to be surveyed has already been drilled and that the grooved casing has been cemented into the hole.

MODE OF OPERATION

To make the field survey, the truck is driven to the spot from which the surveys are to be run. It is backed near the hole with the hole roughly in

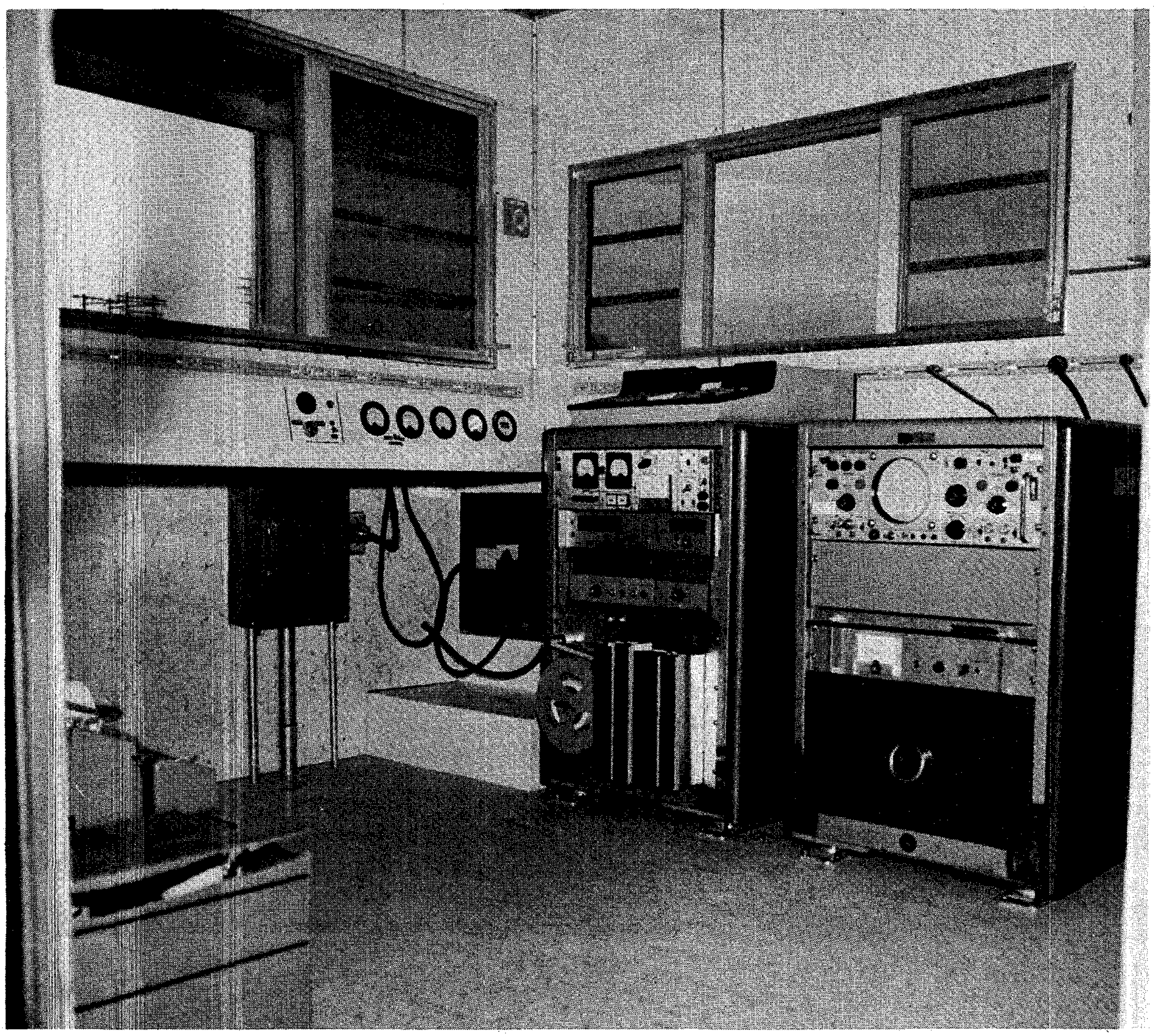

FIGURE 1. - Data Acquisition System for Borehole Inclinometer Probe.

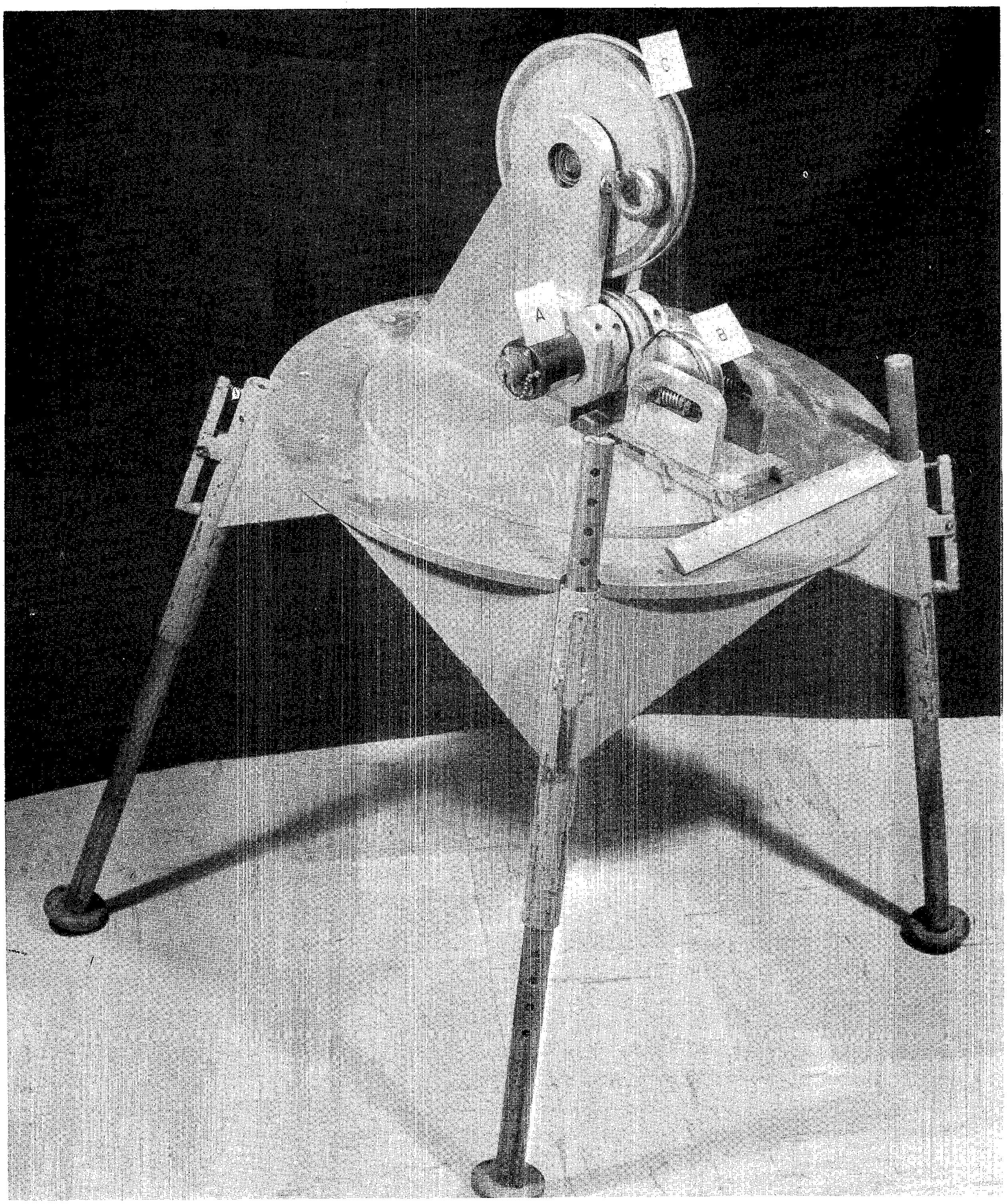

FIGURE 2. - Over-the-Hole Sheave Device. *A,* Rotary shaft position encoder (with backup depth counter); *B,* Spring-loaded wheel (forces cable against counting wheel attached to *A*); and *C,* Guide pulley.

line with the center of the level wind winch. The hole collar must be visible to the operator from his position at the winch controller. As soon as the truck is in position, the motor generator is started and the electronic equipment turned on to allow the equipment to warm up and stabilize. The quadripod over-the-hole device, shown in figure 2, is placed over the hole.

The dummy probe is removed from the truck, lowered through the over-the-hole device, and placed in the hole. The cable is run through the guide and over the sheave. The top plate and legs are adjusted so that the plate is perpendicular to the axis of the drill hole and the cable is centered in the hole. The chain, which prevents the device from being pulled over by the pull of the winch, is attached to the hook beside the sheave and to an anchor placed in the ground for this purpose. The turnbuckle on the chain is then adjusted so the chain is taut. Figure 3 shows the truck set up at a test hole.

Four trial runs are then made in the hole, using the dummy probe to check whether the inclinometer probe will pass through the hole with the fixed

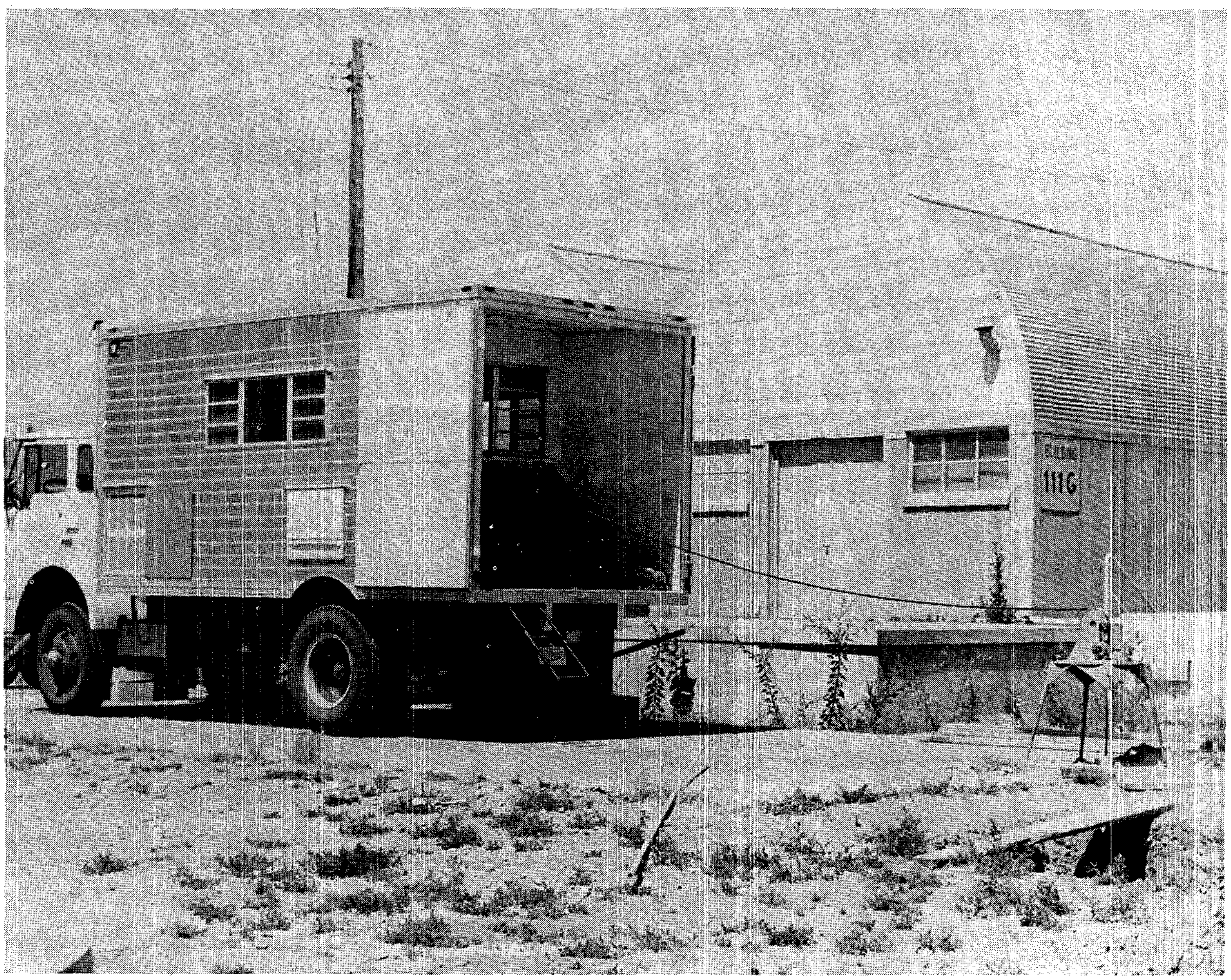

FIGURE 3. - Borehole Inclinometer Probe Ready To Begin Logging.

wheels in each of the four grooves. The dummy probe is run at 50 feet per minute, so the time to make these runs is dependent on the length of the hole. A 400-foot hole would require about 70 minutes for this test; 64 minutes actual running time and a few minutes to remove the probe from the hole, rotate it, and reinsert it into a different set of grooves. The dummy probe is removed from the hole, the cable is wound up, and the probe is then put back in the truck.

The inclinometer probe is then removed from the truck, the cable is run out and through the over-the-hole device, and the probe is put in the hole with its fixed wheels in the groove closest to north. The transmission through which power is supplied to the winches is then shifted so the inclinometer cable-drive winch is connected to the motor. The probe is lowered to the bottom of the hole at 50 feet per minute, then raised 0.2 foot above the bottom. The depth counter in the data acquisition system indicates when the probe is on the bottom, since it stops indicating a change in depth when the probe stops. The depth counter is a bidirectional counter driven by a rotary shaft position encoder in the probe. The top fixed wheel drives the encoder through a chain and gear train so that the encoder disk makes one revolution when the wheel traverses 1 foot. The resolution of the depth counter is 0.01 foot.

The depth counter is reset to zero, the paper tape punch is turned on, the reset button on the control panel is pushed, the calculator set to receive the first reading, and the start button is pushed. This causes the first set of readings to be made, punched on tape, and entered into the calculator. The readings consist of the depth counter reading and the voltage output from each axis of the biaxial force balance servoaccelerometer in the probe. The voltage readings are made to a resolution of 1 millivolt, equivalent to 27 seconds of arc at hole inclinations near vertical.

The winch motor is then started and adjusted to pull the probe up the hole at 30 feet per minute. Readings are made automatically each foot of probe travel, punched on tape, and entered into the calculator. The probe is stopped with the top fixed wheel 1 foot below the top of the casing and the start button is pushed to obtain the last reading for that run. The cable is marked with a white band 1 foot above the top fixed wheel.

The probe is removed from the hole, rotated 180°, and put back in the casing with its fixed wheels in the southern groove. It is again lowered to the bottom of the hole at 50 feet per minute and another run is made, the data acquisition system again obtaining readings each foot of probe travel, punching them on tape, and furnishing them to the calculator.

Runs are also made with the fixed wheels in the eastern and then the western groove. At the present time, 12 runs are made in each hole for each survey, three runs with the fixed wheels in each of the four grooves, in the sequence north, south, east, west, north, etc.

The time needed to make a complete set of survey runs in a hole again depends on how long the hole is. A 400-foot-long hole, for example, is surveyed 12 times (traversed 24 times), once each way for each survey. The total travel of 9,600 feet consumes a nominal 4 hours. Including the time to set up and remove the over-the-hole device, run the dummy probe, run the inclinometer probe, and do the necessary probe rotation, total elapsed time would be about 6 hours for a 400-foot hole.

During the time required to make the series of runs in the hole, it is assumed that no significant hole movement is taking place. If a trend in the average reading from either accelerometer axis is noticed, for runs made in the same groove, this assumption may not be valid. These averages are obtained from the calculator at the end of each run. Differences in this average reading from either axis, obtained during runs made in the same groove, which are more than the known repeatability of the system, could indicate that some component in the system is not working correctly. Occasionally, punched data tapes are listed on the teletype to verify that the punch is punching what appears to be legitimate data. The operator of the equipment can be confident that he is obtaining valid data by knowing that the average readings are within known repeatability standards and that the paper punch is operating correctly.

REDUCTION OF DATA

The paper tapes are converted to cards at the Bureau's Automatic Data Processing Division in Denver by a tape-to-card conversion program. These cards are then used as input to a computer program that calculates the hole offsets versus depth. These offsets are calculated in a north-south and in an east-west plane, using the inclinometer probe data and down-the-hole groove azimuth data. These offsets are the amount, in feet, that the center of the hole differs from a straight vertical line from the center of the hole collar. The program calculates these offsets for each foot of hole depth for each run. Average offsets are then computed for each foot of hole depth, and the standard deviation of the individual offsets which went to make up this average. Because any single run furnishes sufficient information to calculate the hole offsets, the standard deviations of the offsets computed from individual runs are a statistical measure of the repeatability of the entire system.

Surface surveys of the position of the hole collar must be made at the time of inclinometer probe surveys, in order to determine absolute hole displacement. The difference between the average offsets and top-of-the-hole position determined during the initial survey and subsequent surveys are plotted to determine the hole movement which occurred between surveys. These plots are made in north-south and east-west planes. Figure 4 is an example of one of these plots. It shows that between June 1971 and March 1972 this drill hole moved 0.8 foot north and 0.33 foot west at a depth of 217 feet. The standard deviations of the 12 offsets computed during the March 1972 survey at a depth of 217 feet were 0.038 foot for the north offset and 0.032 foot for the west offset. The standard deviation of the average of a number of sets of 12 runs would therefore be 0.011 foot for the north offset and 0.009 foot for the west offset.

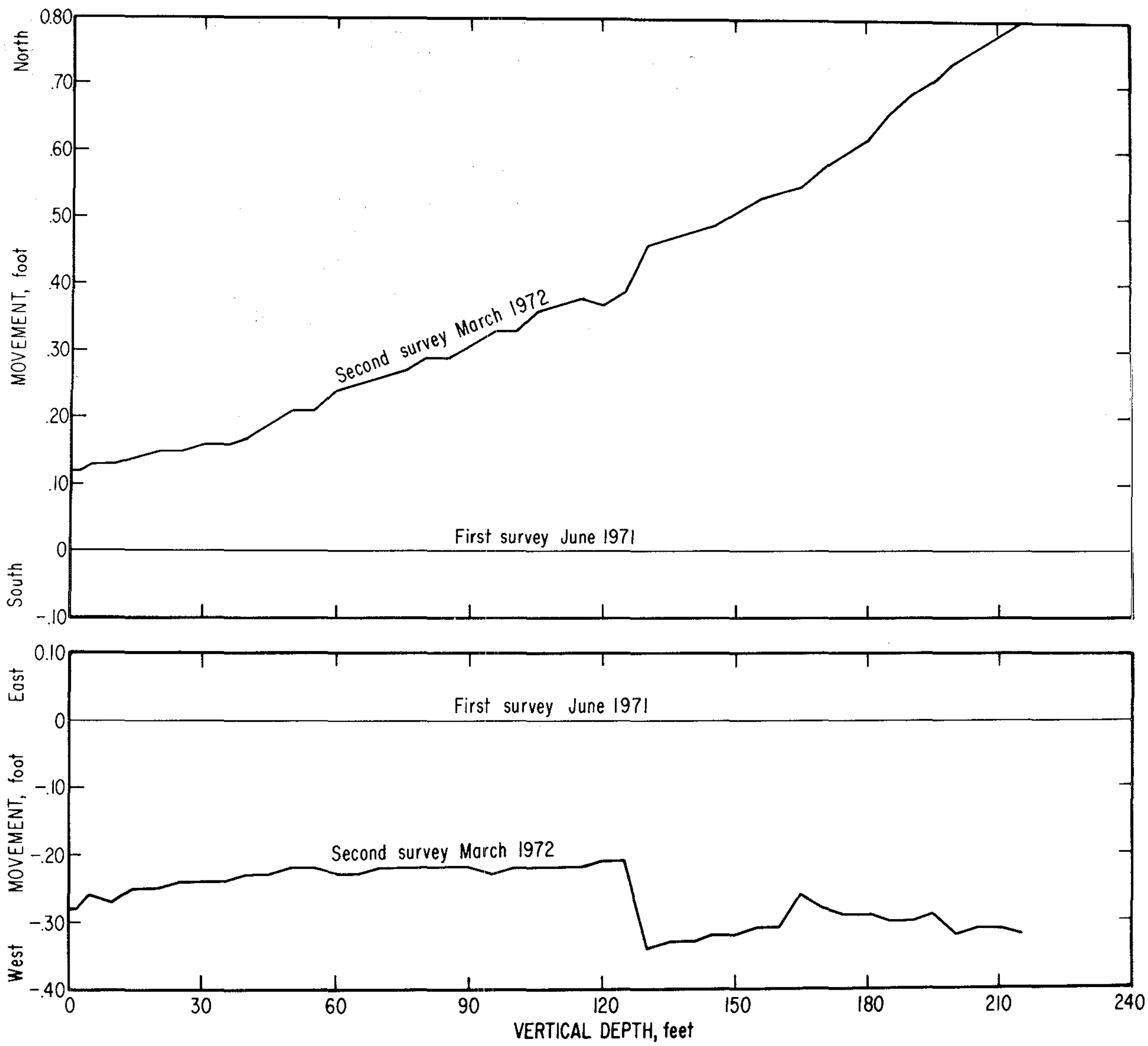

FIGURE 4. - Plots Showing Changes of Profile Along a Hole.

TYPICAL APPLICATIONS

During a recent 3-week field trip, one man drove the truck from Denver to Louisiana and made 12 runs in each of six drill holes over a salt mine. The total length of these six holes surveyed was 927 feet. He then drove to Tucson, Ariz., where he made 12 runs in each of two 407-foot-long drill holes next to an open pit mine. Five days of this time was spent actually making the surveys. The measurements over the salt mine were made in an attempt to locate stable ground through which to sink an air shaft as well as to measure ground movement caused by mining. The measurements at the open pit mine were to measure the pit wall stability in one area.

This equipment will be demonstrated after the seminar.

SUMMARY

by

Harry R. Nicholls [1]

Gentlemen, please allow me a few words in summary this afternoon. First, we'd like to thank you all for coming. It's been very gratifying and I hope the communication channels developed today will remain open and grow.

This is the first in a series of seminars to be conducted by Mining Research. The next is scheduled for September 26 at Pittsburgh and will cover the problem of intersecting oil and gas wells. The third in the series will be in Denver, December 5 and 6, and will be directed towards mine design with specific application to coal mining. This will be a natural sequel to today's meeting, which was oriented toward instrumentation to obtain the numbers for mine design and included some philosophy and basics of design.

The question arises--Why must we have the seminar? What is its purpose? We've been doing research for many years. Most recently we have been charged under Public Law 91-173 of 1969 to conduct studies, research, experiments, and demonstrations. We have been doing just that. One of the problems is how do we get the results to industry and other potential users? We use the words technology transfer. How do we transfer the technology? In the past we have published--in Bureau form, in technical journals, and so forth. We have attended technical seminars arranged by others and presented our results. We have worked through cooperative agreements with mining companies on both specific and general problems. These are all good mechanisms for transfer and we will continue to use them. However, these are not enough. We have therefore adopted the use of seminars. The dictionary gives us our definition: Seminar--a meeting for giving and discussing information. We sincerely hope that these seminars become true dialogues in the future. Our meeting today has not been as open and free as I would have liked, but it is a start. Our research people are familiar with the law insofar as it provides the purpose or direction for their research but they don't want to discuss the law. They also don't want to stand in front of you and lecture. Perhaps future seminars can be more open, specifically between our research people and the coal industry. It is the establishment of a dialogue that we both need. As we proceed towards obtaining results in various areas of research, we will hold similar meetings. I hope we can make these <u>our</u> meetings, not Bureau meetings.

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In our audience we have representatives of the coal mining industry, metal and nonmetal mining industry, universities, consultants, and instrument manufacturers. There are representatives from other Government agencies that work in the field of rock mechanics, such as the Bureau of Reclamation, Geological Survey, Corps of Engineers, Bureau of Public Roads, and so on. In addition, we have Health and Safety Technical Support people who are intermediate between our Mining Research and industry. These Technical Support people are working daily in your mines. Our relationship with them is another part of technology transfer.

Let me now talk a little about what happened today. We talked about the philosophy of design and gave an overview of some of the things we've been doing in structural design. We've talked some about coal, some about hard rock. But remember our emphasis in ground control in coal is only 2 years old. We're transferring our own technology to coal problems. We had been working in coal, but now, with greater emphasis. For those of you in coal who do not have design problems, you have an overview of what we're doing in this area. For those who do have problems, you also have an overview, but you now have seen and met some of our people who are involved. We enlist your support. I don't mean verbal backing or carte blanche approval. I mean that if you have problems, contact these people you have seen and met today. Sit down with us, discuss problems in ground control or instrumentation. This is the type of support we need. Help us open up the needed dialogue. As other areas of research are discussed in seminars such as this, we ask for this kind of support.

We also talked about tools. The devices discussed by Mr. Smith have been used in coal for several years by Dr. Panek. The cell with the soft jacket may advance the state of the art when adequately validated. The recent developments in the borehole deformation gage will, we hope, make this technique more useful in coal and coal rocks. Data developed using the borehole inclinometer may help solve some of our problems of stability or instability in the strata from the underground opening to the surface.

What do we have on the back burner? Back burner used to mean 5 to 10 years, now it means a year or two. We have a contract to develop a low-cost cylindrical stress gage. If a gage were of low cost, one could use 10 or 20 gages in a section and observe pillar and rib loading or unloading as mining advances. Designs could thus be modified. But remember, we live with the same permissibility requirements established under the Coal Mine Health and Safety Act that you do in developing instrumentation. We have a contract to develop a portable borehole shear strength tester for use in coal which will be useful for mine design. We are working in-house on the post-failure behavior of coal and coal measure rocks; that is, the behavior once failure has been initiated. This would become part of our design technology through failure criterion or other considerations. We are working on various types of behavior and finite element models discussed briefly by Drs. Panek and Brady. These results and more we hope to discuss with you here in Denver in December. We'll look for you then, and thanks again for coming today.

Tour

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