OCTOBER 18, 1974

ROCK ENGINEERING

OHIO RIVER VALLEY SOILS SEMINAR, 1974

CLARKSVILLE, INDIANA

PROCEEDINGS

OHIO RIVER VALLEY
SOILS SEMINAR, 1974

October 18, 1974
Clarksville, Indiana
(across the Ohio from Louisville)

Presented by

KENTUCKY SOIL MECHANICS AND FOUNDATIONS GROUP
Kentucky Section, American Society of Civil Engineers

SOIL MECHANICS
Kentucky
AND FOUNDATIONS GROUP

Cosponsored by

Cincinnati - Dayton Soils Group
American Society of Civil Engineers

Department of Civil Engineering
University of Louisville

Department of Civil Engineering
and
Office of Engineering Continuing Education
University of Kentucky

FORWARD

The 1974 Ohio River Valley Soils Seminar was held on October 18, 1974, at the Marriott Inn in Clarksville, Indiana. This was the fifth annual seminar presented alternatively by the Kentucky Soil Mechanics and Foundations Group and the Cincinnati-Dayton Soils Group. The seminars are presented as a forum for discussion and interchange of ideas and techniques among those practicing the geotechnical disciplines, particularly soil mechanics and engineering, rock mechanics, and geology. The purpose of this fifth seminar was to present a discussion of principles of rock engineering as it relates to various civil engineering activities.

This seminar was principally organized by the Kentucky Soil Mechanics and Foundations Group of the Kentucky Section, American Society of Civil Engineers. Cosponsors of the meeting included the Cincinnati-Dayton Soils Group of the American Society of Civil Engineers, the Department of Civil Engineering at the University of Louisville, and the Department of Civil Engineering and the Office of Engineering Continuing Education at the University of Kentucky. Special thanks for support of the social functions of the meeting are extended to the Mobile Drilling Company, Inc., the Central Mine Equipment Company, the Commercial Shearing Company and Celtite, Inc. Support in the form of exhibits and displays was also provided by Karol-Warner, Inc. and Associated Pile and Fittings Corporation. The planning committee for the seminar consisted of

R. C. Deen, Kentucky Bureau of Highways, Lexington, Kentucky;
M. M. Greenbaum, Greenbaum and Associates, Louisville, Kentucky;
C. R. Lennertz, H. C. Nutting Co., Cincinnati, Ohio;
A. D. May, Fuller, Mossbarger and Scott, Lexington, Kentucky;
H. Mathis, Kentucky Bureau of Highways, Frankfort, Kentucky; and
D. J. Hagerty, Department of Civil Engineering, University of Louisville, Chairman.

Presentations and discussions at the seminar included the following:

Exploration and Testing in Rock
J. Hagerty, University of Louisville, Louisville, and
J. Coulson, Tennesee Valley Authority, Knoxville

A Rock Classification Schema
R. C. Deen, Kentucky Bureau of Highways, Lexington;
M. Palmer, University of Louisville, Louisville; and
C. Tockstein, Tennessee Valley Authority, Knoxville

Foundations On and In Rock
E. D'Appolonia, D'Appolonia Engineers, Pittsburgh

Rock Foundations – Panel Discussion
C. R. Lennertz, H. C. Nutting Co., Cincinnati;
H. Thomas, USCEC, Louisville District; and

M. M. Greenbaum, Greenbaum and Associates, Louisville
Moderator: V. P. Drnevich, University of Kentucky, Lexington

Design and Construction of Rock Cuts for Kentucky Highways
G. Brandenburg, Kentucky Bureau of Highways, Frankfort

Design of Open Excavations in Rock
D. Ross-Brown, Dames & Moore, Los Angeles

Rock Excavation -- Panel Discussion
J. Mahar, University of Illinois, Urbana;
V. Cox, Vibration Measurement Engineers, Louisville; and
J. A. Waddell, Martin Marietta, Louisville
Moderator: M. D. Hensey, Proctor & Gamble, Cincinnati

Rock Engineering
Don U. Deere, Worldwide Consultant, Gainesville, Florida

Because of the nature of some of the presentations, written papers were not available so that they could be printed and distributed at the seminar. The presentations will be recorded and appropriate transcriptions will be made and printed. These will be distributed to all attendees at a later date.

Robert C. Deen
Planning Committee
Ohio River Valley Soils Seminar, 1974

EXPLORATION AND TESTING IN ROCK

by

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INTRODUCTION

This paper is concerned with the topic of exploration and testing of rock materials in the field. Testing and exploration in rock are of significant importance to all engineered construction activities undertaken in and on rock materials for a number of reasons. Most importantly, preliminary exploration and testing will disclose the nature and characteristics of the earth materials present at the site of a proposed facility and allow project planners to decide whether or not a proposed undertaking is technically and economically feasible. The nature and integrity of a rock mass may permit the construction of certain types of facilities at a given site while precluding the installation of other types of structures or excavations. The presence of discontinuities or weak areas within a rock mass may virtually eliminate a site from consideration for major construction. Additionally, the presence of water in great quantities (sometimes at high temperatures and pressures) may adversely affect proposed rock construction. The existence of limiting conditions or factors at a particular site must be disclosed during planning stages through the use of comprehensive investigation and testing.

Field exploration and testing is also important from the viewpoint of project safety. The existence of weak zones within a rock mass may pose a considerable threat to construction workers during the installation of a particular facility, and may also pose a long-term threat to the safety of individuals occupying or using a completed facility. In this connection, the unexpected occurrence of water within earth materials often presents a serious hazard to construction workers. Adequate and thorough exploration and testing at a site will significantly reduce the hazards for construction personnel and may also significantly reduce any future potential hazards to persons occupying or using that site.

Field testing and exploration are also important financially. The expenditure of funds associated with comprehensive exploration in rock is significant, and owners of proposed facilities often oppose the expenditures of large sums of money for preliminary exploration and testing. Such opposition to these necessary activities is certainly ill-advised. The procurement of comprehensive information on the nature and characteristics of the earth materials at a construction site will lead to the presentation of reasonable bids by contractors seeking work at that site. When a contractor is faced with inadequate or incomplete information about a site, he may have recourse to very conservative bidding to allow for unforeseen adverse occurrences associated with circumstances not shown in the preliminary information. When comprehensive information is available concerning the physical characteristics of the materials at a constriction site, interested contractors are enabled to present bids which reflect an informed opinion concerning possible difficulties which could materialize during construction. Because they are able to judge the possibilities and probabilities of such occurrences, informed contractors can present bids which will not contain a "buffer" designed to absorb unforeseen costs. Also, the dissemination of comprehensive information to contractors during bidding stages for a project will do much to prevent unnecessary litigation which often arises through the occurrence of unforeseen difficulties during construction. The familiar claim of "changed conditions" should be heard only when circumstances truly support such a claim, provided adequate information about underground conditions has been provided to the contractor prior to bidding. The monies associated with litigation involved in claims of changed conditions generally will greatly exceed the amounts of money necessary for comprehensive investigation of subsurface conditions prior to design and construction.

Field exploration and testing are also important from a strictly technical point of view. Obviously, design of constructed facilities in and on rock will depend upon information concerning the strength, compressibility, permeability and continuity of the materials at a given site. This dependence of design upon information secured during exploration has a corollary in project costs. In a situation where scanty or incomplete information concerning underground conditions has been obtained, design engineers justifiably tend to produce very conservative designs. The use of very high safety factors is often used as a defense against unforeseen weaknesses or discontinuities in rock masses. The dissemination of comprehensive information on rock properties and rock continuity to design engineers will produce more efficient and less conservative final designs.

Finally, field testing and exploration are important, along with monitoring of the behavior of completed facilities, because they yield information to promote the art of rock engineering. The state of the art of rock engineering can progress only through the gathering of information on the characteristics of rock masses and their behavior under imposed stress fields associated with engineered construction. In this regard, there is an unfortunate tendency for individuals concerned with design and construction in rock to obtain information sufficient for their own needs and to fail to contribute this information to other members of their profession. Only through widespread exchange of information on the behavior of rock and on the applicability of developed theories will the practical aspects of rock engineering advance. It must be remembered that rock engineering is exactly similar to all other forms of engineering in that it is a blend of developed theories from science and empirical information gathered through observation of previous construction. The empirical aspects of rock engineering must not be neglected. Consequently, more extensive efforts at data accumulation and exchange must be undertaken.

Because field exploration and testing in rock are so important and vital to the success of engineered projects in and on rock, attention should be devoted to the systematic planning and application of such exploration and testing. In gathering information on the materials at a construction site, several stages of activity are followed. In the succeeding paragraphs, these stages of investigation will be described and methods for systematically carrying out such investigations will be presented.

RECONNAISSANCE

The first phase of exploration and testing in rock ordinarily does not take place "in rock." This paradoxical statement can be explained in the following way. There is an unfortunate tendency among many engineers and contractors who work in soil and rock to undertake physical exploration and testing in these materials before they have performed a comprehensive review of existing information on the materials found at the site in question or at nearby sites. In this connection, a preliminary reconnaissance of existing information can greatly assist in the planning of site investigations and can form, for the investigator, an overall impression of site conditions. A good rule of thumb may be to "Look before you drill."

Surficial landforms and underground structures previously investigated may both give valuable indications of conditions to be expected at a new project. The primary and most obvious source of information on such conditions are the various geologic maps, memoranda and memoirs published by geological societies and similar groups throughout the world. Indices to available geologic and topographic maps have been published by the United States Geological Survey and many state geological surveys. Additionally, detailed reports on mineral resources, groundwater occurrence, geologic structures and similar circumstances of interest have been prepared and published for specific localities by local, state and federal agencies. The investigator should ascertain the availability of such materials through consultation with representatives of these agencies and then should spend a significant amount of time looking through these publications for information on the site under consideration. Geologic maps and similar materials should be examined to determine the characteristics and origin of surficial deposits. These surface deposits many times may indicate the characteristics of the underlying bedrock. Landforms should be examined for shape and distribution as clues to the lithology, structure and weathering of the underlying bedrock. The areal extent and occurrence of various rock units should be determined and, if possible, such determinations should be supplemented with very cursory surficial examination. In instances where sedimentary rock is present, a stratigraphic column should be prepared on the basis of available information to show the thicknesses and lithologies of the various strata. Unconformities and disconformities between strata should be indicated. In areas of igneous rock, the location of the rock units should be shown and this basic information should be supplemented with data on any planar or linear structures such as dykes, sills, etc. In metamorphic rocks, the primary fabric of the rock should be noted and attention should be devoted to determining the cleavage, linearity and foliation of the rock materials. All geologic structures such as joints, faults and folds should be determined from available information. The dip and strike of and amount of relative movement along faults should be noted. Joints should be examined and categorized into systems. These systems should be examined for their spacial orientation and their continuity. The regularity and smoothness of joint surfaces should be noted, if such information is available. From the available information, an assessment of groundwater conditions should be prepared. All springs should be located and the quantity and temperature of the water coming from such springs should be noted. The effects of groundwater on soluble rocks such as limestones, and the possibility of secondary permeability arising from such action, should be noted. From geologic maps and reports, areas for more intensive study by geophysical exploration and actual physical exploration and testing should be determined.

In addition to geologic maps and reports, other existing information can be utilized. Aerial photographs have been taken for vast areas in the United States and Canada. These photographs are available to the investigator for use in preliminary investigations of geologic structure and characteristics. The Map Survey publishes a periodically revised map of the United States showing the status of aerial photography. Indicated on this map are the agencies from which aerial photographs may be obtained. This map is available at a scale of 1:5,000,000, at no cost. The Agricultural Stabilization and Conservation Service of the U.S. Department of Agriculture annually publishes and distributes at no cost a booklet which shows all ASCS photo coverage in the United States, by county in each state. This booklet is entitled "Aerial Photography Status Maps." This booklet indicates the year and scale of the photographs and shows the numbered photo index sheets for each county. Photo index sheets are available at a cost of several dollars per sheet and positive contact prints of desired areas are available for less than $2 per print, with lower prices available for large orders. Photographs of areas in Canada can be obtained from the National Air Photo Library in Ottawa. Approximately 4 to 6 weeks delay will be involved in the transmittal of aerial photograph prints from governmental agencies to the investigator. When photographs are needed quickly, various commerical airphoto services may be consulted. Obviously, if photographs of a particular site are not in existence in the files of any agency or company, commerical airphoto firms may be employed to take such photographs.

The most immediate use of aerial photographs is in the preparation of topographic base maps with elevation contours. Variations in the appearance of vegetation, soils, rock strata and other physical features may be used in the interpretation of aerial photographs to yield additional information. This information may vary in accuracy and complexity from one site to another, but generally valuable indications of surface and underground conditions may be obtained from aerial photographs. In addition to photographs, various other methods of investigating terrain remotely have been employed in recent years in preliminary investigations of sites.

Remote sensing, like aerial photography, consists in the identification of variations in the appearance or characteristics of natural physical features such as vegetation, soils, etc., and the interpretation of unseen conditions from the visible variations. Remote sensing techniques include the use of radar, infrared photography, thermal sensing and television. These techniques, used alone or in conjunction with airphotos, save valuable time in locating rock exposures and in revealing subsurface geologic structures. Surface indications of linear features such as faults or shear zones are valuable applications of remotely-sensed information. Additionally, the character and distribtuion of landforms, the distribution of vegetation, the occurrence and character of stream patterns and the distribution of soils as determined through remote imagery can indicate much about the subsurface bedrock.

Some of the more successful remote-sensing techniques which have been used in recent years include color infrared photography, side-looking radar imagery and thermal radiometric imagery. Special color films which are sensitive to light with wavelengths in the near-infrared range can be used with selected filters to obtain indications of geologic features that are not visible in conventional black-and-white or color aerial photographs. With side-looking radar equipment, short pulses of radio energy are emitted along a narrow beam through the use of a scanning directional antenna. Energy reflected back to the antenna from terrain features can be displayed in various ways (e.g., a cathode ray oscilloscope). The characteristics of the radio energy are such that ground and bedrock exposures can be investigated in regions where heavy forest cover may preclude the use of photographs. Thermal images are obtained by scanning the earth's surface with a radiometric system sensitive to very slight differences in the levels of thermal radiation. This thermal radiation is transmitted as radiation in the infrared range with wavelengths between about 8 and 13 microns. A system of thermal imagery produces representations of areas where heat emissions vary. This technique is useful in detecting bodies of soil or rock which contain groundwater and which therefore exhibit temperatures significantly different from surrounding areas. Subsurface features such as fault zones or shear zones which are saturated with groundwater often can be identified using thermal imagery.

In addition to examining maps, air photos and imagery produced by remote sensing, the investigator can review the existing engineering and scientific literature for descriptions of other projects in the area of the proposed project and for the experiences with similar projects in other areas. The expenditure of time and money involved in a library research effort by one or more engineers or geologists is virtually inconsequential compared to the costs involved in carrying out field testing or exploration. For this reason, a necessary (and usually profitable) preliminary effort should always be made to uncover published information concerning the characteristics of the materials at a particular site or at nearby sites. Additionally, efforts should be made to review the experiences with similar projects (tunnels, highway cuts, etc.) which are discussed in the technical literature in order to uncover potential pitfalls and possible problem areas for the proposed facility. Perhaps the most profitable investigation of this sort is the examination of any existing construction in the area of the proposed facility. This examination should include not only a review of published information on such projects but an actual site inspection of any existing structures or facilities near the site of the proposed facility. The review of published information and the inspection of completed facilities near a proposed construction activity will not only point out possible areas of difficulty during construction and operation of the proposed facility, but will greatly assist in the planning and execution of detailed physical investigations of the site.

As a final step in the reconnaisance explorations carried out in connection with rock construction, a physical geological reconnaisance of the actual site should be accomplished. There is no substitute for walking the site. This reconnaisance should be carried out by competent engineering geologists accompanied by project engineers so that communication concerning pertinent features can be established at the earliest possible time. Included in the geological reconnaisance should be an inspection of all outcrops of rock at the site under consideration. The information obtained from an inspection of the outcrops should be used to supplement the information previously gained through an inspection of existing geologic maps and publications. As mentioned in the previous discussion of the use of maps and other publications, this information should be presented in a systematic manner utilizing stratigraphic columns and other similar devices. It is imperative that all indications of faults or other major discontinuities be fully represented in the reports from the outcrop inspection. Similarly, every effort should be made to map joints present in the site outcrops and in nearby exposures of the site strata. The results of the inspection should be presented in the form of site maps showing geologic structure and orientation. Various special projections such as equal area and stereonet projections should be utilized in representing the orientation and occurrence of joints. As a summary of the investigation up to this point, the information gained from the geological reconnaisance should be used together with the information gained from a search of the existing literature to classify the landforms and structures apparent at the site and to predict as far as is possible the subsurface conditions likely to be encountered during the actual construction and operation of the proposed facility.

FEASIBILITY INVESTIGATIONS

After preliminary or reconnaisance investigations have been carried out to provide the investigator with an overall impression of site conditions, a detailed investigation can be carried out to determine the feasibility of construction of the proposed project at the site under consideration. The information gained during the reconnaisance survey and literature review can be used very profitably in planning the feasibility investigation. In general, the purpose of the feasibility investigation is to locate and characterize the mass features of the rock strata at the site. Emphasis is placed upon locating discontinuities and assessing their influence on the behavior of the rock mass. Structural conditions are examined in a variety of ways with a view toward predicting the behavior of the rock mass under the geometry and imposed loadings associated with the proposed construction activity. The feasibility investigation can be carried out in an indirect manner by using geophysical techniques or it can be undertaken through direct probing of subsurface materials with soundings, borings, trenches, etc.

Geophysical exploration techniques consist essentially of measuring differences in physical characteristics of earth materials and differences in response of earth materials to stimulation by physical impulse. Included in geophysical exploration techniques are seismic reflection and refraction surveys, electrical methods designed to measure natural electrical potentials and differences in electrical resistivity of earth materials, techniques based upon differences and variations in the gravitational field of the earth, methods measuring variations in magnetic properties of rock and techniques based upon the measurement of radioactivity in rocks. Seismic methods are based upon the fact that there are differences in the velocities of propagation of seismic impulses among different types of rock. Because of the characteristics of the disturbance created in an elastic medium, it is possible to roughly estimate the elastic constants of rocks on the basis of the velocity of propagation of seismic waves through the rocks. A more reliable application of seismic techniques is the detection of differences in rock such as boundaries between various types of rocks, discontinuities, etc., based upon the differences in propagational velocity of seismic impulses. In certain applications, waves are reflected from strata and in other applications the refraction of seismic waves is used to establish the location and extent of rock strata at depth. One of the most widespread uses for seismic techniques is the determination of depth to rock from an overburden surface.

Electrical techniques for geophysical exploration are based upon differences in electrical resistivity and electromotive potential from one type of rock to another. In one application, electrical potentials are investigated; such potentials may develop through electrochemical action between minerals and solutions with which the minerals are in contact. These techniques based upon self-potentials have little application in rock engineering. A more generally applicable set of techniques is based upon the measurement of differences in the apparent electrical resistivity of different earth materials. These techniques are most applicable for the study of subsurface conditions at depths not exceeding several hundred feet. The greatest success in the use of electrical resistivity techniques has been in the correlation of various rock units and in the detection of bedding planes and boundaries between strata on the basis of different electrical potentials. Additionally, electrical resistivity techniques have been used successfully in the location of saturated zones within otherwise dry rock masses and in the location and correlation of geologic materials with very low resistivity or very high resistivity. Resistivity techniques have been useful in locating groundwater elevations and in detecting water-filled discontinuities or shear zones. In addition to electrical resistivity methods, other methods based upon electrical properties of materials have been devised on the basis of differences in electrical and magnetic fields within rock. However, these methods have not been extensively applied and are not as important in rock engineering as are electrical resistivity techniques.

Of only minor importance in rock engineering have been gravitational, magnetic and radioactivity methods of geophysical exploration. Gravity methods have been utilized extensively in exploration where anomalies in the earth's gravitational field are detected and associated with particular geologic structures. Additionally, gravity anomalies occur wherever there are near-surface changes in the mass density of earth materials. The most extensive use of gravitational methods has been in petroleum exploration. Magnetic techniques have been used to detect differences in bedrock beneath thin layers of overburden and have been used extensively in studies of polar migration and continental drift. These techniques have found limited use in rock engineering, primarily in applications where the results of exploration in several different localities are being correlated. Radioactivity methods have been utilized to a very limited extent. The primary use for geophysical techniques based upon radioactive properties of rock has been in the prospecting for economic deposits of uranium and similar materials. However, the radioactive emissions from various types of rock significantly differ and these differences have been used in differentiating strata.

Basically, geophysical methods serve as a useful transition in the exploration program between the examination of existing published data on the geologic characteristics of the site and detailed direct physical examination through subsurface exploration. Geophysical methods based upon physical differences between rock types, as displayed through emissions or responses to impulses, can be utilized to detect the presence of some types of discontinuities and to define in a preliminary way the geologic structures at a given site. In this way, geophysical exploration techniques are useful in the planning of effective and economical subsurface direct exploration.

After the conclusion of the geophysical exploration phase in the investigation of a particular site, the overall feasibility survey can be continued through the use of direct subsurface exploration techniques. In some cases, the most effective and efficient means of subsurface exploration is simple probing for the rock surface beneath an overburden cover. In many cases the overburden soils will be of such poor quality that it is immediately obvious that structural loadings must be supported on bedrock. In such cases, a very useful technique is simple probing with metal rods or similar crude devices in order to locate bedrock. Of course, more detailed investigation and examination will be necessary but the initial establishment of bedrock elevation over an entire site may have important connotations for the overall feasibility of the proposed construction. Detailed exploration of the subsurface is carried out through borings, trenches, exploratory adits, etc. Exploration by boreholes is by far the most common technique for subsurface exploration. At the present time, the operation of advancing an exploratory hole through soil and rock strata usually is carried out in such a way that a core of the penetrated material is recovered. This core can be examined in the field and in the laboratory. In connection with field examination, two simple and useful measurements are percent core recovery and Rock Quality Designation (RQD). The RQD, unlike the simple percentage core recovery, is defined as the total length of unweathered pieces of rock core greater than 4 inches in length in a core run divided by the length of the core run -- usually 5 or 10 feet. The RQD has been correlated with mass properties of rock and serves as a more useful measure of stability and strength than does the simple core recovery. In addition, at the time of the coring operation, the investigator should examine and fully describe the recovered core and the included discontinuities. In many cases, depending on the orientation of the core to geologic features, it is possible to determine the strike and dip of discontinuities in the core. Holes often are drilled at an angle to the surface to facilitate orientation of the core. It is important that this examination take place immediately after drilling since some rock materials deteriorate significantly with time and exposure to the atmosphere.

In addition to the recovery of a rock core for examination, the advancing of a bore hole into rock strata provides access to the subsurface for a wide range of tests. These tests include the application of geophysical methods to the materials exposed in the walls of the borehole and the direct physical measurement of rock response to imposed stresses in the walls of the borehole. One of the simplest measurements that can be carried out in a borehole is measurement of the diameter of the hole along its length by means of spring-loaded calipers. This simple measurement shows the location of boundaries between strata, weathered zones, and cavities. Since rock strata of differing characteristics respond in different fashions to the drilling of the borehole, a slightly larger diameter hole generally will be produced in weaker rock.

A number of geophysical techniques are available to validate these boundaries and to further explore the rock in the walls of the borehole. For example, a cylindrical sonde can be utilized to test the electrical properties of the rock in the sides of the borehole. Such a sonde usually is equipped with two contact points and an electrical potential is established between the two points. Two additional points can be installed in the device so that the electrical resistivity of the rock contacted can be measured in a manner similar to the determination of electrical resistivity on the surface of the earth. Additionally, self-potentials can be monitored through the use of a down-the-hole device. Cross-hole and up-hole seismic surveys can be used to supplement information obtained from surface seismic work. The up-hole technique employs a small blast charge at depth in a drill hole. Geophones are placed in contact with the side of the same hole or along the rock surface to measure seismic velocities. In cross-hole seismic work, the blast charge is placed at depth in one drill hole and a geophone array is placed in another hole some distance away. Cross-hole and up-hole seismic surveys are well suited to the determination of rock structure, weathering profiles, and seismic characteristics at depth.

The principle of the up-hole technique has been streamlined in a widely used geophysical device called the sonic logger. With this device, a continuous seismic refraction survey can be made along the walls of the borehole. The sonde contains a sonic pulse generator at one end and a receiver at the other end. Using compression and shear wave velocities obtained from the logger, investigators can calculate dynamic elastic Young's moduli, shear moduli, bulk moduli, and Poisson's ratios. The measured velocities are used to correlate strata and to infer the location of discontinuities within the rock mass.

Although seismic and other remote surveys within a borehole can provide a wealth of useful information, there is no substitute for visual examination of the rock at depth. Cores from a borehole do not always answer all the questions of the investigator, especially where core recovery is low or where weathered seams, open cavities, or filled voids are a matter of concern. To facilitate direct examination of the rock mass at depth, visual observation of the walls of the borehole has been made possible through the use of borehole cameras. The initial development of cameras for use in boreholes utilized a conical lens and produced a doughnut-shaped picture illustrating the walls of the borehole around the complete circumference of the hole at a given elevation. Exposures could be made at widely spaced intervals or a continuous picture could be obtained. Through the use of scribe marks or similar indications on the rock wall of the borehole, the photographs so obtained could be oriented and any difference in rock characteristics or discontinuity could be spatially oriented. Within recent years the initially developed borehole camera has been modified to include the use of television. In this way, an investigator at surface can rapidly obtain a complete visual representation of the entire depth of a borehole and can easily detect weakened or altered zones or the presence of structural features such as joints and faults.

Because of the expense involved in core drilling, there is an increasing trend toward the use of percussion holes for exploration. Logs prepared by remote sensing with geophysical equipment and the borehole television camera can provide as much (and often more) information about the rock mass as a whole than can be obtained from rock cores. Where percussion drilling is used extensively, it is generally necessary to provide several cored holes for identification of rock types, for obtaining samples for laboratory tests, etc.

The permeability of the rock exposed in the walls of a borehole can be determined using any of a number of standard techniques. In the most useful type of tests, water under known pressure is applied to an isolated section of the borehole. Mechanical or pneumatic rubber packers are inserted below and above the section to be tested and expanded to isolate that section of hole. Water is then pumped under known pressure to the isolated section and the flow of water into the rock for a given period of time is monitored. As an alternative, one packer can be used at various elevations in the hole. Flow to each segment of hole below the packer are assumed to be equal to the difference in flow between successive tests for greater lengths of pressurized hole. Water testing of isolated zones also can be done during drilling by using one packer placed a short distance above the bottom of the hole. In addition to permeability tests, simple observations of water level in drill holes often can provide valuable information about groundwater conditions and permeability.

The use of permeability testing is of great significance in evaluating the feasibility of sites where the flow of water through the rock mass is either detrimental or desirable. Permeability profiles can sometimes be used to advantage for correlation of weathering, structure, and rock type.

Where the nature of the project warrants, exploration of the subsurface can be expanded through construction of deep trenches, exploratory adits, shafts or pilot tunnels. These openings allow maximum access for detailed analysis of the rock mass and provide room for large-scale in-situ determinations of those physical properties of the rock mass which are important in design. The response of the rock mass to various excavation and support techniques can also be studied in these openings.

Because of the expense involved with such large-scale exploration procedures, they are reserved for projects, generally, where the expense can be offset by less conservative designs based on data obtained from the exploration program. Although there have been exceptions, large-scale access into the rock mass usually is not provided until after the feasibility of the project has been established.

DETERMINATION OF DESIGN PARAMETERS

After completion of the feasibility testing and exploration described in the previous paragraphs, a further phase of investigation must be undertaken to determine the characteristics of the rock mass for use in design of the structures to be built. The permeability, in-situ state of stress, shear strength and deformability of the rock masses at a particular site can be determined quantitatively. More detailed direct testing of the rock mass must be accomplished in order to determine these parameter values.

As mentioned in a previous paragraph, the local permeability of various rock units at a site may be determined through the use of packers in boreholes. Of more general applicability for the determination of overall permeability of the rock mass are so-called multiple-hole pumping tests. In these tests water levels are monitored in a series of boreholes and pumping is conducted in one of the holes. Standard techniques of groundwater hydrology can be employed in a determination of the overall mass permeability of the tested rock strata. These tests are of primary use in the design of and exploration for water retaining structures.

Of more general importance is the determination of the strength of the rock mass with its discontinuities and its ability to support imposed loads, stand on slopes, etc. Various testing methods have been developed to determine the shear strength of rock blocks and discontinuities within a rock mass. Tests can be performed in the laboratory or on large blocks prepared in situ. In the field, a mass of rock can be isolated through properly placed saw cuts or overlapping drillholes and subjected in place to normal and shearing forces to determine the shear strength of intact rock, of discontinuities, or of combinations thereof. So-called "shear box" tests can be conducted in which a mass of rock is completely removed from the surrounding rock and encased in a shear box. The upper and lower halves of the box then are sheared with respect to each other and the strength of the rock mass along a predetermined plane can be measured. All of these tests are designed to furnish shear strength parameters for use in analysis and design.

The design of structures on and in rock often requires the determination of the static deformability of the rock mass. The deformation response to applied loads is of importance, for example, in the design of underground structures, concrete dams, tall buildings, and structures sensitive to differential settlement. As is the case for tests of shear strength of rock masses as discussed in the preceding paragraph, access to the rock mass must be obtained either on the surface or in boreholes, test shafts, adits, or pilot tunnels. Since the deformation characteristics of a rock mass are to a large degree governed by discontinuities within the mass, three considerations are important. First, the size of the test should be such as to stress as many discontinuities as possible; second, the modulus of the mass is seldom equal in all directions and tests in several directions may be necessary to provide sufficient information for design; and third, tests should be conducted at the stress levels imposed by the structure since the stress-strain relation for rock is rarely linear.

One of the most reliable techniques which can be employed in openings in the rock is the application of pressure to opposite sides of an opening through the use of plate jacks. This test is ideally suited for different orientations and can be made large enough to give representative values for most rock masses. Jack pads approximately 3 feet in diameter are common in the United States. In this method, struts are placed across an opening. These bear on pancake-shaped flat jacks resting on carefully prepared rock surfaces on both sides of the opening. Deformations are measured across the opening and within the rock mass below and adjacent to each jack pad. The loads and deformations obtained from the test are used with various equations from mechanics to calculate the pertinent moduli.

Where important discontinuities are too widely spaced for adequate testing with plate jacks of moderate size, an entire length of test adit can be stressed by using the radial jacking technique developed by the U.S. Bureau of Reclamation or by using a pressure chamber constructed by erecting bulkheads at the ends of the test section. Because of the complexity and high cost of radial jack and pressure chamber tests, they are not used commonly.

Testing procedures are available also for the determination of moduli from an exterior surface. Devices such as the Menard Pressuremeter and the Goodman jack have been designed for use in boreholes. Instruments are available for all types of material ranging from the stiffest rock to soil. Although the details of design and operation vary considerably in borehole devices, the basic principles involved in the determination of moduli with these devices are the same. The device imparts a stress normal to the side of the hole. Data on this stress and the resulting volume or diameter changes in the hole allow calculation of moduli perpendicular to the axis of the hole. Borehole devices have two major drawbacks which must be taken into account when they are used. The volume of rock under stress is usually small, and the modulus obtained is often applicable to rock deformation in directions oriented as much as 90° from the direction critical for design of the structure.

Both of the shortcomings associated with borehole devices can be overcome with large-scale cable jacking tests conducted from the rock surface. In this type of test, a long cable is anchored at depth within the rock mass. It is tensioned from the surface with a large hydraulic jack. Since the force of the jacking is applied perpendicular to the rock surface (and is usually vertical), moduli can be calculated for direct use in the design of building foundations and similar structures.

A common failing of all in-situ deformability tests is the fact that the tests are conducted on surfaces disturbed by exposure, drilling, blasting, etc. For this reason, considerable judgement is necessary in the interpretation of deformability tests.

In order to determine the behavior of rock mass in response to structural loading, it is sometimes necessary to determine the in-situ state of stress in the rock mass prior to construction. A number of specialized techniques have been developed for such stress determination. Determination of the state of stress which exists in a rock mass at a particular point can be accomplished by relieving the stress at that point and monitoring deformations. In one of the simplest tests of this kind, a slot is cut into a rock surface at an exposure where the determination is to be made. This slot can consist of a series of overlapping holes or a smooth cut made with a rock saw. Before the cut is made, monitoring points are established around the slot. Rock deformations are measured as the slot is cut. Commonly, measurement of deformation continues for several days after the slot is cut. After the deformation has stabilized as shown by the measurements, a flat jack is inserted into the slot and the space between the flat jack and the rock wall of the slot is filled with grout. After the grout has hardened, the flat jack is expanded through the application of hydraulic pressure and the rock is restored to its original position. When all deformations have been fully cancelled, the stress applied by the flat jack is taken to be the stress which existed in the rock prior to cutting of the slot. The major shortcoming of this technique is that stresses are measured near a surface and not within the rock mass. Because of the presence of the surface, stresses measured by the flat jack technique generally are not representative of stresses within the rock mass. To avoid this problem, several methods are available for the determination of stresses at depth in a borehole.

The most reliable and widely used method is based on an overcoring technique developed by the U.S. Bureau of Mines. In this method, a 1.5-inch diameter hole is drilled beyond the depth at which stresses are to be measured. An instrument which measures diameter changes in three directions is inserted then into the hole. The small hole then is concentrically overcored with a 6-inch diameter core barrel to relieve the stresses in the resulting annulus of rock. This cycle is carried out at 1 to 2-foot intervals to a maximum depth of about 40 feet. Sections of the rock annulus then are stressed externally in a pressure chamber with the instrument at the same location and orientation as during the overcoring. The resulting stress versus hole deformation data along with hole deformation data measured during overcoring permit the calculation of in-situ stresses. If three holes are drilled at different angles (generally orthogonal), the three-dimensional state of stress including principal and shear stresses can be calculated. Another stress relief technique involves overcoring an instrumented plug glued to the bottom of a borehole. In addition to the overcoring techniques mentioned above, a method has been developed to determine stresses using hydraulic fracturing.

In this method, a section of borehole is isolated with packers. The isolated zone then is pressurized; pressure is increased until a tension failure is caused in the rock. The stress required to cause the tensile failure, the orientation of the parting, and the tensile strength of the rock are used to calculate the stress perpendicular to the plane of the crack. This method is limited in that a complete state of stress cannot be determined. However, the method can be employed at much greater depths than can other available methods. Hydraulic fracturing has not been used widely by civil engineers, but it shows promise for future investigations where deep underground works are involved.

In addition to the tests previously described, other miscellaneous test methods have been developed for use in determining the characteristics of a rock mass which govern design. For example, for some projects a serious question may arise concerning the durability of exposed shales. In this connection, field exploration should include some observations of in-situ durability as well as laboratory testing. Other specialized tests have been developed for use in field exploration where particular attributes of the rock or specific features of a given construction project dictate the development of specialized tests.

MONITORING CONSTRUCTION

Field exploration and testing of rock do not cease when construction operations begin. The actual construction operation offers an opportunity for full-scale testing and direct observation of the behavior of the rock to vindicate the criteria used for design. This generally is not possible in any prior stage of exploration and the opportunity cannot be neglected. Often initial designs are modified and construction programs are changed because of conditions or characteristics in a rock mass which are found during construction. When instrumentation is used to monitor possible trouble areas, a well-defined chain of command for the trouble-shooting process should be established at the outset of construction, since many failure situations in rock are time-dependent. For example, a slope will creep, perhaps at an increasing rate, before failure. A failing arch in an underground opening may show steady-state or increasing deformations with time, often aggrevated by adjacent blasting. Advance planning can reduce greatly the instrument-data analysis time, design time and construction time required for remedial measures, and often a complete failure can be averted with fast, efficient corrective measures. The amount of additional support required to rectify a stability problem can be reduced many times, for example, by fast action, since rock masses generally weaken as a function of internal deformation.

In addition to the obvious operation of visually examining exposed rock surfaces, the most common activities during construction are the measurement of displacements in the rock mass, porewater pressures, and strains in required support systems. Of course, on projects where access to rock strata is not possible prior to construction, pertinent in-situ strength tests and other direct determinations of rock mass properties cannot be undertaken until construction has begun.

One of the most important considerations to be made in designing an instrumentation program to monitor construction is to ensure that the measuring devices themselves do not directly affect the quantity being measured. Two types of instruments are available. For sake of clarity, herein they will be termed "passive" instruments and "active" instruments. Passive instruments are not load-carrying members and their readout yields data on strain or displacement only. Since they do not share stress with the body being monitored, they have little if any effect on the behavior of the body and simply "go along for the ride". Active instruments, on the other hand, usually are designed to measure directly changes in stress or force, and all or part of the force in the body being monitored passes through the instrument.

Passive instrumentation is available in many forms. One of the most overlooked and neglected (and yet most readily available) methods of monitoring deformation behavior during construction is the use of bench marks for which line and grade can be established periodically by surveying. First-order accuracy should be required for all surveys used to define behavior. Care also should be taken that the reference point for surveys is well outside the area influenced by the construction project.

The distribution of deformation with depth below a rock surface can be determined with borehole extensometers. These instruments are installed in boreholes and consist of an anchor at the inner end of the hole and another anchor at the collar of the hole. Through the use of a rod or a spring-loaded wire which is fixed to (and moves rigidly with) the inner anchor, relative movement between the two anchors can be measured. Remote readout units equipped with a variety of displacement transducers are available. The multiple-position borehole extensometer has several anchors at different depths in the borehole, all referenced independently to the collar anchor. Sophisticated remote readout units have been installed successfully with monitoring depths of up to 200 feet. Simple single position rod-type extensometers which are monitored with a dial gauge or micrometer are limited to monitoring depths of about 75 feet. Simple extensometers can be installed in closely spaced clusters to achieve the multiple-position effect.

Another tool in the class of passive instrumentation is the inclinometer. This instrument, which is used most often to monitor slope movements, consists of a sonde lowered on an electrical cable into a vertical or inclined hole cased with plastic or thin aluminum tubing. The sonde is used to measure the angle between the sonde axis and vertical. It travels on spring-loaded wheels which ride along grooves in the casing. The position of the casing relative to its original position is calculated using a summation procedure applied to the measured angles and the distances between readings.

Passive instrumentation also includes many other devices such as vibrating wire or electrical resistance strain gauges, piezometers, seismographs, etc. Generally, there is a wide selection of instruments to meet any particular demand. To reinforce the reliability of measurements, it is often highly desirable to capitalize on variety by using two or more redundant instruments of different types to measure the same quantity.

Active instrumentation usually involves the measurement of stress or force. A good example of active instrumentation is a load cell placed in series with a rock bolt or steel tunnel support. The bolt or the support transmits force through the load cell to a reaction point. Valid measurements of stress or force cannot be made if the deformation behavior of the loaded members plus the instrument is different from the behavior of the members without the instrument. If the instrument increases or decreases the effective stiffness of the system, it will tend to attract (or shed) load and obtained data will be erroneous.

Other examples of active instrumentation are the inclusions sometimes placed in boreholes to measure changes in stress in the rock mass. As was stated for the load cells discussed above, if the stiffness of the inclusion is not matched to that of the rock mass, its presence will affect the flow of stress and erroneous results will be obtained.

In general, active instrumentation should be avoided unless an uncommon amount of effort can be expended to prove the validity of the obtained data and the propriety of installing such instruments. It is far better to measure strains passively and to compute stresses based on measured stresses and modulus values obtained in the field or in the laboratory than to measure stresses and forces directly with active instrumentation.

The monitoring systems and devices which are installed during construction operations can be left in place and monitoring of performance can continue in completed structures long after construction operations have ceased. The value of these long-term measurements during operation of a completed facility should not be misjudged. Such long-term measurements can be very important to the safety and proper operation of such facilities. In rock masses which exhibit creep characterisitics, the long-term behavior of the rock mass after construction is completed can be significantly different from its short-term response. Additionally, long-term changes in rock characteristics in response to environmental conditions such as changes in water pressures and flows can be monitored for their effects on the integrity of completed projects. In general, the technology which is used to accomplish post-construction monitoring is exactly similar to that utilized prior to and during construction. However, emphasis should be placed on long-term reliability and redundancy of measuring systems and devices when consideration is given to post-construction monitoring.

PLANNING FIELD EXPLORATION AND TESTING

In planning and selecting the types of tests and exploration methods to be carried out at a particular site for a specific construction activity, the investigator must apply systematic criteria. First of all, in planning exploration the character of the project must be comprehensively examined. The consequences of failure in the constructed facility must be examined. The consequences of failure of a high rock-fill dam located near densely populated areas will be vastly different than the consequences of failure of a rock slope adjacent to a construction haul road. Obviously, more complete and detailed exploration will be justified for projects where the consequences of failure are great. In any assessment of failure consequences, the greatest emphasis must be placed upon danger to human life. A second factor in characterizing the project is the overall cost of the project and the amounts of money available for exploration operations. The scale and diversity of exploration operations will be directly related to the overall importance of the project as expressed in terms of construction cost. This analysis can be likened to an assessment of the consequences of failure from the point of view of money involved and not from the point of view of danger to life. An additional consideration in characterizing a project prior to planning for field exploration is the establishment of performance criteria for the completed facility. For example, in a water retaining structure, a primary criterion for performance is the limitation of seepage losses through the structure and through underlying and surrounding rock and soil materials. Obviously then, the exploration carried out in connection with such a project would be directed toward establishing the overall permeability of the surrounding rock masses with a view to identifying possible areas of seepage loss. In other projects, the overall stability of the rock mass may be of primary concern and the exploration program can be directed toward stability analyses and the determination of information needed for such analyses.

In addition to a characterization of the project and the site itself, the various test methods should be characterized prior to their inclusion in any exploration and testing program. Serious consideration should be given to the types of information which can be gained from any exploration technique or test method. For example, the applicability of the information should be assessed. Is the information of general applicability for all facets of the project or is a specific quantity of limited applicability being produced through the application of this method? In some cases, a simple test which produces information of general applicability is much more valuable than complicated and expensive tests which yield information of only very specific character. Obviously, the costs of test methods and exploration techniques must be assessed during the selection and planning of the testing program. Whenever the monies available for exploration are limited -- and such is always the case -- the cost of exploration procedures and the costs of test methods must be closely examined. A ratio of benefits gained in terms of information furnished compared to costs per test or per foot of borehole must be established. The cost of some test methods may immediately preclude their use from certain projects. For example, the use of a pressure chamber test would be totally inappropriate during foundation exploration for a warehouse structure and, for that matter, for many dams. The total project cost and the range of loadings associated with such a structure would hardly necessitate the intensive testing of rock by means of a very expensive tunnel and the equipment and costs involved in pressurizing a section of test tunnel.

In addition to assessing the information gained from various exploration techniques and establishing the cost of particular exploration and testing methods, consideration must be given to the reliability of the readily available personnel and equipment which could be used for conduct and interpretation of the various exploration and testing methods. A considerable amount of experience and expertise is necessary for the operation and interpretation of the results produced by some of the sophisticated devices used in testing rock. Geophysical testing techniques are particularly susceptible to random variations in material, and considerable experience is required for the interpretation of data obtained through geophysical exploration. Additionally, the reliability of various pieces of equipment necessary for the conduct of monitoring or testing operations should be judged. In remote areas of the world it may not be possible to obtain at short notice sophisticated pieces of equipment or equipment which can be considered reliable. In these situations, a more judicious expenditure of funds may be realized through the conduct of many simple tests designed for use by unskilled workers or uninformed technicians.

SUMMARY

In summary, it may be stated that field exploration and testing of rock is very important for the safety and efficiency of structures built in and on rock. More efficient construction operations result when exploration has been carried out in a conscientious and comprehensive fashion prior to construction. Lower bids for construction prices are produced through the circulation of comprehensive prebid information on rock characteristics and behavior. Safe and competent designs for openings and structures in and on rock can be achieved on the basis of information secured during exploration and field testing. Finally, the long-term reliability of constructed facilities which rely on support and containment from rock masses can be assessed only through adequate monitoring and exploration in rock prior to, during and after construction. In selecting exploration and testing techniques, the project proposed for construction should be examined and characterized for its importance and costs. Performance criteria for the completed project should be established and the exploration and testing program designed to correlate with these performance criteria. Finally, the exploration and testing techniques themselves must be examined to determine what information is gained from each test, what is the cost involved in each test or exploration technique, and what is the reliability of the results obtained from available personnel and equipment for each test.

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Exploration in rock is directed toward locating and characterizing mass features such as joints and faults. The joints apparent in the rock mass at the right have caused the failure of this highway cut. Now they can be located and studied in detail. Not so apparent are the echelon faults exposed during excavation for a dam foundation, as shown below.

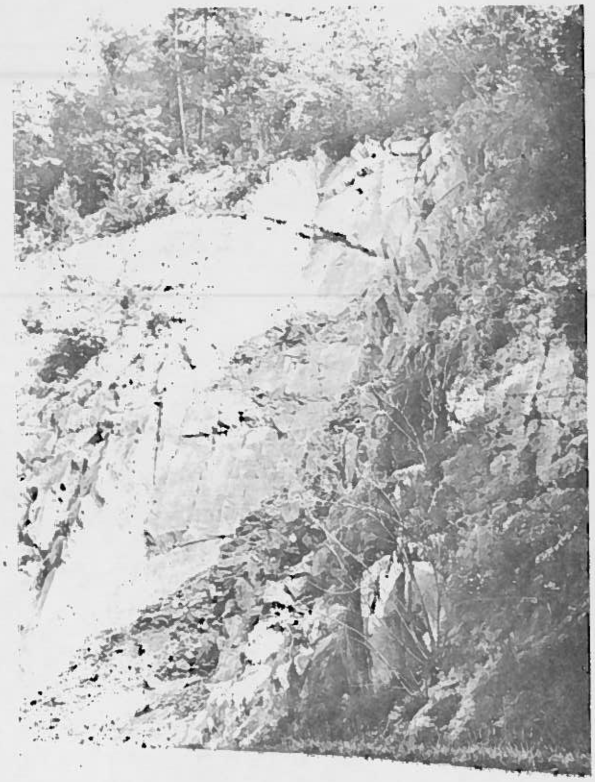

Figure 1.

Figure 2.

Figure 3.

The soluble nature of some limestones makes rock exploration very difficult in such rocks. Above, solution along surficial joints has produced very irregular bedrock topography. Below, lateral expansion at depth with subsequent solution along joints has produced subsurface cavities of which there are no indications on the rock surface. See also Figures 5 and 6.

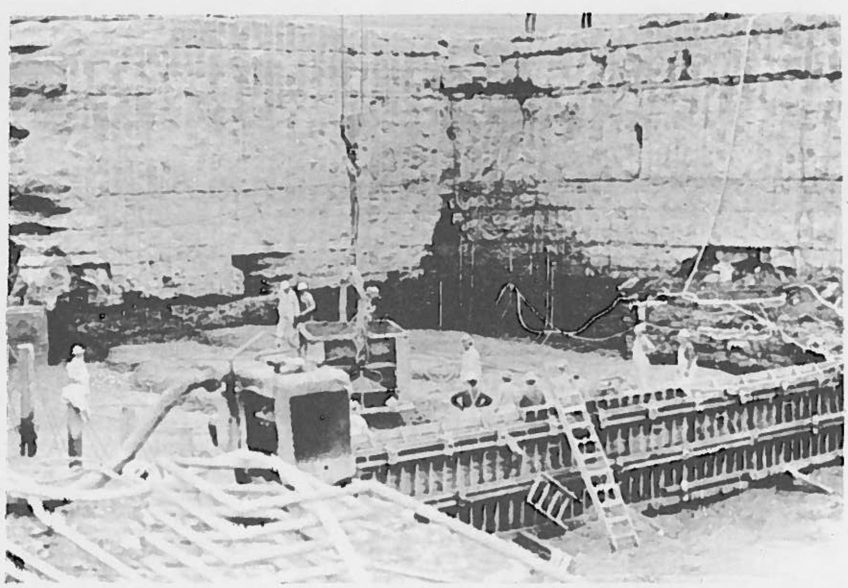

Figure 4.

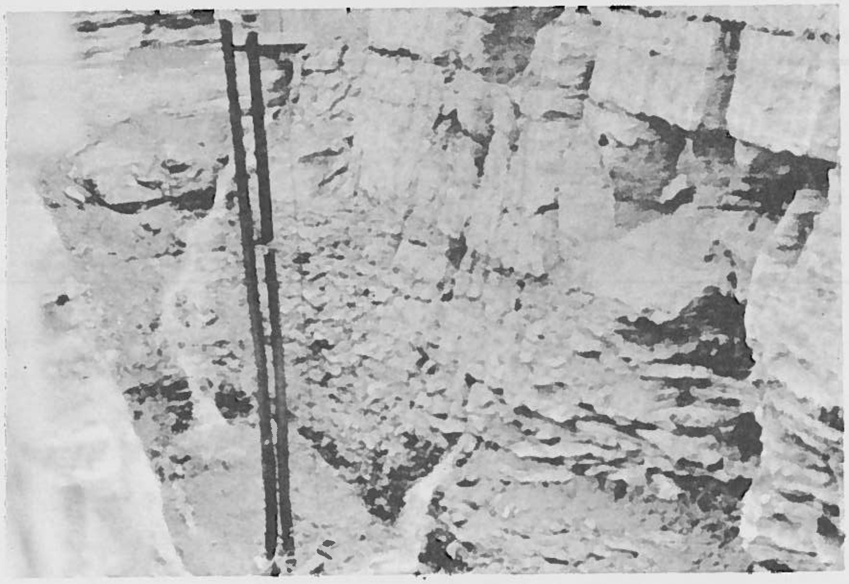

Figure 5.

Water-bearing solution cavities in limestone in a dam foundation are illustrated. Joint widths at the rock surface varied from practically zero to no more than 1/8-inch.

Figure 6.

Figure 7.

During construction, piezometers were installed in this dam foundation as shown above and below to facilitate long-term monitoring of porewater pressures. A major fault was discovered during construction and is shown (foot-block surface) at the left in these figures.

Figure 8.

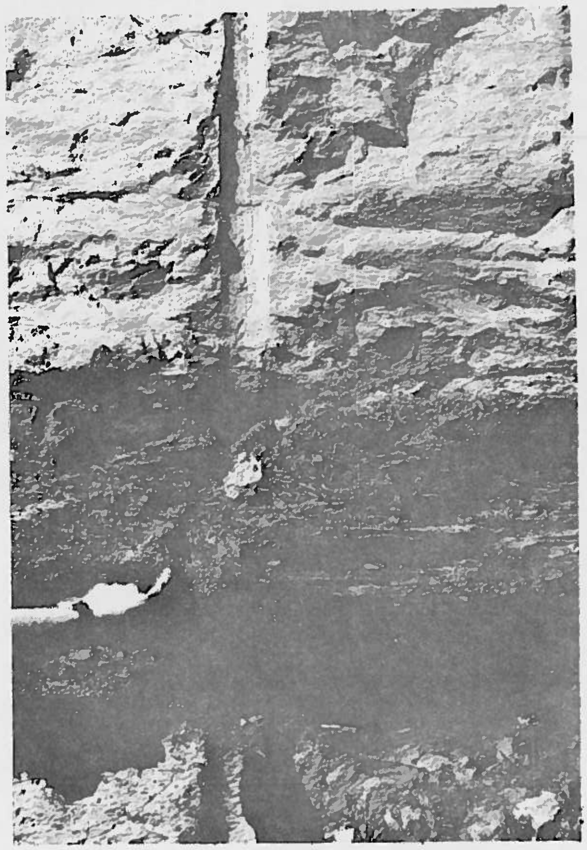

Figure 9.

At times, direct observation of displaçements by simple means can be a very valuable technique in monitoring rock mass behavior. The offset in the drillhole cast shown above indicated a significant movement of the upper limestone stratum and the underclay layer and gave warning of impending sliding failure of the upper stratum which supported a retaining structure.

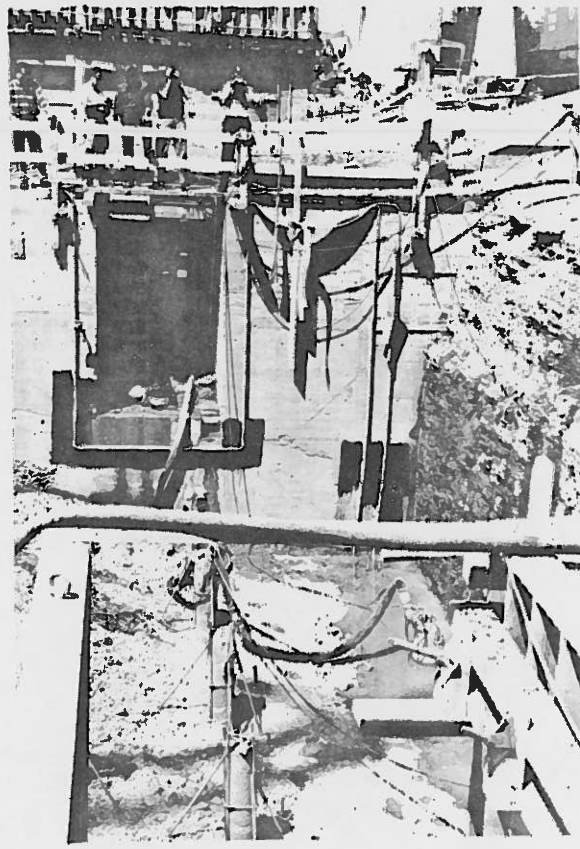

Figure 10.

Provision of galleries in dams and other massive structures permits access for various forms of rock-mass monitoring after completion of construction, especially for measurement of water pressures.

A ROCK CLASSIFICATION SCHEMA

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INTRODUCTION

The need for engineering data on earth materials for use in site selection, design, construction, and maintenance of major engineering structures is generally accepted. Probably the most pressing need is for such data to use in preliminary considerations of site selection and design alternatives. Maps and(or) surveys giving the areal distribution of earth materials and their characteristics, together with topographic maps available for many areas, would permit much preliminary work on engineering structures to be done without the engineer ever having to leave his office.

Field and laboratory methods used to obtain engineering data are many and varied and often too expensive to use in preliminary reconnaissance surveys. There is, therefore, a need for the development and use of short-cut methods. Information on the areal distribution of soils and rocks can be inferred from aerial photographs and pedologic and geologic maps and surveys. Some four or five decades ago, when engineering activities were more restricted than they are at the present time and were founded more or less in or on **soil** materials, it was recognized that the pedological soil classification and mapping system could be of great use to engineers. Since that time, many agencies have devoted much effort to providing engineering data to supplement information provided by the pedological classifications and mapping.

In about 1955, the Research Division of the Kentucky Bureau of Highways began a program of adapting existing U. S. Department of Agriculture soils maps for engineering purposes by adding engineering data to the pedological soil series classifications. The Division has provided engineering soils data for numerous samples submitted by the Soil Conservation Service, U. S. Department of Agriculture. Other soils test data are also available from project files of the Division of Materials. As a result, soils profile data have been accumulated and tabulated for use in preliminary site investigations.

In recent years, as the size and extent of engineering structures have increased, the engineer has become more and more concerned about performance relationships between his structures and consolidated earth materials (**rock**). Because extensive areas of the country have been mapped geologically, much information is available concerning the areal distribution of **rock** materials. It would seem, however, that the use of geologic maps could be greatly enhanced for the engineer if engineering test data were provided and could be associated with various geologic formations. Kentucky has a particular advantage in that there is an extensive geologic mapping program, and within a few years, there will be complete coverage of the state with 7-1/2 minute quadrangle maps.

The initial goal of the study reported herein was to devise an engineering classification system for intact rock samples based on simple index tests which could be used to categorize Kentucky surface and near-surface rock types. This system would also provide for the accumulation of engineering test data for use in future site investigations. While conducting the literature survey, several facts became apparent:

1. a large number of rock classification systems -- geologic and technical, general and specific -- already existed;
2. an equally large number of index tests had

been devised; and

3. there was a lack of communication among those involved in specialized areas of rock-related work (geologists, civil engineers, mining engineers, etc.) and, to some extent, among individuals within each field.

It was evident that developing yet another "specialized" classification system with associated index tests would not be a significant contribution. It was decided, therefore, to develop an overall rock evaluation schema which would avoid the undesirable disparate characteristics of narrowness or over-generalization prevalent in many classification systems. It was desired also to develop the program format in such a way that accumulated information could be systematically stored for easy access and use. It was apparent that full development and implementation of a program of this nature would require years of further study and cooperation of many individuals and organizations. Such a program, properly developed and used, would substantially contribute to an advancement, and delineation of the schema and guidelines for its implementation would be a worthy goal.

A first logical step in approaching rock-related problems is the development of a systematic method of data collection. Presently, the only method of rock classification in Kentucky is geologic in nature. Engineering design values are based on empirical experience or building code values that are vague and, in many cases, overly conservative. Only in rare instances are tests actually performed. Lack of a systematic method for recording, cataloging, and storing data results in duplication of effort, loss of valuable information to the engineering community, and inadequate communication between practioners.

A second step is the development of a method of presenting collected data in a form convenient for a variety of uses. Classification systems or data banks are not ends in themselves but only provide a means for organizing existing knowledge, and facilitating interpretations. A method of further quantifying classification parameters with engineering data is needed.

The task of completely delineating, testing, and implementing a rock classification schema of the magnitude suggested is beyond the scope of this paper. It is important, however, that initial groundwork and guidelines for completion of such a program be carefully set forth. Successful completion of the program can be expected through additional studies based on the proposed guidelines. It is the intent of this paper to outline, in descriptive terms, such a rock classification program and provide sufficient guidance for eventual implementation.

The formulation of a viable rock evaluation program requires that the subject material (rock) be defined in a satisfactory manner. Since both intact and in-situ characteristics of rock are important to engineering considerations, rock must be considered both as "rock material" (intact samples), herein defined as a lithified aggregate of mineral particles in varying proportions along with associated voids (pores, microfissures), and as "rock mass" (in situ), which consists of rock material segmented by various forms of discontinuities (joints, bedding planes, faults, etc.) and associated filling materials.

Since it has been suggested that geologic maps provide much useful information for the engineer and that the usefulness of these maps could be enhanced by providing engineering data, the need for a familiarity with geology is evident. Earth materials of concern to the engineer exist in a geological environment. These materials possess physical characteristics which are a function of their mode of origin and subsequent geologic processes that have acted upon them. To adequately devise a rock evaluation program which will be useful and practical, it is essential to know the location of major structural features in a study area, the distribution of rock types, and the lithologies which have been created during geologic history. Additionally, a knowledge of local geologic nomenclature (Figure 1) is necessary so that information gained from former investigations and past experience can be incorporated into the evaluation system. Information from this base can then be used to

1. ensure that index tests selected for classification purposes are compatible with the range of rock types to be encountered,
2. locate potential trouble areas which are associated with particular types of geologic structures,
3. identify those formations which have exhibited undesirable characteristics (i.e., swelling, solution cavaties, rapid weathering, etc.),
4. evaluate the probable in-situ stresses that have developed during geologic history, and
5. provide an aid in designing a subsurface and testing program to be used for a particular project at a particular site.

ROCK CLASSIFICATION

"Rock Mechanics" may be defined as the study of basic processes of rock behavior and their technological significance. The time scale for these basic

ERA	SYSTEM	SERIES	McFARLAN'S NOMENCLATURE	FORMATION			PREDOMINATE ROCK TYPE			PHYSIOGRAPHIC REGION OF OUTCROPPING
Cenozoic	Quarternary	Holocene Pleistocene	Glacial Drift and Loess	Alluvium Loess Continental Deposits			Silts, Gravels, Sands Silts Sands, Silts, Clays			Jackson Purchase Region
	Tertiary	Eocene	Jackson Wilcox	Jackson Clairborne Wilcox			Unconsolidated Sands, Silts, and Clays			
		Paleocene	Midway	Porter's Creek			Clay, Silt, and Sand			
Mesozoic	Cretaceous	Upper	Ripley Eutaw Tuscaloosa	Eutaw Tuscaloosa			Unconsolidated Sands, Gravels, and Clays			
Paleozoic	Pennsylvanian			Western Coal Field	Eastern Coal Field		Western Coal Field	Eastern Coal Field		Eastern and Western Coal Fields
		Upper	Monogahela Conemaugh	Henshaw-Dixon Lisman	Absent Conemaugh (Boyd Co.)		Sandstones, Shales, and Coals Shales, Sandstones, Limestones, Coals	Sandstones, Shales Limestones, Coals		
		Middle	Allegheny	Carbondale	Allegheny (Boyd Co.)		Shales, Sandstones, Coals, Underclays	Sandstones, Shales, Clays		
		Lower	Pottsville	Tradewater Caseyville	Breathitt Lee		Shales, Sandstones, Coals, Underclays Sandstones	Interbedded Shales Sandstones, Siltstones, and Coals Sandstones and Conglomerates		Knobs Region
	Mississippian	Chesterian		Fluospar Region	West of Arch	East of Arch	Fluospar Region	West of Arch	East of Arch	Mississippian Plateaus Region
		Chesterian Upper	Kinkaid Degonia Clore Palestine Menard Waltersburg Vienna Tar Springs	Kinkaid Degonia Clore Palestine Menard Waltersburg Vienna Tar Springs	Leitchfield (Buffalo Wallow)	Pennington	Limestones Sandstones Limestones Sandstones Limestones Sandstones Limestones Sandstones	Interbedded Shales, Limestones, and Sandstones	Shales	
		Chesterian Middle	Glen Dean Hardinsburg Golconda Cypress	Glen Dean Hardinsburg Golconda Cypress	Glen Dean Hardinsburg Golconda	Bangor Hartselle	Limestones Sandstones Limestones Sandstones	Limestones Sandstones Limestones Sandstones	Limestones Sandstones	
		Chesterian Lower	Paint Creek Bethel Renault Aux Vases	Paint Creek Bethel Renault Aux Vases	Elwren Reelsville Sample Beaver Bend Paoli	Monteagle (Newman)	Limestones Sandstones Limestones Sandstones	Shales Limestones Sandstones Limestones	Limestones	
		Meramecian	Ste Genevieve St. Louis Salem Warsaw	Ste Genevieve St. Louis Salem Warsaw (Harrodsburg)			Limestones			
		Osagean	Waverly, New Providence, or Borden	Ft. Payne (South) Borden (North)			Cherty Limestones and Shales Interbedded Shales and Siltstones			
		Kinderhookian	Sunbury Bedford	Sunbury Berea Bedford } occur only in east			Shales Sandstones Shales			
	Devonian	Upper	Ohio Chattanooga New Albany	New Albany			Shales			
		Middle	Sellersburg Jeffersonville Boyle	West of Arch: Sellersburg Jeffersonville	East of Arch: Boyle		West of Arch: Limestones Limestones	East of Arch: Dolomites		Outer Blue Grass Region
	Silurian	Middle	Louisville Waldron Laurel Osgood Bisher Crab Orchard	Louisville Waldron Laurel Osgood	Bisher Crab Orchard		Limestones Shales Dolomites Shales	Limestones Shales		
		Lower	Brassfield	Brassfield	Brassfield		Dolomites			
	Ordovician	Upper	Richmond	Southwest Blue Grass: Drakes	Northwest Blue Grass: Drakes Bull Fork		Southwest Blue Grass	Northwest Blue Grass: Dolomitic Limestones Shales		
			Maysville Eden	Ashlock Grant Lake Calloway Creek Garrard Clays Ferry	Grant Lake Fairview Kope Clays Ferry		Limestones Limestones Limestones Siltstones Shales	Limestones Limestones Shales Shales		
		Middle	Cynthiana Lexington	Lexington Limestone			Limestones			Inner Blue Grass Region
			High Bridge	High Bridge			Limestones			

Figure 1. **Major Surface and Near-Surface Geologic Formations of Kentucky.**

processes ranges from millions of years to microseconds, from orogenesis to blasting. Mechanical properties are affected by stress history, anisotropy, inelasticity, size effects, deformability, and others too numerous to mention. Processes of inelastic, elastic, and time-dependent behavior are all natural occurrences in rock.

Testing of rock in its native environment naturally would be the best approach to determination of mechanical properties to use in the design of structures. The expense of such an approach in obtaining necessary parameters is often economically prohibitive. Elimination of direct determination of rock mechanical properties implies that indirect determinations are the next best approach to obtaining values of these properties, at least for preliminary considerations and planning. Concepts of index properties and index tests encompasses these indirect determinations of significant rock mechanical properties.

Index Properties and Tests

Even the most common rock types are composites of highly variable materials. **Intact rock** may be considered to be a solid consisting of a matrix aggregate of minerals, the properties of which are a function of the mechanical properties of the aggregate constituents and the nature of bonding between the aggregate constituents. Intact rock may be sampled and specimens devoid of large scale structural features can be tested.

In-situ rock masses, however, are affected by geological features such as partings, fractures, bedding planes, cleavage planes, chemical alteration and decomposition zones, stress history effects, and environmental changes. Physical discontinuities, present in all rock masses, occur in the form of planes or surfaces of weakness that actually separate blocks of rock material. Mechanical property tests should be conducted on a scale such that a particular test specimen includes these defects in proportion to their presence in the rock mass so as to obtain results which will be representative of behavior of the in-situ mass. As would be expected, size of the specimen that would encompass these geologic conditions would generally be much too large to be tested under laboratory conditions. The obvious solution would be to test the in-situ rock mass; this solution is limited by difficulties encountered in preparing an "area specimen" and applying a necessary and sufficient magnitude of force on undisturbed rock masses. It is necessary to develop and use simple, inexpensive, reproducible indicator tests which predict intact sample rock properties and to forecast rock mass behavior on the basis of index test values and a knowledge of discontinuities and other features present in the rock mass. Development of index tests is an integral part of any rock engineering evaluation scheme. Probably the greatest usefulness of index properties lies in the fact they provide quantitative methods for assigning a particular rock a specific classification independent of the background knowledge and experience of the operator performing the index test. Once a rock has been classified, expected ranges of the values of such mechanical properties as strength, deformability, weatherability, and permeability can be estimated. This allows design parameters to be established and alerts the engineer to potential problems and(or) expected performance.

Complexities involved in even the most superficial overview of rock geognosy require extreme simplification because of physical and mathematical continuity considerations:

1. the scale of rock discontinuities and structural features cannot be preserved in intact laboratory specimens, and thus considerable uncertainty as to the extrapolation of laboratory property values to field situations is inevitable;
2. rock discontinuities and inhomogeneities play a dominant role in terms of rock deformation and failure for both intact and in-situ conditions;
3. "constants" used in simplified mathematical models are statistical functions of these discontinuities and heterogeneities; and
4. discontinuities introduce a probability of unpredictable variations in the geologic conditions which should be considered.

Mechanical properties which are a function of the structural competence of a rock sample may be predicted on the basis of empirical relationships among "index properties" obtained in specific physical-mechanical classification tests.

Except in certain specialized applications, there are no standards to guide the engineer in selecting appropriate indicator tests. Of course, classification tests should be chosen so that, regardless of geologic origin, specimens with similar index properties should exhibit similar mechanical behavior. Obviously, an engineering classification system for intact rock should be based upon index properties statistically related to important physical-mechanical properties of the rock mass. "Index tests" are used for classification purposes and should be distinguished from "design tests," which are usually expensive and may involve considerable complexity because of size requirements and the need to simulate field conditions. In general, an index property should have three characteristics:

1. the test result must be an index of a material (mechanical) property which the design

engineer can use effectively;

2. the test should be simple, inexpensive, and rapidly performed (minimum sample preparation); and
3. test results must be reproducible, within reasonable limits, by various practitioners in various locations using standard equipment and procedures.

Additionally, index properties may be used to define exactly what constitutes rock within the context of a particular investigation. It would be useful, in many situations, to establish the index property which would delineate "rock" from "soil" or "rock-like" from "soil-like" materials.

A variety of index properties relevant to the mechanical quality of rock masses includes

anisotropy
apparent specific gravity
brittleness
brokenness
core recovery
deformation modulus
degree of alteration
dilatational wave velocity
fracture frequency
hardness (rebound and indentation)
joint extension
modified core recovery (RQD)
Poisson's ratio
porosity
relative absorption
residual shear strength
resilience
secant modulus
slake durability
swelling
tangent modulus
tensile strength
toughness
uniaxial compressive strength
unit weight
void index
water content
weatherability
Young's modulus

Additionally, complete testing of rock material should not be confined strictly to tests of the rock core; valuable information may be obtained within a borehole. Pumping tests, borehole sonic velocity, electrical resistivity, and gamma ray emission logs are useful for stratigraphic and mechanical or physical correlations. Since local or overall displacements limit the utility of an engineering structure, index tests and(or) properties that are indicative of compressibility or displacement should be included in classification systems. However, measures of deformation moduli or mass compressibilities are extremely difficult to obtain and involve complexities (state of in-situ stress, discontinuities, etc.) which are yet to be resolved.

Geologic Classification Systems

From a geologic overview, there exists an almost universal division of rocks with respect to their origin (genesis) into three primary groups:

1. igneous rocks -- rocks formed by cooling of molten magmas or by the recrystallization of older rocks after the application of heat and pressure of such magnitude as to render them fluid;
2. sedimentary rocks -- rocks formed as products of deposition of plant and animal remains, from materials formed by chemical decomposition, and from products of the physical disintegration of pre-existing rocks; and
3. metamorphic rocks -- rocks produced from pre-existing rocks by the effects of heat, pressure, or permeation by other substances.

Each of these primary rock groups have been the subject of individual rock classification systems.

One of the first classifications of igneous rock considered the general composition of the rock. Many authors have modified the original system, but essentially glassy, aphanitic, and granular igneous rocks are described in terms of their proportions of orthoclase feldspar, quartz, plagioclase feldspar, and ferromagnesian minerals. Additional megascopic classification of igneous rock is accomplished on the basis of the degree of visibility of grains (crystals) within a particular rock.

Classifications of sedimentary rocks notably group the rocks into origin, texture, and particle size or composition categories; e.g., detrital, inorganic, and biochemical genetic categories; clastic and nonclastic textural categories, and particle-size classes. Rocks of mixed fabric or composition can be further classified as to predominant constituents -- clays, sands, etc.; e.g. sandy shale, clayey sandstone, or calcareous shale.

Metamorphic rock classifications are generally based upon visible fabric and mineralogy. Foliation or schistosity is conspicuously apparent in metamorphic rocks with the general exceptions of quartzite, marble, dolomitic marble, and hornfels.

Petrographically, the most important properties in terms of a classification system are texture, structure, and mineralogical composition. Because of the lack of agreement among geologists as to exactly which physical features should be included in "texture" and which features should be regarded as "structure", the term fabric has been coined to include both concepts. Texture may be thought of as the size and shape of rock constituents, including accompanying variations of properties. Structure includes distribution and grouping of minerals, which are constituents of rock. Petrological data can aid in predicting mechanical performance (behavior); for example, microfractures detected in quartz crystals in a granite would be significant with respect to strength of granite. Megascopic fabrics in rocks also have been classified with respect to isotropy and anisotropy; e.g., isotropic fabrics and anisotropic fabrics include such subdivisions as linear, planar, intersecting planar, omni-directional planar, folded planar, and composite fabrics.

A chemical classification system is primarily useful only for rock comparison on the basis of chemical activity since, in most chemical classification systems, constituent oxides are reported as percent by weight. It is impossbile, however, to estimate many physical characteristics of a rock from chemical analysis alone since rocks of closely related chemical composition may differ in genesis as well as in texture and mineralogy. However, chemical classifications may be of use in predicting the behavior of rock in certain "chemical" applications (e.g., bituminous concrete mixtures, portland cement mixtures, resistance to chloride attack, expansibility, etc.).

Such descriptive indicators as genesis, petrography, texture, mineralogy, and chemical composition give only vague information concerning the engineering behavior and capabilities of the rock. Geologic classification systems do not give comprehensive information as to rock properties in terms of mechanical behavior of the in-situ rock masses. Geological rock classification systems emphasize the solid constituents of intact rock while an engineering rock classification should consider discontinuities of the rock mass (e.g., pores, cracks, and fissures) because of their great mechanical significance.

Topographic relief is often sufficiently characteristic to be indicative of the geology of the bedrock, even though very few rock exposures may be present. Thus, classification of landforms as they relate to errosional or depositional history and subsurface geology have been developed utilizing aerial photographs, topographic maps, and drainage patterns.

An interesting exception to the qualitative approach of most geological mapping surveys is the Pattern-Unit-Component-Evaluation. Terrain was classified into three major stages; pattern, unit, and component. A geomorphological description was found suitable for a qualitative description of "terrain pattern" while relief amplitudes and stream frequencies were found to be factors suitable for a quantitative expression. A "terrain unit" was descriptively a physiographic unit and was quantified by dimensions of the unit (relief amplitude, length, width, etc.). Finally, the "terrain component" was described by the lithology, soil type, and vegetation association. The quantified terrain component measured in situ identified particle size distribution, strength, permeability, mineralogy, and various dimensions of surface obstacles, vegetation, and relief.

Engineering Classification Systems

Intact Sample Classification

Classification systems based on the physical character of intact rock materials (Figure 2) overcome the problem of irrelevant geologic nomenclature based on a wide range of mineralogical compositions, textures, and weathering conditions occurring in different rock types. Often the mechanical performance of rock material is predicted more rapidly and more accurately by mechanical testing, but usually both visual observations and mechanical tests are required to provide data for design purposes. A rock classification system may be based upon inherent rock characteristics, may be formulated on the basis of the particular purpose for which the rock is to be used, or may be based on a combination of both inherent characteristics and intended usage. An intact rock classification system can form the basis of systematic analyses for the prediction of performance.

There are six characteristics important to rock engineering which should be the basis for a rock engineering classification system:

1. strength,
2. deformability or pre-failure deformation characteristics,
3. lithology,
4. gross heterogeneity or anisotropy,
5. durability or failure characteristics, and
6. rock continuity or mass partings.

These characteristics tend to overlap when used in intact sample and in-situ classification systems. An intact sample system, because of the very nature of specimen size effects, should include the following properties: strength (tensile), lithology, specimen anisotropy, and durability (Figure 3).

Tensile Strength – Since rock strength is an important property, a suitable strength index test is required. Penknife, pick, and hammer tests seldom provide objective, quantitative, or reproducible results. Although unconfined uniaxial compressive tests have been used in rock classification systems, the test requires machined specimens. Hardness tests tend to be strongly influenced by variations in testing techniques. Irregular lump tests have been used successfully by many investigators as a strength indicator. The point load

Anisotropy	Moisture Content
Lithology	Petrofabrics
Slake Durability	Porosity
Tensile Strength	Seismic Velocity
Compressive Strength	Shear
Density	Swelling
Drillability	Tangent Modulus
Dry Specific Gravity	Texture
Failure Characteristics	Toughness
Hardness	Unit Weight
Hysteresis	Weatherability

Figure 2. Summary of Typical Attributes of Intact Sample Rock Classification Systems.

strength index provides a measure of tensile strength, and empirical results show excellent correlation between this index and unconfined compression strength.

Lithology -- Traditional geologic rock names are based on such properties as texture, mineral content, structure, particle size, and cementing matrix. Although these properties provide a better indication of geologic history than of mechanical properties, a rock name may provide a "feeling" for the rock character and suggest mass effects which might be widespread among specific groups of rock.

Specimen Anisotropy -- In general, most rock is anisotropic (measured mechanical properites are a function of specimen orientation). Most elastic sedimentary rocks are slightly to strongly anisotropic in such mechanical properties as thermal conductivity, velocity of elastic waves, electrical conductivity, and fluid permeability. Permeability and the point load test has been applied successfully in the logging of cores. The point load test is used to define the "strength anisotropy index" as the ratio between the maximum and minimum point-load strength indices.

Durability -- Durability refers to the extent of alteration a rock will exhibit under different environmental conditions. Short-term weathering of rock has been measured with various degrees of success by abrasion tests, sulfate soundness tests, absorption tests, slake tests, and swelling tests.

Differentiation between soil and rock materials for classification purposes is important in terms of laboratory procedures to which the materials will be subjected. Several methods for separating compacted (soil-like) materials from cemented (rock-like) materials have been used. Probably the better methods for a measure of durability from an engineering standpoint are swell tests and(or) slake-durability tests. Plots of dry apparent specific gravity versus saturation water content have also been proposed to delineate weak rock and soil materials from "rock-like" cemented and compact rock materials. A qualitative differentiation whereby rock material is that which cannot be sampled by driving a steel sampling tube, whereas most soil material can be so sampled, is susceptible to operator bias. The use of wet-dry cyclic weathering to distinguish among transitional materials has been proposed by many investigators. Thus far, the best method of soil-rock differentiation appears to be a durability-plasticity rating (Figure 4).

In most instances, design parameters necessary for construction projects are unattainable from direct testing of intact samples; most in-situ tests are uneconomical to perform both with regard to time and expense. Rock mapping investigations to determine the behavior of rock in its natural environment, first through an analysis of the current in-situ state of the rock and

	TENSILE STRENGTH		ANISOTROPY		DURABILITY		LITHOLOGY	
CLASS NO.	WORD DESCRIPTION	POINT-LOAD INDEX[a] (MPa)	WORD DESCRIPTION	STRENGTH ANISOTROPY INDEX[b]	WORD DESCRIPTION	SLAKE-DURABILITY INDEX[c] (percent)	SYMBOL	WORD DESCRIPTION
1	Very Strong	> 10	Isotropic	1.0 - 1.2	Very Durable	> 50	SS	Sandstone
2	Strong	3 - 10	Slightly Anisotropic	1.2 - 1.5	Durable	25 - 50	SH	Shale
3	Moderately Strong	1 - 3	Moderately Anisotropic	1.5 - 5.0	Moderately Alterable	10 - 25	LS	Limestone
4	Weak	0.3 - 1	Anisotropic	5 - 20	Alterable	5 - 10		
5	Very Weak	< 0.3	Very Anisotropic	> 20	Highly Alterable	< 5		

[a]Point-Load Index = Force at Failure/Square of Distance between Loaded Points in a test method developed by Franklin (1970)

[b]Strength Anisotropy = Maximum Strength/Minimum Strength

[c]Slake-Durability Index = Percent Retained on 2-mm Screen after slaking in a test developed by Franklin and Chandra (1972)

Example: 1 - LS - 2 - 1 indicates a very strong, slightly anisotropic, very durable limestone

Figure 3. Proposed Intact Sample Classification System.

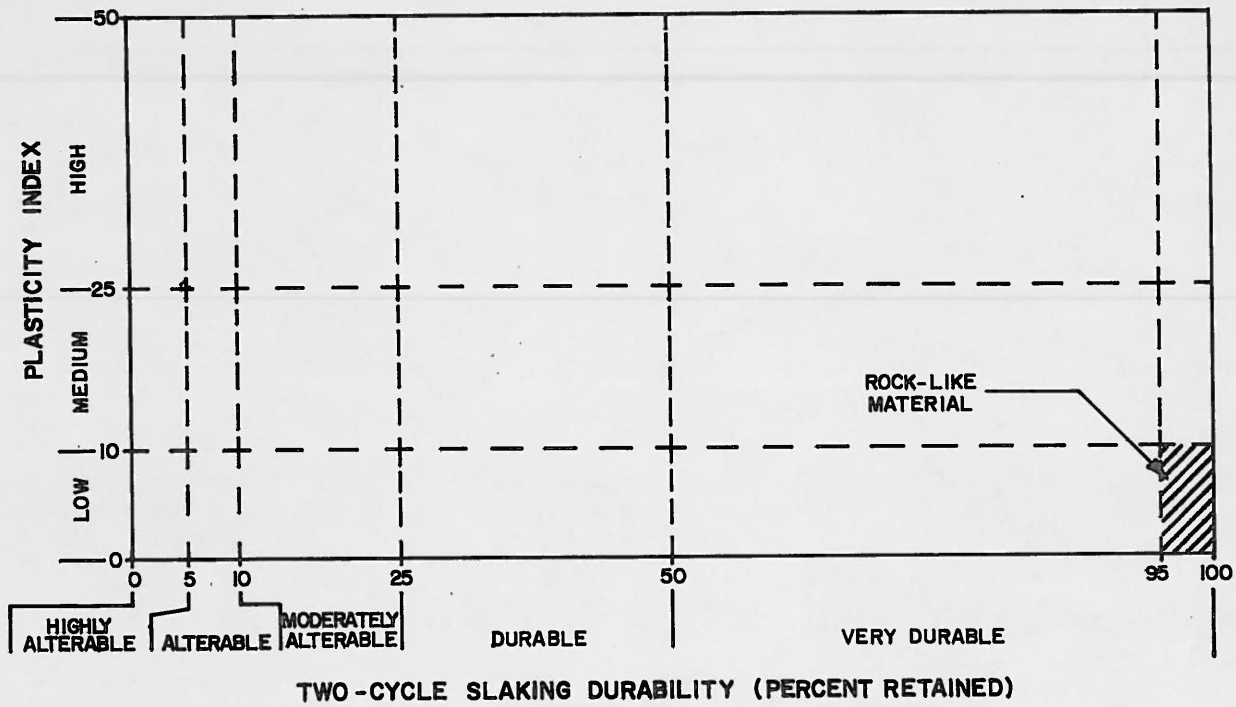

Figure 4. Durability-Plasticity Classification for Shales and Other Argillaceous Rocks (after Gamble, 1971).

second through prediction of the consequences of anthropogenic activities which may occur, require specific testing techniques (procedures): rapid sample preparation and testing, simplicity of testing, portable apparatus for some field testing to obviate deterioration of samples in transit, relevance to rock properties, relevance to engineering problems, and power of discrimination. These should be guidelines to simple, efficient relevant testing without inherent large errors of measurement.

In-Situ Classification Systems

Significant engineering properites of a rock mass can be measured directly in situ (i.e., direct deformation or shear tests, measurements of deformations resulting from environmental alterations, etc.). In most cases, the expense of these tests is prohibitive. Such circumstances warrant use of exploratory tests (for example, borehole logging tests, borehole photography, pumping tests, and geophysical tests) which can be related to engineering properites. Such correlations are the basis for an engineering classification of in-situ rock.

A brief survey of in-situ classification systems (Figure 5) revealed several interesting facts:

1. there are relatively few general in-situ classification systems;
2. in-situ systems have been, for the most part, working site evaluations either for tunneling or blasting requirements or for characterizing a particular site and rock complex;
3. major concerns in existing systems have been rock quality (bedding character, joint frequency, and weathering or alteration), lithology, deformation characteristics, and velocity ratio;
4. some systems utilize laboratory measurements such as unconfined uniaxial compression strength, static modulus, and static sonic velocity on intact specimens; and
5. in-situ tests utilized to a significant degree included seismic velocity, plate jacking, permeability, modified RQD, and borehole analysis tests.

Rock Quality	Intact Sample Tests
Bedding Character	Uniaxial Compression
Joint Frequency	Sonic
Weatherability or Alteration	Saturated Sonic
Lithology	Static Modulus
Deformation Characteristics	Point Loading
Velocity Ratio	Slake
Engineering Performance	In-Situ Tests
Slope Stability	Seismic
Powder Factor	Plate Jacking
	Permeability

Figure 5. Summary of Typical Attributes of In-Situ Rock Classification Systems.

Strength and deformation characteristis of in-situ rock are dependent upon both the physical properties of the intact rock and the number, nature, and orientation of discontinuities in the in-situ rock mass. To evaluate in-situ rock behavior, the engineer first should investigate the physical-mechanical properties of representative intact samples. Then, because the in-situ rock is discontinuous, the engineer should use reduction factors to adjust the "upper limits" defined by a statistical analog of intact samples. Both intact sample properties and discontinuities determine the engineering behavior of the rock mass with respect to strength, deformability, and permeability.

There has been, in recent years, a tendency to characterize a rock mass by means of a rock mass model and(or) a joint survey. The model may be physical, mathematical, or physio-mathematical consisting of three basic parts: consitutent rock material, joints and faults as potential planes of structural weakness, and environmental conditions before, during, and after construction. These three aspects lend themselves to intact sample classification, in-situ classification, and rock monitoring systems as part of the proposed rock evaluation schema. The joint survey is a systematic, statistical procedure by which data are collected to construct the rock mass model. While the use of such techniques as impressographs and coefficient of joint volume decrease are beyond the scope of this research, the use of a modified joint and(or) fault survey is an integral part of the rock quality description within the in-situ rock classification system (Figure 6).

PROPOSED ROCK EVALUATION SCHEMA

A viable rock evaluation program must allow practitioners and researchers to exchange information to their mutual benefit and advancement of the study of rock behavior in general. The practitioner brings performance information and experience to the exchange and receives data on which to base future design and construction procedures. The researcher is provided with a data base from which advancement in behavior prediction can be made. For planning purposes, a program must provide engineers with a sufficient basis for

1. site selection,
2. facility design,
3. construction considerations, and
4. maintenance considerations.

To be universally acceptable, a rock evaluation schema must present general information in such a way that it can be used for many specific purposes. Most importantly, the rock evaluation schema is task oriented. The task is to present a total description of rock -- intact, in situ, and the ensuing environmental effects.

The proposed rock evaluation schema consists of two segments (Figure 7). The central feature of the **acquisition segment** is the data bank. Input for the data bank will come from field and laboratory testing and case history information (i.e., previous experience, contemporary construction experience, and monitoring the performance of completed projects). The **application segment** involves the classification and use of the acquired data for specific purposes. The program is versatile in that classification and use tables for several purposes may be devised and used interchangeably without affecting the acquisition segment of the program.

Acquisition Segment

Data Bank Format

The data bank consists of a system of computer files arranged in three categories (Figure 8) which allow systematic storage and convenient retrieval of accumulated information. Category 1 contains information pertinent to the location, identification, and natural environment from which the sample or information (case histories, performance reports) is(was) taken. Category 2 contains results of visual observations, index tests, and advanced tests for both intact and in-situ rock. Category 3 provides for an indication of the existence of case history reports of previous experience, contemporary construction experience, and information to be derived from rock monitoring

	STRENGTH AND DEFORMABILITY -- ROCK QUALITY (CONTINUITY)												
	BEDDING		JOINT SPACING		JOINT FREQUENCY			GROSS HETEROGENEITY		INTACT - IN-SITU REDUCTION FACTOR[a]		LITHOLOGY	
CLASS NO.	WORD DESCRIPTION	BEDDING THICKNESS (mm)	WORD DESCRIPTION	SPACING (mm)	WORD DESCRIPTION	JOINTS PER METER	JOINT INFILTRATION MATERIAL[a]	WORD DESCRIPTION	PERMEABILITY (nm/s)	DEGREE OF CORRELATION	VELOCITY RATIO[b]	SYMBOL	WORD DESCRIPTION
1	Very Thin	< 10	Very Close	< 10	Very Low	< 0.3	Air	Very Low	< 1	Excellent	> 0.8	SS	Sandstone
2	Thin	10 - 50	Close	10 - 50	Low	0.3 - 1.0	Water	Low	1 - 10	Good	0.6 - 0.8	SH	Shale
3	Medium	50 - 300	Moderately Close	50 - 300	Medium	1 - 2	Cohesionless Soil	Medium	10 - 100	Fair	0.4 - 0.6	LS	Limestone
4	Thick	300 - 1500	Wide	300 - 1500	High	2 - 4	Inactive Clay	High	100 - 1000	Poor	0.2 - 0.4		
5	Very Thick	> 1500	Very Wide	> 1500	Very High	> 4	Active Clay	Very High	> 1000	Very Poor	< 0.2		

[a]Subject to modification with further testing
[b]Velocity Ratio = In-Situ Sonic Velocity/Intact Specimen Sonic Velocity

Figure 6. Proposed In-Situ Rock Classification System.

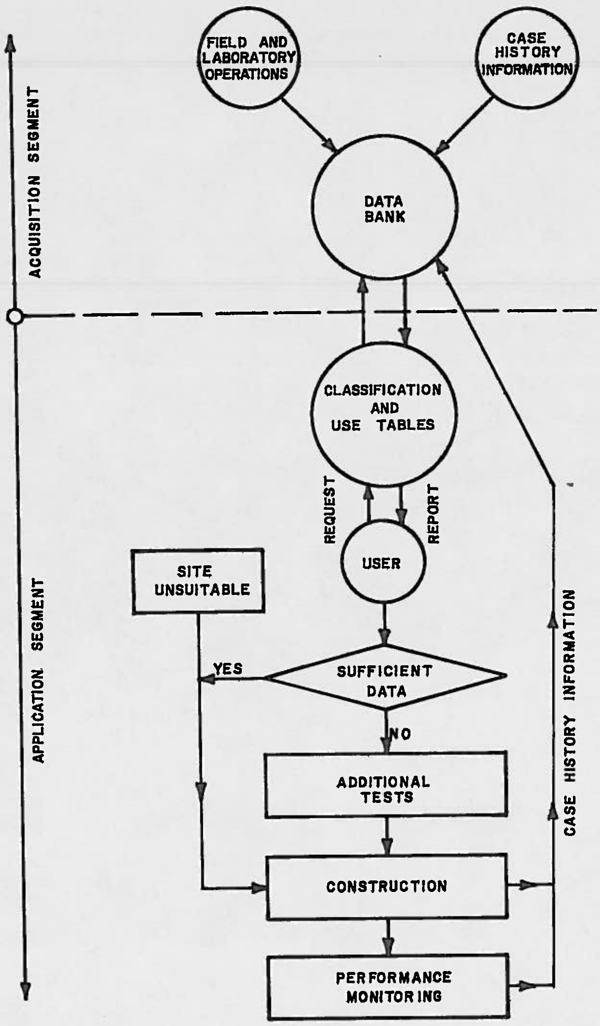

Figure 7. Schematic Diagram of the Proposed Rock Evaluation Schema.

programs.

Field and Laboratory Sampling and Testing Data

There is some overlap between field and laboratory methods used to obtain data for Categories 1 and 2 of the data bank. Information for Cateogry 1 (Figure 9) is acquired in the field and provides a description of the sampling site and of the sample type, orientation, and source.

Rock material removed from its environment should be characterized by quantitative and qualitative descriptions. Before performing index or other tests, intact specimens should be described on the basis of a visual examination to include petrographic and megascopic fabric indications of color, texture, structure, particle size, and relative content of calcium carbonate.

Ideally, samples should be tested at the site immediately after removal from the core barrel. This is not practical in all situations, however, because of insufficient qualified personnel, lack of portable equipment, or both. In such cases, samples should be preserved at their natural water content and carefully transported to the laboratory for testing. Testing should always begin with the swell test and the slake-durability test to indicate whether the material is to be treated as a soil or is to be subjected to rock classification.

Unfortunately, the variability of rock material is such that the identification and testing of intact specimens provide only a limited description and(or) indication of rock character and engineering performance. A complete rock evaluation schema requires minimal in-situ competency and rock quality investigations. In-situ rock material requires different indexing parameters and testing procedures even though the major concern, as with intact specimens, is strength, deformability, and permeability characteristics. Tests and observations as indicated in the visual and indexing sections of the intact and in-situ portions of Cateogry 2 (Figure 10) are performed to describe the rock material.

More refined laboratory (direct shear, triaxial, etc.) or large scale in-situ (pumping, plate jacking, etc.) tests may, at times, be required for detailed study of special projects. Information obtained from these tests is also stored in Category 2.

Case History Information

Certain types of empirical knowledge are not easily quantified for inclusion in a data storage system. Such data include information obtained through previous experience in an area or with a particular formation (i.e., occurrence of landslides, swell or heave tendencies, settlement, hydrologic problems, etc.), information obtained from contemporary construction procedures (i.e., success or failure of excavation methods, problems encountered, corrective measures, etc.), and information that can be gained from performance monitoring programs (i.e., weatherability rate, performance of slopes, maintenance required for various types of facilities, notations of swell, heave, and settlement, etc.). Information of this type will be handled somewhat differently. A concise version of the empirical information obtained is to be placed in a coded reference file. The code and identification of the site and(or) formation will be entered in the data bank (Category 3) (see Figure 11) so that, when a search is made, the existence of the information will be made known to the searcher. It is desirable to have or obtain samples for index testing from sites where case history information is available for correlation purposes.

Application Segment

Use of this segment of the rock evaluation program to obtain information for a specific purpose requires two

- CATEGORY 1
  - LOCATION
    - COUNTY
    - PHYSIOGRAPHIC REGION
    - USGS QUADRANGLE NUMBER
    - LONGITUDE
    - LATITUDE
  - SAMPLE IDENTIFICATION NUMBER
  - MAJOR GEOLOGICAL FORMATION
  - ROCK TYPE (GENETIC)
  - GROUND ELEVATION
  - SAMPLE ELEVATION
  - WATER TABLE ELEVATION
  - SAMPLE ORIENTATION w/r GROUND SURFACE
  - SAMPLE ORIENTATION w/r BEDDING PLANE
  - METHOD OF OBTAINING SAMPLE
  - RELEVANT COMMENTS
- CATEGORY 2
  - INTACT
    - VISUAL
      - COLOR
      - TEXTURE
      - STRUCTURE
      - GRAIN SIZE
      - CALCIUM CARBONATE
    - INDEXING
      - FREE SWELL
      - SLAKE DURABILITY
      - POINT-LOAD INDEX
      - ANISOTROPY INDEX
      - LITHOLOGY
    - PHYSIO-MECHANICAL RESULTS
      - LAB SONIC VELOCITY
      - SHORE SCLEROSCOPE
      - SCHMIDT "L" HAMMER
      - UNIAXIAL COMPRESSION
      - TANGENT MODULUS @ $\sigma_{ult(50)}$
      - NATURAL MOISTURE CONTENT
      - SATURATION WATER CONTENT
      - DRY APPARENT SPECIFIC GRAVITY
      - UNIT WEIGHT
      - APPARENT POROSITY
      - REAL POROSITY
      - VOID FILLING
      - APPARENT SPECIFIC GRAVITY
      - WATER ABSORPTION
      - VOID INDEX
      - DRY DENSITY
      - DIRECT SHEAR STRENGTH
      - TRIAXIAL COMPRESSION
      - LOS ANGELES ABRASION
      - DEVAL ABRASION
      - TRETON IMPACT
      - FRACTURE ENERGY
      - COST ANALYSIS
      - STRENGTH COEFFICIENT OF VARIATION
      - SCALE EFFECT
      - MINERALOGY
  - IN SITU
    - MASS DESCRIPTION (INDEXING)
      - ROCK QUALITY
        - BEDDING THICKNESS
        - JOINT SPACING
        - JOINT FREQUENCY
        - JOINT INFILTRATION MATERIAL
      - GROSS HETEROGENEITY
      - VELOCITY RATIO
    - ORIENTATION
    - JOINT SURVEY
    - SECONDARY INDEXING
      - CORE BORINGS
        - CORE RECOVERY
        - RQD
        - FRACTURE FREQUENCY
        - WEIGHTED LENGTH TECHNIQUE
      - SCHMIDT HAMMER TEST
    - GEOPHYSICAL SURVEYS
    - FIELD TESTS
    - PHYSIOGRAPHIC/TERRAIN CLASSIFICATION
- CATEGORY 3
  - PREVIOUS EXPERIENCE
  - CONSTRUCTION PRACTICES
  - PERFORMANCE MONITORING

Figure 8. Data Bank Attributes.

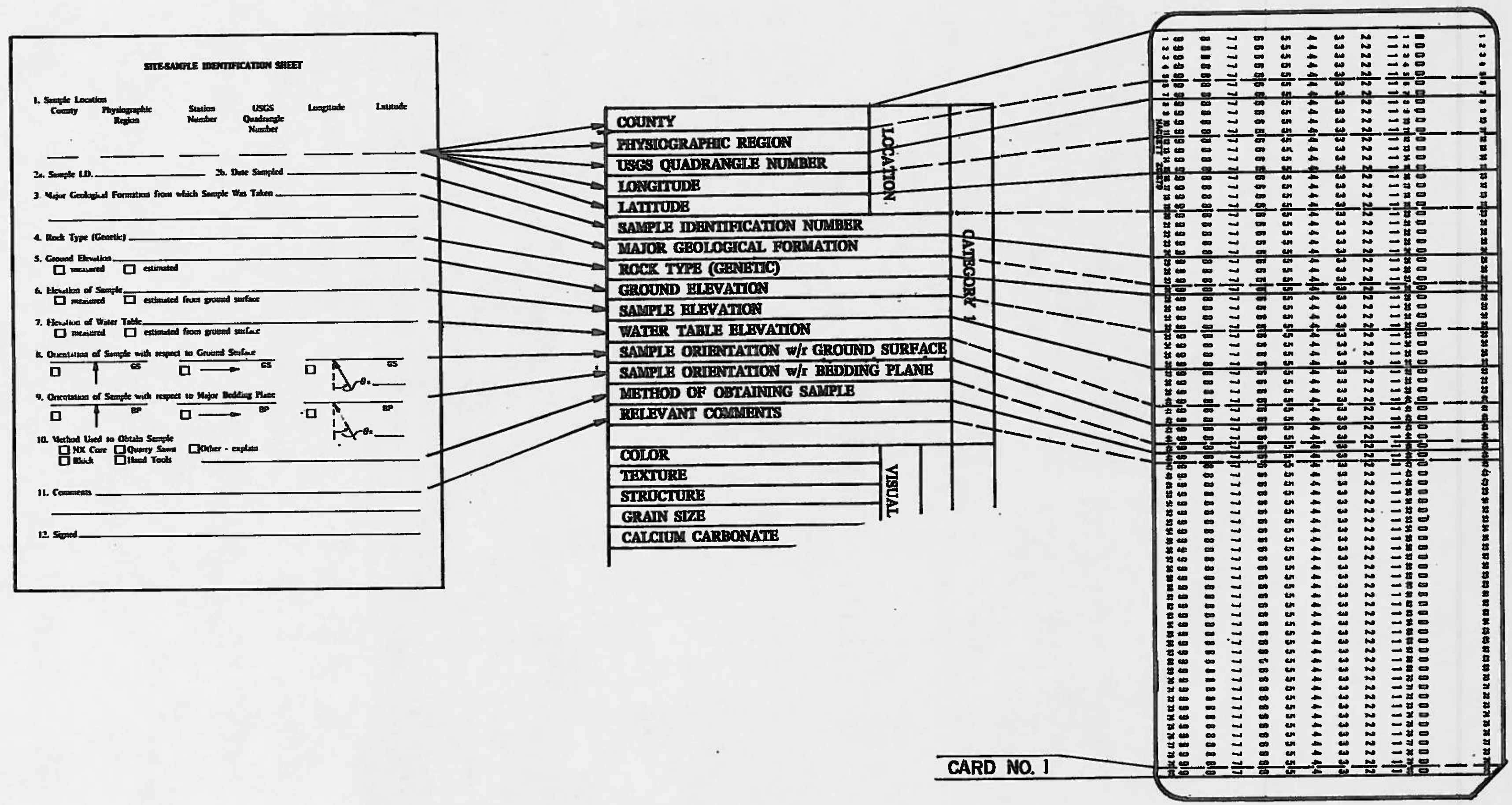

Figure 9. Category 1 (Site and Sample Description) File Subsystem.

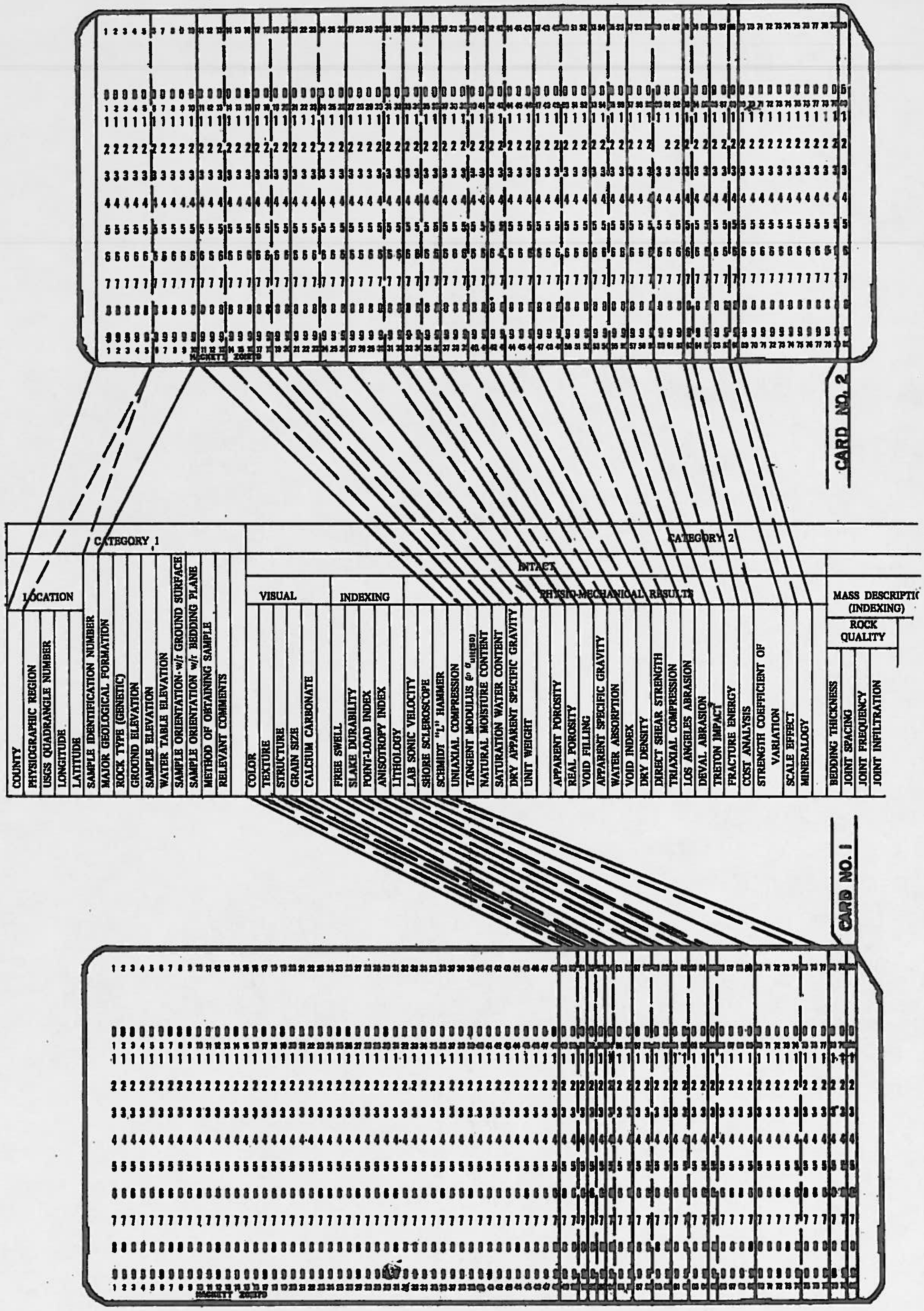

Figure 10a. Category 2 (Intact Sample Data) File Subsystem.

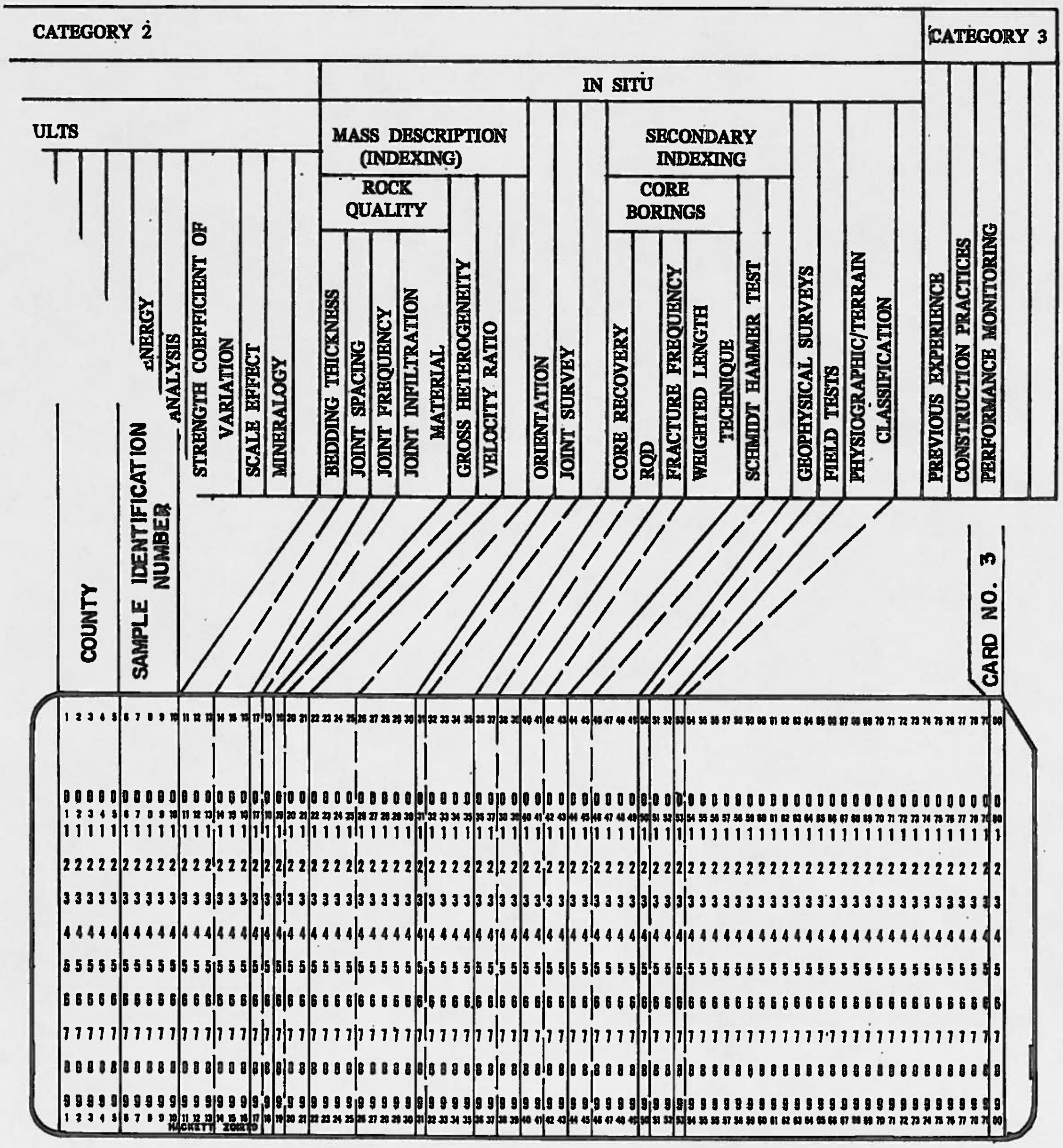

Figure 10b. Category 2 (In-Situ Data) File Subsystem.

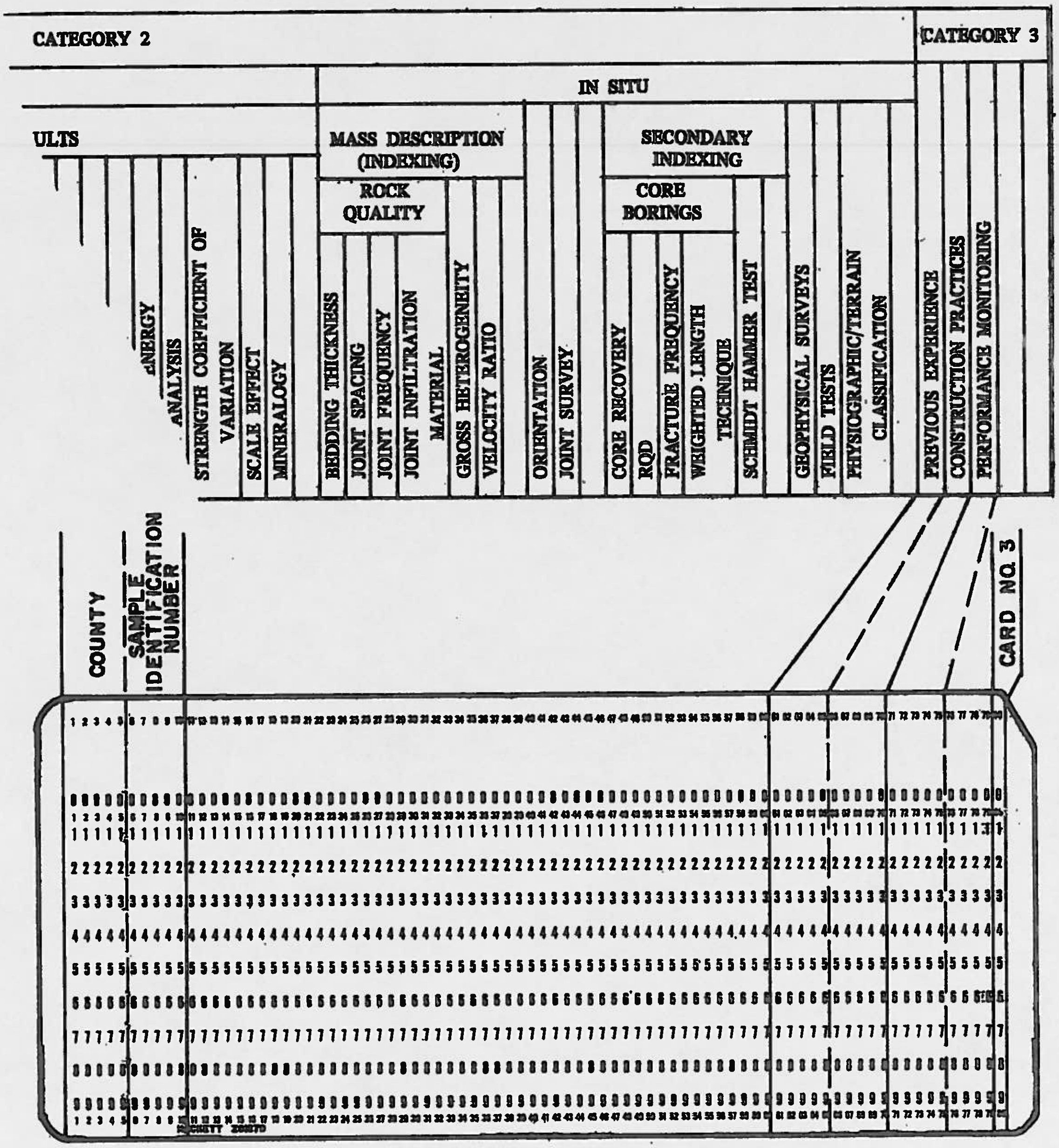

Figure 11. Category 3 (Case History Data) File Subsystem.

preliminary steps. First, the classification system must be adapted (ranges of properties for each parameter or the parameters themselves changed) depending on the intended use. Second, an use table (Figure 12) encompassing applications relevant to the intended use must be developed and appropriate ranges of the index parameters determined. An use table provides a rock model for a particular situation. For example, a specific use table would indicate the minimum values of parameters necessary to implement a design criteria while the data bank is a systematic accumulation of physico-mechanical rock characteristics which will eventually enable a total description of the rock. The program itself is very versatile due to the fact index parameters used in the acquisition segment are standardized to a great extent. Therefore, any classification system that uses these standard parameters can be used with it.

Once the classification system and use tables have been established, use of the accumulated data is quick and convenient. The data may be used to obtain statistical information of a specific geological formation and(or) to obtain specific information about a particular site. A request for data is input into the system; a detailed report of all available information is returned. Using this information in conjunction with classification and use tables, a decision is made that

1. there is sufficient information available for the particular design requirements,
2. the site or formation is not suitable for the intended purpose, or
3. the site or rock formation appears feasible but further investigations are needed to obtain design parameters.

The value of the schema depends upon the amount and quality of information which is fed into the system. Information gained during and after construction and monitoring should be fed back into the data bank for retention and future reference. In this way, the program becomes self perpetuating.

SUMMARY

The scope of rock engineering encompasses at least three major concepts: engineeing interpretation of geological considerations, determination of engineering properties of in-situ rock masses for analysis, and application of these analyses to designs related to rock masses. To facilitate communication among various professions associated with rock engineering, a rock evaluation schema has been proposed in which engineering data are inserted into a classification system wherein the data are evaluated in terms of specific needs. Input data are derived by means of completed and future testing, project construction experience, and monitoring designed to quantify environmental effects on the performance of engineered facilities. To aid in this endeavor, both an intact rock sample classification system and an in-situ rock mass classification system have been designed. In addition, the usage table concept in which ranges of acceptible engineering parameters are developed for use in designs using rock as an engineering construction material has been suggested.

CLASSIFICATION ELEMENT	RANGE OF ACCEPTABLE VALUES						
	AGGREGATE	ROCKFILL	ROADWAY SURFACE	STABLE SLOPES	OTHER USES		
Point-Load Index							
Lithology							
Strength Anisotropy Index							
Slake-Durability Index							

Figure 12. Typical Format of Use Table.

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D. J. Hagerty, Ed.

FOUNDATIONS ON OR IN ROCK

by

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INTRODUCTION

Although the selection of foundations on or in rock to support loads without excessive settlement or lateral movement is important, the engineer will usually find that his major design decisions will be in selecting the bearing stratum for the foundation and deciding on statements for the specifications relative to construction. The engineer's choice as to the type of foundation, its size and the selection of procedures for construction control and quality assurance will usually be a compromise between the overall cost, schedule and the probability of success.

During design, the engineer has to consider the possibility of unexpected changes that may occur during construction -- changes not apparent from the subsurface data. The probability of encountering such unexpected conditions is far greater for foundations on or in rock than for foundations bearing on soils, because certain characteristics of rock masses such as faults, joint patterns, bedding planes, voids, and thin seams of soft rock often go undetected.

DESIGN

Foundation designs are seldom alike since no two sites have identical subsurface conditions and categorizing foundations on or in rock is difficult. Nevertheless, dividing the subject into two broad classes,

A. Excavated Foundations placed on or in rock at a "shallow" depth, and
B. Drilled Foundations bypassing soil and set at "depth" on or in rock,

appears reasonable. This may be somewhat of an oversimplification of the variety of foundation types that the engineer encounters, but will help in this presentation which is directed to the first category -- excavated shallow foundations.

The designer's first decision is to choose the type of foundation that will best fit the subsurface conditions. Usually, the choice is obvious; however, in some cases a comparative economic analysis of two or more foundation types may be required. Factors that the engineer should consider in deciding on the type of foundation are at least the following:

1. the depth below the usable portion of the building to the top of rock;
2. the condition of the rock at or near the top of rock;
3. the existence of soft layers, open joints, faults, voids or mined-out areas, fractures, etc.;
4. the magnitude and direction of the loads imposed on the foundation;
5. the depth to the water table;
6. the effect of the proposed foundation on surrounding structures;
7. the relative difficulty of constructing and/or inspecting each type of foundation; and
8. the likely construction procedures to be used and their influence on the predicted performance of the foundations.

The basic approach to the design of spread footings bearing on or in rock is not unlike that of a spread footing bearing on soil, for the same basic expression for combined direct load and bending is used to obtain a first estimate of the compressive stress distribution over the base of the foundation. The engineer may use a beam-on-elastic foundation-type analysis, but such sophistication is usually not warranted. In any event, the designer will often find that the foundation size will be dictated by the allowable bearing capacity imposed by the building code of the particular city or region. Values of presumptive bearing capacity allowed by various building codes vary widely for apparently similar rock, e.g., neglecting California, where seismic conditions enter, the allowable values for sound sedimentary, horizontally-bedded rock vary as shown in Table I from a low of 15 tons per square foot in Philadelphia to a high of 100 tons per square foot in Chicago. While differences are expected because of the local geologic conditions and rock characteristics such as fracturing, solutioning, etc., such differences must relate to a reluctance in adopting new rock mechanics technology. The major difficulty in assigning a safe bearing capacity to rock involves an evaluation of the influence of the non-homogeneity and geologic defects on the behavior of rock under load.

The properties of in situ rock may bear little resemblance to the properties determined from laboratory tests on small-diameter intact rock cores. Strength determined from axial compression tests on

TABLE I
PRESUMPTIVE BEARING VALUES FOR ROCK*
(TONS PER SQUARE FOOT)

CODE	YEAR	Massive Crystalline Bedrock	Sound Foliated Rock	Sound Sedimentary Rock	Soft and Broken Rock Excluding Shale	Shales Soft	Broken Shales
Baltimore	1962	100	35	-	10	-	To be fixed by Commissioner
BOCA**	1970	100	40	25	10	4	1.5
Boston	1970	100	50	10	10	-	To be fixed by Building Official
Chicago (1)	1970	100	100	100	-	-	-
Cleveland	Code 1951 Suppl 1969	-	-	25	-	-	-
Dallas	1968	Max. Value = 20% of Ultimate Crushing Strength					
Detroit	1956	100	100	100	12	12	-
Indiana	1967	Not More Than 20% of the Ultimate Crushing Strength					
Kansas City	Code 1961 Rev. 1969	Max. Value = 20% of Ultimate Crushing Strength					
Los Angeles	1970	10	4	3	1	1	1
New York City	1970	60	60	60	8	-	-
Ohio	1970	100	40	15	10	4	-
Philadelphia	1969	50	20	15	10-15	8	-
Pittsburgh	Code 1959 Ord. 1969	25	25	25	8	8	-
Richmond	1968	100	40	25	10	4	1.5
St. Louis	Code 1960 Ord. 1970	100	40	25	10	1.5	1.5
San Francisco	1969	3-5	3-5	3-5	-	-	-
Uniform Building Code	1970	Max. Value = 20% of Ultimate Crushing Strength					

* The values in this table should not be used without first checking with the particular code for changes.

** Building Officials Conference of America, Inc.

(1) Allows increase of 20% of bearing capacity for each foot of embedment (beyond initial required depth), but should not exceed twice the given values.

core samples is not representative of the intact rock. The strength needs to be reduced to account for joints, fractures and discontinuities, all of which influence the strength and deformability of the in situ rock mass. If rock cores are tested in unconfined compression, usually one-fifth or one-eighth of the test value is used as the allowable bearing stress. Field load tests may be used to obtain an improved reduction factor, but such tests are seldom practical and reflect only the conditions at the locations of the test.

Reduction factors are assessed by inspection of the rock after excavation, and on certain projects appropriate design changes may be made during construction. Generally, the engineer has to rely on the core samples and the corresponding core recovery and RQD values as indicators of the rock condition which, when coupled with appropriate and adequate geologic investigations, provide sound, reliable data for design.

Basically, design of foundations on rock is highly qualitative, and foundations are sized using presumptive values as specified by local building codes or by the engineer's experience and judgment. For massive hard rock, the allowable bearing capacity will be limited by the strength of the concrete.

For shallow spread footings, the support offered by bond on the vertical surface of the foundation is usually neglected. However, the Chicago building code and the recommended code of the British Institution of Civil Engineers allow an increase in bearing capacity of twenty percent for each foot of embedment, with a limit equal to twice the specified presumptive bearing capacity. This increase accounts in part for the adhesion between the surface area of the footing and the surrounding rock, and in part reflects the supposed improvement in rock quality with depth. Although the bond component of total load resistance of a spread footing in rock is small and is neglected, it is usually a major factor in the design of drilled piers socketed into rock. Insofar as increasing rock quality with depth is concerned, this may be valid for limited cases such as homogeneous, massive rock, but should be used generally with caution since the bearing quality of the rock may be much more influenced in depth by geologic factors involving changes in rock type, tectonic movements, differential weathering and man-made disturbances such as deep mining.

In most cases, after the designer has selected the depth and size of the foundation to satisfy bearing capacity, it is assumed that movement of the foundation under the imposed load will be small and, therefore, can be neglected. However, in certain cases, the foundation movements under load, even though small, are significant and the engineer may be asked to estimate the vertical and/or lateral movement of the footing under various loading conditions. The settlement or lateral movement of the foundation can be estimated from a knowledge of the modulus of the rock mass and the loads imposed by the foundation. A finite element analysis, with the aid of electronic computers, provides an expedient means for a solution. Programs to treat non-homogeneous and layered systems are available. Of course, the movements computed by such models can be no more accurate than the rock parameters used in the analysis.

The moduli may be determined from laboratory tests on rock cores. However, the results of tests on unfractured specimens are not representative of the properties of the entire rock mass influenced by the foundation. If the project is of sufficient importance to justify cost, geophysical tests may be conducted to determine the properites of the rock. Cross-hole seismic tests and seismic refraction surveys are used to measure the in situ compression and shear wave velocities of the rock mass. From these data, the modulus of elasticity can be computed and used in the finite element analysis.

The results of the geophysical tests and the subsequent finite element analyses of foundation movement may suggest that the foundation will behave adequately under load. However, if the analyses indicate excessive vertical or lateral movements of the foundation or if the quality of the rock is otherwise questionable at the proposed depth of the footing, the designer must usually choose one of the three alternates:

1. stay with a spread footing, but place the foundation on a deeper, more competent rock stratum,
2. change from an excavated to a drilled foundation to reach a more competent bearing stratum, or
3. leave the foundation at the proposed level but improve the bearing quality of the rock.

All of these alternates will no doubt cost more than the originally proposed foundation. The choice is simply one of performance and economics. If the bearing quality of the rock is poor due to the existence of extensive fracturing and if this condition does not improve with depth, the third alternate of rock improvement may be used. In this case, the bearing quality of the rock may be improved and movement reduced by grouting or by precompressing the rock mass within the zone of influence of the foundation by the installation of rock bolts or tension ties.

The designer must not only choose the type, size and depth of the footing or pier, but he will also find it necessary in many cases to specify the method of construction so as to achieve the design objective. For example, if the foundation is to be subjected to large lateral thrust loads, the designer may specify the use of presplit blasting to form the sides of the excavation. If properly carried out, presplitting the perimeter surface of the foundation excavation will preserve the lateral bearing capacity of the rock surrounding the pier or footing. There are also benefits to the contractor and the owner in reducing the time and cost of making the foundation excavations. A carefully planned presplit blasting program will minimize the amount of hand-scaling required on the sides of the excavation and the volume of rock overbreak that would require backfilling with concrete. Presplitting, which has been common for years in the construction of steep rock-cut slopes, is also becoming popular among contractors and engineers to form the sides of foundation excavations in rock.

SPECIAL PROBLEMS AND TECHNIQUES

There are as many special problems as there are rock conditions. Broadly, they may be categorized as follows:

1. Highly jointed, fractured or faulted rocks. The foundation loads or lateral thrusts may tend to close the joints or cause shearing along the faults or laminations, leading to excessive vertical or lateral movements. If the problem cannot be corrected by placing the foundation deeper or by using deep, drilled piers in lieu of spread footings, the problem may be corrected by grouting the joints to prevent their closings or by precompressing the rock mass with rock bolts or tension ties.
2. Potential or subsided rock due to caving of deep mines or solution caverns. Typical solutions are (a) grout the cracks and voids of the rock to increase its bearing capacity, (b) drill through the caved rock to the base of the mine or cavern and install caissons, (c) if the mine is shallow, excavate the caved rock and replace with a compacted fill or place the footings directly at the base of the mine.
3. Cavernous or pinacle limestone. There are several solutions depending on the thickness of soil cover. (a) Place the building on a structural mat foundation to span over soft areas, (b) excavate the soil and rock pinacles and backfill with a structural fill; place building on spread footings; (c) drill caissons through the pinacles or voids to reach competent bearing strata.
4. Sensitive rocks, e.g., claystones, indurated clays, clay-shales. If the engineer cannot economically avoid placing a spread footing on sensitive rocks, he must protect the rock from exposure to weathering and from remolding under the weight of heavy equipment during construction. Methods to protect the rock are (a) do not excavate the last foot or two of rock until immediately prior to placing steel and concrete, (b) place a "mud mat" of lean concrete over the foundation area immediately after excavating.
5. Alternating hard and soft rock strata. The borings should be drilled sufficiently deep to recognize that the condition exists. If the softer rocks are not sensitive, the foundation may be placed either on the soft or the hard rock strata (but not both). The design may be dictated by the compressibility of the softer rocks. If the bedding planes are steeply inclined, stability of the entire structure or portions of it may be a problem. If the softer strata are too weak to support the load, drilled piers extending through all soft strata to a competent bearing stratum may be a solution.
6. Thin soft seams or slickensided rock not observed in the subsurface investigation or highly weathered seams that may lose shear strength as a result of the new construction. Usually this condition is not a problem for foundations under axial loads. However, for lateral loads, the lack of or loss of shear strength may result in excessive lateral displacement or complete failure of the structure. The problem may be solved by placing the foundation below the weak plane or by installing tension ties to resist the entire lateral load placed on the foundation.
7. Expansive rocks. Certain types of rocks -- usually, but not always, shales -- contain pyrite and marcasite which are potentially expansive sulfide minerals. In most cases, this is a problem for lightly loaded footings and floor slabs. Uplift forces of two to five tons per square foot have been measured during crystal growth. Generally, if the potentially expansive sulfide minerals are below the water table, expansion will not occur due to lack of oxygen. Methods of controlling the problem are: (a) overexcavate and backfill with non-expansive material; (b) underexcavate the expansive area and seal for construction period. When exposed, the expansive material should be promptly concreted to prevent oxidation; (c) avoid cutting trenches in expansive rock; (d) rock-bolt into sound rock below the expansive

layers to resist the uplift force generated by the active sulfide minerals.

CONSTRUCTION

While choosing a foundation, the engineer should always keep in mind the method by which the foundation is to be installed. The designer often specifies the method and sequence of construction. Typically, specifications give provisions for dewatering of open excavations, presplitting, blasting, ground conditioning, etc. In the process of writing the specifications, the designer should consider both the short-term and the long-term effects of the construction on the surrounding area. In many cases, the construction procedures are more critical to the success of a foundation than the design concept itself. Generally, the deeper the foundation, the greater the number of problems that may arise in the field. This is especially true of deep, drilled piers where experience has shown that the designer and the contractor can get into more trouble, often difficult to remedy, from unanticipated conditions during construction.

In order to reduce to a minimum the possibility of encountering unforeseen problems in the field and eliminate "extras" from the contract, the engineer should:

1. make certain that the subsurface investigation is sufficiently complete so that the effects of potential construction problems can be carefully evaluated before choosing the type and depth of the foundation and that the appropriate requirements are included in the specifications,
2. urge the selection of a contractor with proven capabilities in the type and method of construction anticipated, and
3. insist that adequate field engineering supervision be provided and paid for by the owner or architect-engineer rather than by the contractor.

The designer should objectively evaluate potential construction problems and include appropriate conditions in the specifications so that bids from contractors adequately reflect the actual site conditions and construction difficulties. Unanticipated troubles during construction can lead to significant cost overruns and delays of the work not only to the detriment of the contractor but also to the engineer and, more importantly, the owner. Therefore, the designer should make available to the prospective contractors the engineer's report and all available subsurface data.

The types of problems that may be encountered in construction of foundations on and in rock and the ways of anticipating, avoiding or solving them are many and varied.

With excavated foundations bearing on rock, some blasting will be required to reach the proposed depth of the foundation. If the work is performed in a densely populated area, the blasting vibrations around the site may be critical. Although in most cases the contractor would normally be considered the responsible party as to any nuisance or damage by his blasting operation, the engineer would be prudent to include provisions in the specifications to limit the blasting vibrations produced at various distances from each excavation. Since people are much more sensitive to vibrations than buildings, it may be necessary in congested areas to limit vibrations to levels far below that which would cause damage to the nearby structures. Whereas, a vibration which produces a peak ground motion of two inches per second is normally considered the upper limit of safety to protect buildings against damage, a ground motion velocity of only one-tenth this value may produce numerous complaints from nearby residents, and could cause a job shutdown. The engineer should consider these problems since the owner, whom he represents, is interested in finishing the project on time. Fortunately, a great deal of research has been done on the subject of blast vibrations. It has been found that the peak velocity of ground motion, which correlates most directly with building damage and human response, is related to the so-called "scaled distance" between the blast and the point in question. The scaled distance is the actual distance in feet divided by the square root of the weight of explosives detonated per delay interval. Since explosive manufacturers produce a variety of delay blasting caps, an experienced blaster can limit the vibrations at a given distance to any predetermined level by "spreading out" his shot over a number of delay intervals. The engineer may include in his specifications a provision whereby the contractor must maintain a scaled distance greater than a certain value unless the blasting vibrations are monitored so as to show that smaller scaled distances (i.e., larger charges) are safe. Pennsylvania law limits scaled distance to no less than 50 unless the vibrations are monitored.

The engineer may also specify that the contractor use the presplit blasting technique to form neat excavations in the rock and preserve lateral bearing capacity. If the ultimate condition of the remaining in situ rock around the perimeter surface of the excavation is truly critical to the performance of the footing, the engineer should include provisions in the construction contract which will permit the contractor to experiment with presplit hole spacing and charge distribution until the desired result is achieved. This kind of flexibility can be best assured if presplitting is called out as a separate bid item in the contract and is paid for on a lineal foot basis. Thus, the engineer can maintain the cooperation of the contractor, avoid arguments and "extras" while still maintaining the flexibility to change the method of presplitting to suit the results or varying rock conditions that may be encountered.

Another major problem with deep excavated foundations in rock, expecially in urban areas, is the damage to surrounding structures that may be caused by lowering of the water table during construction. Thus, the engineer should have an accurate knowledge of the location of the water table and make provisions in his design and construction specifications. For example, the engineer may find it necessary to seal the sides of the excavation to permit dewatering of the interior while not lowering the water table outside of the foundation area. In some cases, the entire construction site may have to be sealed off from surrounding urban areas in order to make extensive excavations below the water table.

Certain types of sedimentary rock such as claystone and clay-shales are sensitive to weathering or disturbance in the presence of water. Such rock types may have adequate bearing capacity in their undisturbed condition but may turn to mud if the contractor is not adequately forewarned of their sensitivity. One method of preventing disturbance to sensitive rocks of this type is to prohibit the contractor from running heavy equipment on the base of the excavation or performing any work during wet weather. He may also be required to leave one to two feet of material in place above the proposed depth until immediately prior to pouring of the concrete. If considerable time is required to place reinforcing steel at the base or if the footing cannot be otherwise protected against the elements, the engineer may specify the pouring of a thin layer of lean concrete over the entire base of the excavation to protect the rock from weathering while the reinforcing steel and concrete are being placed.

The prudent designer will include a provision in the specifications to check the rock conditions below the proposed base of the footings by drilling pilot holes or core borings in some or even all of the foundations. If unexpected soft zones are encountered, the foundation can be deepened or other provisions can be made to account for the change in the anticipated conditions. A rule of thumb for determining the necessary depth of pilot holes or core borings in the base of spread-footing excavations is to drill the holes to a depth equal to the minimum width of the foundation.

The designer, familiar with the design intent and performance of the foundations, should be at the site to inspect the condition of the rock at the base and on the sides of the excavation to assure that actual conditions are as assumed in the design. The designer's field inspector should be a qualified geologist or engineer experienced in engineering geology and the influence of construction procedures so that he may recognize the significance of fracture patterns, faults, bedding planes, etc., as they relate to the performance of the foundation.

CONCLUSIONS

A thorough exploration program should be conducted to establish the depth and quality of the supporting rock and the difficulties likely to be encountered during construction. Once the feasibility and economy are established, the actual design of a foundation involves selecting its size and the stratum on which it will rest. Generally, the designer will specify the construction method involving blasting, dewatering and treatment of the bearing surfaces prior to placing of the steel and concrete. In most cases, the design will involve mostly qualitative considerations of presumptive bearing values, the designer's previous experience in the area and his judgment of the load-carrying capacity of the bearing material. In applying his judgment, the designer should consider at least the following:

1. the strength and deformability of the rock on which the foundation will be founded, including possible changes in the quality of the rock caused by the construction;
2. the loading from the structure to the foundation, including dead load, sustained live load and peak live loads;
3. the allowable total and differential foundation settlements as provided by the structural engineer;
4. the properties of the available concrete and steel materials which will form the foundation; and
5. the effect of construction on existing adjacent structures and the possible effects of future construction in adjacent areas on the installed foundations.

Since the design is partially qualitative and involves considerable judgment, a field representative of the designer should be at the construction site to insure that the rock conditions are consistent with the criteria used by the designer.

ROCK FOUNDATIONS

A Panel Discussion

Comments by
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Most of my own and our company's experience with foundations on rock can be divided into two general categories insofar as rock type is concerned, shale and limestone. Certainly almost all of our problems are involved with one of these two kinds of rock. Since shale is the bedrock of the Greater Cincinnati Area, we have, of course, more frequent experience with foundations on shale than anything else. The Cincinnati Formation is of Ordovician age, a rather thinly bedded, soft to tough shale, with thin layers of fossiliferous limestone. There can be considerable variation in the total amount and thickness of the limestone layers and in the mineralogical and engineering properties of the shale layers. Also, we often work with hard carbonaceous shale such as the Ohio Black Shale of Devonian age and soft clay shales of Pennsylvanian age.

In the early days of our practice, we limited foundation pressures on shale to presumptive values published in building codes, of which Ohio's was typical, which allowed 4 tons per sq ft for soft shale and 15 tons per sq ft for hard shale. At the beginning of the construction of high rise buildings in the late 50's, a typical foundation consisted of drilled piers carried through glacially deposited over-burden soil and sometimes a substantial depth of soft weathered shale to essentially gray shale and belled out to limit bearing capacity to 15 tons per sq ft. Belling in the Cincinnati Formation, with its frequent thin layers of limestone, had to be done by hand. Observation of this slow and increasingly costly method over a period of years caused us to believe that there had to be a better way. This served as an impetus for laboratory and field testing programs by a number of people which resulted in about a three-fold increase in design bearing capacity over figures that had been previously used for comparable material.

In exploration of shale, our emphasis is on a determination of the average strength and elastic modulus and on the presence of layers that are substantially softer than average. In most of the area in which we worked, the shales are essentially flat lying and conditions are fairly uniform laterally. At hillside sites, however, very closely spaced borings are required. Our normal practice is to auger through soft and weathered shales and sample with the standard spoon, most typically at 2.5-ft intervals, to refusal and then core with an NXM-series core barrel.

Our evaluation of bearing capacity is based on laboratory compression tests on suitable samples, penetration resistance and moisture content of soft and weathered shales, and occasionally, if the project warrants, field load tests. The most important part of design is often an economic study of all feasible types; frequently spread footings or belled piers near the surface of the shale for low to moderate loads and straight shaft piers drilled well into the shale for higher loads, occasionally considering both end bearing and side friction. High capacity bearing piles are also frequently driven into the shale in the valley areas where shale is covered with a thick alluvial deposit.

For light to moderate structures, inspection of foundations on shale is certainly no more critical than for foundations on soil. The main thing is to look for any obvious difference from what the designer assumed and any softening due to exposure. Most heavy loads on shale above the water table are on straight shaft drilled piers drilled well into the shale. We normally inspect these piers by drilling 1.5 to 2" test holes, 3 to 5 ft below the bottom, and probing the side for soft layers with a right-angled probe rod.

Whereas foundations in shale are relatively routine and probably less hazardous than, say, foundations in glacially deposited soils, I consider that foundations on most of the limestone deposits we deal with to be, along with landslides, the most difficult assignments that we receive. Whereas in shales our emphasis is on determining the strength of the shale, we could care less about the properties of the limestone itself. We are concerned about how much limestone there is and how irregular the surface is (it is always more irregular than the worst you can imagine) and what is between the individual pieces and layers of limestone. It is usually soft clay, sometimes air or water.

The basic exploration tool is still the core borings with emphasis on complete recording of all anomalies in the drilling process. On major projects, drill as many as you can get the client to buy. We often supplement these with auger soundings to refusal to aid in developing the rock surface. Occasionally we will make test excavations with a backhoe or a pier drilling machine. We learn what we can from examination of exposed cuts and experience on other projects.

Testing and theoretical soil mechanics is of little help for foundations on limestone. Experience and luck are probably the most valuable attributes. The most important thing on a limestone job is that the geotechnical engineer work closely with the design-build team entirely through the design and construction phase. You cannot write a report and then forget about it.

After all available information concerning the geology of the site is developed, the most important task is to make a realistic estimate of the probable cost of all feasible foundation schemes. We have used straight shaft drilled piers a great deal. We often use an allowable bearing capacity of 40 tons per sq ft for piers taken to reasonably sound limestone, although we have gone as high as 100 tons per sq ft on the building quality stone of the Bloomington-Bedford, Indian area. Any excavation of limestone in a drilled pier must, of course, be by hand using jackhammers or by drilling and shooting. This is extremely slow and the unit prices are astronomical. It is very difficult to estimate the quantity of rock excavation for a pier foundation on limestone. We have been involved on jobs where the cost has doubled the estimate and I have heard of projects where the difference has been greater than this. I therefore feel that any investigation of limestone sites should include a full examination of all possible shallow foundations, either in clay overburden or on the uneven highly weathered limestone surface using low to moderate pressures with close control and dental work where needed; i.e., digging out the clay by hand and filling it with lean concrete. We are even using piles more often in solution limestone using moderate loads, reinforced tips and close field control.

In closing I might comment that it seems that when I went to school some years ago the impression I received is that if you can take your foundation to rock that is the thing you should do. After some years in this business I am more inclined to feel that if you can avoid going to rock you should.

Comments by
Harry Thomas
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Louisville District

I will discuss the foundations for two navigation structures on the Ohio River downstream from Louisville. I will place emphasis on unique design and construction features. It has been the practice of the Louisville District to found as much of the navigation structure on bedrock as feasible. We lean toward dry, open-cut excavation so the foundation can be examined and features of the rock mapped and analyzed prior to placing concrete. I believe the wisdom of this practice has been borne out in recent years on projects in which design modifications have been made due to (1) earthquake considerations, especially since the San Fernando earthquake in 1971, and (2) changes in the consideration of rock strength, placing more emphasis on residual rather than peak strength in certain situations.

The two projects I am going to discuss will be Cannelton and Newburgh Dams. Unusual design features at the Cannelton Dam were the need for a very deep foundation and the exceptionally high cofferdam construction. The cofferdam dam was designed by the contractor who elected to construct the dam in two stages, beginning on the Kentucky shore. An open-cut excavation was made with two rings of cells around the construction area. The outer rings consisted of 60-foot circular cells with protection grade approximately 140 feet above bedrock. Sheet piling varied in length from 80 to 140 feet. This cofferdam encompassed approximately 17 acres. The inner subcofferdam consisted of 60-foot diameter cells.

Dewatering was accomplished by means of 27 deep wells around the outer cofferdam and 17 deep wells around the subcofferdam. During excavation of the foundation to bedrock at Pier 8, six upstream cells of the subcofferdam moved into the excavated area. Internally braced cofferdams were then driven for each individual pier.

The dam piers were founded on hard, well-cemented sandstone and crystalline limestone of the Mississippian System. An interesting situation noted in the deep portion of the bedrock valley after the foundation was exposed was a lack of open joints and, in some cases, a lack of any jointing. Normal and thrust faults were encountered in some of the piers. The normal faults had a maximum displacement of 8 feet and appeared to be late Paleozoic in age; much of the faulting in the area occurred during the late Paleozoic. However, the compression ridges and thrust faults, which had displacements measured in inches, may be more recent and may be due to compressional forces caused by abutments moving into the eroded Ohio River valley. In the deep portions of the valley, there is compressional faulting and a lack of other jointing.

In contrast to the lack of joints in the deepest portion of the valley was a pronounced relief-type jointing found in piers closest to the abutment. It was obvious that these foundations needed to be strengthened. It was decided to found the piers on a well-cemented sandstone 70 feet deep. This was accomplished by means of 30-inch diameter drilled-in caissons battered upstream and downstream. Eighty-eight drilled-in caissons were installed, 55 of which were battered downstream. Some modulus of elasticity tests were performed using rock jacks in the field, and laboratory testing was accomplished; our designers told us that the caissons needed to be battered due to the moment in the upper portion of the foundation.

At Newburgh, the dewatering system consisted of submersible pumps along the interior perimeter of the main cofferdam and a well-point system around the construction extremities. Pier 10 at Newburgh had a base width of 65 feet and was 128 feet long. The foundation was of silty shale, Pennsylvanian in age. No blasting was required; the shale was excavated by ripping.

During the final cleanup in Pier 10, a small one-foot displacement fault was found in a downstream corner. The areal extent of the disturbed material was investigated by a series of 36-inch diameter inspection holes drilled in the foundation with an auger. After a close examination of the foundation made possible by the 36-inch holes, additional testing was done on the weaker zones. By drilling overlapping 36-inch holes, one-foot cube samples were obtained for testing. As a result of the additional testing, and new earthquake design criteria, it was decided that the foundation should be strengthened by installing rock tendons into a more competent sandy shale at depth. The tendons were installed after the pier had received a layer of approximately 50 feet of concrete. The tendons consisted of fifty-two 1/2-inch diameter steel strands with a bond length of 25 feet. In addition to the bond, four 21-inch diameter bells were installed.

The most efficient means of drilling the 9-inch diameter tendon holes was with a diamond hole hammer. We could drill through 50 feet of concrete and over one hundred of feet of rock in approximately 45 minutes. The 52 strands were stressed simultaneously with a force of 1500 kips per tendon with a 1000-ton hydraulic jack.

Comments by
Milton M. Greenbaum
Greenbaum and Associates
Louisville, Kentucky

Two materials that we encounter locally, in Louisville, are also present in the areas that Bob Lennertz works, in Cincinnati. One is a shale, usually expansive. As soon as water touches it, it not only expands, it totally loses its strength. It becomes a relatively useless material if it is wet. Therefore, we generally are presented with the problem of keeping this material in a dry condition. In its dry condition, it is quite competent.

This shale presents a real problem for people who have to do the soils investigations. Drilling in this material, to say the least, is extremely difficult. We attempt to avoid it whenever we can. We are presently involved in a project in which a coal reserve must be evaluated. Unfortunately, above this coal reserve is a blue-gray slaking shale which "drives us up the wall." Special drilling tools are used for this project. The only way to work with this material, to build on it, is to keep it dry, frequently putting moisture membranes directly on top of it before pouring concrete so as to prevent the shale from drawing moisture out of the concrete and to prevent migration of moisture from above. As often as possible, we try to cast the concrete fully against the sides of an excavation. Generally, we emplant the foundation in the material rather than on it so as to seal out any moisture -- keeping moisture from getting under the foundation.

Generally speaking, when we talk about shales in the Louisville area, we are talking about one competent rock and one rock of very limited competency. In many areas, we have New Albany shale -- dense, quite massive, an excellent foundation material. We have a project presently underway in which the arrangement of these two rocks have managed to complicate what would normally be a very simple problem. This is a bridge for the L & N Railroad -- basically an engine underpass in a high embankment. The embankment rests on a clay which is the product of the decomposition of shales and limestones. The material is loose and wet. The owners cannot tolerate any settlement. "Any settlement" is a kind of nebulous thing. Nevertheless, they want settlement held to an absolute minimum. The engine underpass is to be built at a site with 45 feet of overburden under 25 feet of embankment. Obviously, 25 feet of embankment would cause considerable settlement even in a fairly well consolidated soil. Here we have a very poorly consolidated soil and we have estimated settlement on the order of 5 inches.

We came up with three solutions for minimizing the settlement. One was to do away with the embankment -- put the structure on trusses. This was totally undesirable. The second solution was also undesirable.

The third solution was to carry the load to the rock. In carrying the loads to the rock, two systems, piles or drilled piers, could be used. Open excavation is out of the question; it is 45 feet down to the rock. The piles develop extreme negative skin friction due to the consolidation taking place in the soil. We attempted various methods of separating the piles from the soils to avoid the negative skin friction. They were all prohibitively expensive, particularly in this day of rising steel prices and steel shortages. So the solution, then, was to use drilled foundations. Ordinary drilled foundations to the rock was our first thought. We would bell out the drilled piers to provide a substantial foundation at the rock level. We put in a test hole. Ordinarily, these drilled piers in the New Albany shale will stay open without casing. However, in this particular case, directly above the massive shale was a layer of the weak, relatively broken shale; the water table was substantially above this level. As soon as the drilled pier reached this level, we had a blow-in. Belling at the bottom of such a pier became impossible. We had to reject this scheme. The next thought was one Bob Lennertz mentioned -- go down into the sound shale and then bell out. We can put a casing down to the sound shale, seal out the water, and then bell out after penetrating several feet into the sound shale. We discussed this with various contractors. It is very expensive. The hand labor and the OSHA regulations concerning a man working at the bottom of a 45-foot hole just made this situation prohibitive. We attempted a solution, also mentioned earlier today, involving putting the pier into the rock and using friction as well as end-bearing in the rock. This was the most workable and acceptable method and this is the one that was actually utilized. The piers were drilled some 3 feet into the sound rock after going through about a foot or two of relatively soft rock. Based on the strength of the concrete, rather than the strength of the shale, using approximately 35 percent of the concrete bearing and using roughly 5 percent of the concrete compressive strength as its bond value, we anticipated supporting this pier with as little as 3 feet of penetration into the shale.

We had a situation in which, even though we used acceptable procedures, the solution still bothers me. Here we have taken out several feet of softer rock and three feet of sound rock which has a compressive strength of 10,000 to 15,000 pounds per square inch. We then replaced it with a concrete which has a strength of approximately 4,000 pounds per square inch, and we are utilizing a bond between two materials. If you take the unit which is below the hard rock surface, and compare it to the prior conditions, you have below the pier a material of a lower strength than you had initially. Actually, I have no question that the design is stable. But somehow the theory does not quite ring true. I think in dealing with foundations on rock, even more so than dealing with foundations on soils, you are dealing with so much art and so little real valid science that at times one gets really concerned. Yet we have to make "hard rock" decisions.

Jim Coulson, TVA, at the 1974 Seminar

D. Ross-Brown, Dames & Moore, making his presentation at the 1974 Seminar

DESIGN OF OPEN EXCAVATIONS IN ROCK

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In the design of rock slopes, the pertinent geotechnical principles apply to any type of slope, whether it be highway slopes, dam abutments, or the slopes of open-pit mines. Perhaps open-pit mines represent an extreme not seen in highway cuts, both in regard to their size and also because of the fact that a different philosophy is involved in the design of these slopes. For instance, failures can often be tolerated in a mining situation where they could not be in a road cut situation. We also have to bear in mind the other factors, apart from the geotechnical ones, that come into play; these are the economic, operational, and political factors. I shall be talking mainly about the design of open-pit slopes, and I shall be bringing in some of the other factors because, unless these geotechnical aspects are seen in their proper context, the design cannot be very meaningful.

Perhaps the best known open pit in the world is Bingham Canyon in Utah. It measures approximately 2 miles by 1 1/2 miles across its rim and is 2500 feet deep. Benches in the slope are 40 or 50 feet high. Everyday, about 120 tons of ore and 350,000 tons of waste are drilled, blasted, loaded into trucks and trains, and transported out of that pit. The slopes are 26 1/2° and the operators are continually confronted with slope failures at this copper mine.

At another mine, also nominally in the same category (copper), slopes stand at almost 70°, but they are quite stable. This is proof that, even when there are nominally the same kinds of rock types, slope angles may vary significantly.

Perhaps we should consider not only the feasibility of designing slopes but also the feasibility of designing a mine. The decision to go ahead on a mine will often depend upon what slope angles can be used. These factors in the feasibility study of the mine are basically the geological ones; the most important involves the location and grade of the ore body. Various other factors include the metallurgical, processing, legal, mine engineering, and economic factors.

In passing, perhaps we should note that the different shapes of ore bodies have different effects on the slope design. For instance, for ore bodies of certain shape, the slope angles are not too critical because the slope angle is governed by the shape of the ore body. For another shape, of course, the slopes will be very critical. The steeper we can have the slopes, the lower the stripping ratio, the lower the weight that has to be excavated. And in some cases, such as a disseminated ore body, the cutoff is determined by a sort of statistical analysis of the grades because there is no sharp geological cutoff.

It is instructive just to look at some sort of simple excavation economics because it puts geotechnical studies into their right prospective. Many pits such as the one at Bingham Canyon can be represented by a truncated cone and defined by its bottom diameter, its depth, and slope angle. Using a pit 1000 feet deep with a 300-foot bottom diameter, I did a little study of the volume of rock that has to be excavated for different slope angles from 20° to 90°. An additional scale can be provided knowing the density of the rock and the unit cost of excavating a ton of rock. And it is about the same numerical values in terms of millions of dollars. For an 80° slope on this mine, the total excavation costs will be in the order of $10 million. If the slope angle had been reduced to 20°, then the cost soars to something over $250 million dollars. So you can see the kind of economics game we are playing. Such a relationship also shows something else. For instance, in the case of an existing mine where the slopes are 30°, this kind of relationship can indicate the savings involved in steepening that slope by 10°. In this case, the cost will be reduced from $100 million to about $55 million. The savings of $45 million are the basis on which decisions can be made about further geotechnical investigations, making studies maybe of groundwater control, and use of controlled blasting techniques.

There are, however, various other operational factors. The object of the mining operation is not to design stable, beautiful slopes; the object of the operation is to maintain a constant flow of ore to the processing plant, with regard to tonnage and grade, and also to minimize the operating costs by getting the maximum utilization of the equipment over the whole life of the mine. Thus, slope stability factors often are of secondary importance. For example, blasts are very heavy in most open-pit mines because this reduces production costs. But it has a very detrimental affect on slope stability.

The operating slope angles are very much dependent upon the equipment being used whereas the final slopes are determined by rock mechanics factors. For a pit at a very early stage in life, we see shallow operating slopes at the bottom of the pit which are dictated by the particular type of equipment combinations being used and steeper operating slopes at the side which are determined by rock mechanics considerations. Looking at the same pit 20 years later, one may see what the final shape is.

Even though the pit is being designed with as steep a slope as possible, the effect of having to put in a haul road has reduced the effective angle of the slopes from 55° to 45°. It almost doubles the amount of material to be excavated. But, of course, the haul road is a very essential part of the operation. If the road is jeopardized by falling rock or slides, then access to and from the pit is cut off and production ceases. In a mine where this has happened, the haul road originally spiraled down the mine from top to bottom; failures developed on one side of the mine. Failure is also threatening the existing processing plant. The haul road has had to be relocated on the other side of the mine at a considerable cost.

Failures are an important part of designing these pits. Their importance varies, depending upon the size of the pit. For instance, in a small pit it is necessary to design the complete pit from the very beginning and mine down to the final slopes in the very beginning. Such a design is often based on inadequate information. If slope failure occurs, it may necessitate leaving a berm and consequently the ore in that portion is lost. This may upset the whole economics of the operation and make the profit negligible, or even negative. In a large open pit of the Bingham Canyon size, there is more flexibility because if a mistake is made in the design of the initial pit and a failure occurs, then we can go back at a later stage and mine out a larger pit. When we come to the final pit, maybe 20 or 30 years later, we have a good idea of how those slopes perform, and we can design with much greater efficiency.

A survey of 144 open-pit mines showed the effect of failures on production, and that is the criterion with which we are concerned. Safety, of course, is another criterion but statistics show very few deaths or injuries due to slope failure in mining. These statistics show that in 28 percent of all mines, failures had a very serious effect on production. In another 32 percent, they had a minor effect on production.

Before looking into some of the failure modes that occur in open pits and how we can design against them, care must be taken to distinguish between the natural weathering and other processes that occur in open-pit mines. For instance, in ravelling, the rock gradually flakes off and fills out the benches, thus reducing their effectiveness in catching and retaining material. In another kind of ravelling, decrepitation starts at the bottom of a slope and gradually progresses upwater channels. These are not so serious as failures but nevertheless they do have a nuisance value and we should be well aware of the possibilities of trouble.

We should now consider some of the failure modes that can occur and of which we will have to be aware. For instance, Mode 1 is simple sliding on a through-plane discontinuity. In Mode 2, we have sliding on one discontinuity with separation along another steeper one. Mode 3 illustrates failure on two sets of discontinuity, where maybe the continuity of any one set is not sufficient or the orientations are not sufficient to get either a Mode 1 or Mode 2 failure. Then Mode 4 illustrates the case of three intersecting discontinuities which form a driving (active) wedge and a passive wedge.

There are also three-dimensional aspects of failure modes. For instance, if we look at the two- and three-dimensional aspects of simple sliding, we see we have a sort of block slide, or we can have sliding caused by intersecting discontinuities resulting in a wedge. Two-dimensional modes can be analyzed; even three-dimensional modes have been analyzed in a similar manner using the concept of critical dip.

Other failure modes we have to be aware of are the toppling mode, occurring with nearly vertical joints sets. We have possibilities of slab slidings, particularly on the footwalls of coal workings. We have various circular failure modes in open pits. This may be the result of overburden material within the pit limits, or it may be that there are rock blocks which are very small in comparison to the total height of the slope. Perhaps the only way we can analyze the problem is by means of the simplified Bishop method. In such a case, movement may not be confined to just one shear surface. It may involve a zone in which there is a certain amount of rotation and displacement of rocks. We may also have a sort of a generalized failure surface of a similar nature but where much of the failure surface may follow some defect in the rock such as bedding. Then we have the translational slides, also very common in coal mines, especially where these can occur at very low angles, when the joints or bedding is filled with gouge, or maybe in coal seams with high water pressure. Slides on inclinations of less than 4° have occurred in this mode.

A complicated situation often found in open-pit mines involves a pattern of joints which is overlaid by another system of jointing or faulting plus the interaction of underground workings. The question is how do you analyze that kind of situation because there is no particular failure plane that can be defined. There is rotation and movements of the blocks. Maybe the blocks stabilize themselves again or maybe they stabilize themselves for awhile and then they continue failing. And we have various methods of analyzing all these different modes of failure.

As illustrations of some of these failure modes in open-pit mines, you see a failure on a simple discontinuity, a large fault something on the order of 200 feet high. In another open-pit mine, the joints (there are five very distinct joints sets) give rise to various kinds of block failures and potential or actual wedge failures. A wedge failure on a much larger scale, the famous Liberty Pit slide, involved some 20 million tons of rock and was caused by two intersecting faults. An example of pure toppling, not from an open pit, is the Little Colorado River gorge. The slopes are something on the order of 600 or 700 feet high. Toppling there is due just to tension failure. Toppling usually is of a more complicated nature and involves some sliding.

This leads us to a well known failure, the Fort Payne Springs slide. A literature search indicates that many people have tried to analyze this failure. And I am sure we have not heard the last of it. The point is, when we are dealing with very high slopes, on the order of thousands of feet, it is very difficult to analyze the situation because the geology is very complicated. We have many different domains: structural domains with different joint patterns, overlaid with different faulting; patterns of soil; and many different rock types. They are not easy to analyze.

To discuss how we go about designing these mines, I would like to summarize some of the non-geological factors I have hinted about. First, of course, is the economic fact of why we design these mines with steep slopes -- to improve the economics. There are safety considerations; there are environment considerations; there is politics. I am afraid we often dare not mention this word in technical meetings, but politics do often decide the design of many things and certainly of open-pit mines. And then there are statutory requirements.

Geotechnical problems of open-pit mines can be subdivided into (1) the design of the pit slopes, (2) the excavation of pit, (3) the monitoring of movements, and (4) how to deal with failures once they develop. There are a host of other allied problems into which I will not go. But you can see the interaction of all these factors produces quite a complicated flow chart for open-pit slope design. Another fact we must bear in mind is that we may not be able to come up with a very good estimate given the constraints of inadequate data, inadequate time to do a proper analysis, etc. If we are honest with ourselves, we must admit that a pessimistic estimate could produce a very low slope angle and an optimistic estimate could be high. After the mine is opened and we get some exposures and a few benches, we can then improve our estimates. And then after a few years of observing these slopes and excavating them, our estimate gets better and better. Hopefully, by the time the mine is nearing completion, our estimate is fairly accurate.

I am concentrating the discussion on designing the slopes, and I will say a little about excavation techniques, but I will have to ignore everything else. We are interested in designing operating slopes, final slopes, and benches. It is convenient to consider these in two stages. One is the initial design, which is done at the feasibility study stage of designing the pit. Such a design is completed basically to make sure we have angles which are economical to the operation. The second stage is the final design which is based on better information. The initial design is approached with constraints of time, money, and manpower.

The approach is very similar to that used by geotechnical engineers working on other projects, as already noted today. There is the office study which involves looking at geological maps and aerial photographs, talking to people, etc. A study of the hydrology and seismology of the area is required. A field study is based on natural slopes and a certain amount of structural mapping, index testing, and a limited amount of shear testing. At this preliminary stage, a simple analysis is undertaken because the data are not good enough to justify more sophisticated methods. At this stage, it is usual to use design charts for plane circular failure, for instance, and to use graphical analysis techniques. There is some preliminary idea of slope angles we can use with these techniques. This is then incorporated into the pit design and becomes the first iteration in the design process. Looking at adjacent pits can often reveal very useful information in analyzing these failures or in developing simple observation curves such as slope angle versus slope height curves.

The final design is really a continuing process. It involves more structural mapping, particularly of faults, more hydrology studies, more shear testing, and the observation of modes of failure that have occurred including back-analyzing these, and rates of failure, and bench behavior. It gives us a chance to experiment with blasting and water control. Eventually we end up with a more realistic and sophisticated method of analysis and a better design.

Once the pit is opened, then we are faced with the tremendous problem of dividing the region up into provinces and domains for detailed analysis. For a pit I recently visited, they had in their museum over 300 different rock types which had come from the mine and they had over 60 miles of benches. So you can see that collecting geological data and utilizing it to make some kind of realistic analysis is a very serious problem. Nevertheless, this is what is involved; and the geologists have to map the gross features, the faults, the main lithological horizons, and divide the area into these different structural domains. The gross mapping, perhaps, will give us an idea of the major failures that we might expect on faults and shear planes. These features have to be mapped, not only in relation to their orientation but also their location in space. When talking about a structural domain, the assumption is that the jointing is uniform within the domain; then we have to use statistical analysis because it is not possible to map all the joints. This mapping of domains is usually done on coding forms in a special way, and these data can then be analyzed by a computer. We can use computer programs for clustering the data and calculating the mean joint sets and giving us statistics about those joint sets. On occasions, we can use photogrammetry to help supply this information, by taking stereo-pairs from two stations set up in the mine. The area can be seen in three dimensions when viewed through the proper instruments and can be analyzed in the same way as aerial photographs. However we get this information, of course, we have to be assured that we understand the relationships of all these various structural defects, particularly in relation to any kind of faulting that may be present. Something like 80 percent of our effort probably goes into that geological investigation; it is by far the most important factor in designing open pits.

But we also need information on shear strength, and it is very convenient to use something like a portable shear box. We obtain the samples by picking them out of the face, molding them in concrete using special molds, and loading in a very simple machine. Such testing gives us an idea of the shear strength along various kinds of discontinuities. We must bear in mind that there may be many different rock types, many different joints, and many different types of stressings. So at this stage, it only makes sense to do these simple kinds of tests and to do as many of them as you can. At a later stage, you may have defined a very special problem and that may then justify much more sophisticated tests, such as the 100-pound shear box. We might be talking in the order of maybe \$500 to \$1000 or more for doing the more sophisticated tests whereas the smaller ones are more on the order of \$50.

Other factors we have to take into account include the angle of roughness as defined by Frank Patton. Unfortunately, in open-pit design, this often becomes very important because we are trying to design to the limit. If we base our design just on the shear strength of the joint and its orientation, we might end up with a very uneconomical design. So we are looking to try and get any additional shear strength from roughness or lack of continuity. These roughness studies, of course, are used to help us predict the shear strength characteristics of large planes in the field, large joints in the field, from relatively small scale shear tests. There are various techniques of measuring roughness including the geologic compass method. It is a very crude method whereby a pointer follows the rough slope, which is transferred onto a roll of paper. There are much more sophisticated ways of doing it. Then we have to get an idea of the continuity. We end up, then, with frequency curves of the extent or continuity of the joints and also the spacing of the joints.

It is very important to understand these statistics because, unless we approach the analysis statistically, we are not going to get a very good answer. We are not going to get such a good answer anyway, but we will get a much better answer by analyzing statistically. All the parameters have to be expressed in terms of a mean and a standard deviation or some other kind of distribution which reflects inadequate knowledge of the rock properties and their variation from one location to the next.

Once we have this kind of information for each structural domain, we can analyze it using particularly the stereographic analysis approach, very useful for open-pit design. Sometimes we can go to much more sophisticated methods of analysis, such as a program for the three-dimensional wedge case. Such a program takes into account variations of all of the important parameters such as different angles of the top slope, different water pressure distributions, different positions of the tension crack, and the addition of rock bolt forces. Such a program will perform automatically analyses for four different water pressure assumptions.

Factors of safety from such analyses, of course, are not very meaningful when we have an inadequate knowledge of some of the parameters. We might then go into a sensitivity analysis. This is very useful if you still believe in the factor of safety approach. If you prefer to deal with the probability of failure, then we can analyze it using the same program but a different option. The properties are specified in terms of a mean and a standard deviation. The analysis is reiterated many times, using an overlay technique to sample values for the parameters so that they fit the specified distribution. As a result, we get a large number of factors of safety (defined as the difference between the driving and resisting forces) which can be plotted as histograms or frequency curves. We get various statistics about these distributions and can determine the probability of failure.

Very often in open pits, we are forced to resort to soil mechanic techniques because we cannot define a failure any other way or because the slopes are very high. The problem may be analyzed by many different techniques to find the one with the lowest factor of safety. In other situations, it may be useful to use the finite element method of analysis, especially when there are thick soft layers. We are experimenting with such a program to allow progressive cracking of the elements. This can be very useful for studying the way the failure develops. Of course, it does not assume any failure plane; it actually allows it to happen. There are also other programs, such as the finite-difference modes which also do not make any assumptions about the failure plane. They allow it to develop naturally. The only trouble with these kinds of programs is that they are very expensive to run. We are talking in terms of maybe a minimum of $400 a run, maybe running into thousands of dollars. But it may be that you only need to do one analysis because it does not presuppose the failure plane but allows failure to develop naturally.

One method, which I think has tremendous potential and which has been developed by a colleague of mine at the University of Minnesota under a contract with the US Army, involves a main computer linked with an interactive graphics console. This enables us to draw onto the screen in just a few minutes any kind of geometry we like. After allowing for consolidation, we can allow blocks to form and to rotate or translate. We can see failure developing. This gives one a chance to "play" with different kinds of geological configurations and different material properties and see what happens. This is very instructive.

Having done all these designs, we have to provide the mining company with something they can use. This may be stated in terms of slope design charts where we plot slope angle against slope height for different factors of safety and different water pressure assumptions. Or we can recommend slope design in terms of the possibilities of failure using probability techniques. These can be summarized in tables or in plan showing what the overall slopes should be for the bench design and what berm should be.

Before I finish, I want to make a few remarks about excavation techniques. Such things as rock type, jointing, or faulting are not within our control. These are decided by nature. Also, unlike highway cuts, we have to design open-pit mine slopes where the ore body has been located by nature. These are factors we have no control over whatsoever, but we do have control over the way we excavate the slopes, particularly the use of controlled blasting techniques, the way we control water, the way we clean the faces, and rock improvement techniques, if any. You all appreciate the obvious importance of controlling water. The basic techniques usually involve cutting off the flow before it gets into the pit area or using one of a variety of drainage techniques once the water gets into the pit. These can be very involved designs in themselves. The penalties for not taking into account water in the design could be a stoppage of operation and continually failing slopes.

I am not going to say very much about controlled blasting techniques. This was covered earlier by Mr. Brandenburg. There are two techniques used in mining -- pre-splitting and cushion blasting. Cushion blasting involves firing holes successfully more gently until we get back to the final line. Pre-splitting involves shooting that final line first. The spacing and charges are similar. Cushion blasting is a little less heavy on explosives, as shown in an open-pit mine where they have used very good blasting techniques. I think the lesson to be learned here is that the slopes of nature may stand very steeply, even though there is adverse jointing and faulting. If we can duplicate the same kind of process, then we can also produce slopes that will stand much steeper than if we just go in and blast. Inclined drilling is not new; such drilling has been used on a mine for the last 25 years to drill holes as steep as 70°.

An example of a good cleaning technique is found in a mine from the Mesabi Range. The rocks are not particularly competent; they are sedimentary. The joint block sizes are in order of inches, the dip is about 4°, and the slopes are standing on 70° over 400 feet. This is true of maybe a dozen or more mines. The reason slopes are standing very steeply is because of very good digging techniques. Generally, they step down the final blast holes about 18 feet from the final line. The shovel operator digs back to the excavation line, being very careful not to undercut it. By gentle carving action in this relatively incompetent material, they are able to achieve these very steep slope angles.

On occasions, it may be necessary to use various rock improvement techniques such as cable anchors and wire mesh. This, perhaps, is the great challange of the future for mines. Can we design basically unstable slopes and hold them up using these reinforcement techniques long enough that we can mine out the ore and get out without causing any safety problems, thus making a much bigger profit than we would have done otherwise? This is a good final question.

ROCK EXCAVATION

A Panel Discussion

The Producers' Viewpoint
by
J. A. Waddell
Martin Marietta Aggregates
Louisville, Kentucky

I will address the issue of some of the problems that face the producers in the aggregate industry. In the last decade, we have had a terrific problem overcoming the bad image of producers of aggregate. Our public image was terrible. We have made attempts to rectify that with the reclaiming of land where we have excavated a mineral, making parks and civic clubs in some of these reclaimed lands. Our company donated a large acreage in Illinois for a high school.

One of the problems we face as an aggregate producer, and I do not want to get into too many technical problems, is that land must be made available for the extraction of basic minerals. I think over the last decade or so a lot of expensive real estate has been put over the top of basic minerals that are essential for the economy of this country. Another problem that confronts our industry is the situation where multiple operations cross state lines. In these situations, there is a difference in the standards of various states for gradation of both fine and coarse aggregates. It is a nightmare to operate in three states with three different sets of specifications for base course materials as well as concrete bituminous mix aggregates.

Being in this industry a number of years, one feels as if perhaps we have been singled out by a number of regulatory agencies. We surely are plagued with them. I want to list just a few of the regulatory agencies the aggregate producers are now faced with on a day to day basis, agencies other than those with which any other business would be involved.

1. Federal Metallic and Non-metallic Safety Regulations of the Department of Interior agency known as MESA, the Mining Enforcement and Safety Administration.
2. Public Law 91-452, Organized Crime Control Act of 1970, administered by the Department of Treasury, Internal Revenue Service, Alcohol, Tobacco and Firearms Divisions, for licensing the storage of explosives (plus the state fire marshall in this state) as well as local ordinances covering explosives.
3. The United States Coast Guard, marine safety inspections.
4. Regulation of oil spills and leaks to prevent discharge into navigable streams, US Environmental Protection Agency.
5. The Corps of Engineers, construction in the floodplain.
6. The Environmental Protection Agency, again, on the national pollution and discharge elimination system.
7. The state Air Pollution Control Board, on air quality.
8. The state Board of Health, Division of Water Pollution Control.
9. The Department of Natural Resources, construction in floodplains.
10. The Division of Gas and Oil for exploration and drilling.

It is obvious that regulation compliance is quite important to us. Another environmental problem of our industry is noise, which we have been able to contain in most cases. We are able to control the dust problem, with wet systems or dry bag-type houses and dust collection. We have been able to control dust very well.

Blasting of the rock itself is a problem. We have very little control over the noise of blasting. I have been asked, "What is the most economical way to excavate crushed rock?" I have replied, in a few simple words, "Shoot hell out of it". However, therein lies the problem of our industry, the explosive itself. In most cases, where we are involved in crushing rock near metropolitan areas, which is where the business is concentrated, we have been limited in the amount of explosives we can use and the limit continues to be reduced. Subsequently, the cost of reduction of rock, because of these limitations of use of explosives, continues to rise. In some locations, we are now down to two holes per shot with less than 200 pounds of explosives in those holes. You can see the cost of aggregate has to be astronomical in such locations. The complaints we receive on a daily basis in areas where we shoot is fantastic, even though we use seismographs and monitor each one of our shots. We are positive we are not damaging anyone's property around the perimeter of our operation. Nevertheless, we have failed to convey to these people that we are not damaging their property. It is difficult to convince someone that you have not damaged their property by vibrations even though we can prove it with instruments. Nevertheless, you are still a public nuisance to them. They want the quarries and the sand operations, but they want them located in some other place.

Our industry, speaking of the National Crushed Stone industry, has done a great deal in the last 5 years to improve our public image. As individual operators, we have spent a great deal of money in the last 5 years to improve our image. The film I have here today is a part of what we have been doing at Martin Marietta;

a number of other stone producers throughout the country are also undertaking similar activities.

(Editors' Note: At this point in his presentation, Mr. Waddell showed a motion picture on crushed stone excavation and processing.)

Consideration of Effects
on Surroundings
by

D. J. Hagerty
University of Louisville
Louisville, Kentucky

I want to concentrate on the environmental effects of blasting. One of the biggest problems is a psychological problem. You can go out with a seismograph, and you can monitor blasts, and you can come back and determine that the blasts produced a maximum particle velocity of 1/10 of an inch per second. You can say that that is not nearly enough to cause any damage. But you have to be very careful making sweeping generalizations because at times they can fall back on you.

I made a generalization once, and it turned out that the person making the claim operated a facility that displayed glass and it was not glass panes; it was glass bric-a-brac -- very small and delicate and all kinds of fanciful forms. The blast had not done any structual damage to the building, but there was one hell of a crash when it went off. It was a crash and a lot of little tinkles later. It so happened that the framing of this particular structure acted to amplify vibrations. This was the exception that proved the rule.

It is possible to measure vibrations, it is possible to design blasting shots to limit the rate at which energy is imparted into the surrounding rock mass or earth mass and then, consequently, into the structures. I hope we do not have to get down to the two-hole concept that is being forced on some of the Martin Marietta projects. It is possible to measure these vibrations, and we can talk about seismographs. You saw some of the vibration equipment earlier this morning.

The most important problem, though, is people. I have just finished a project where I completed 150 house inspections associated with a blasting project. This is a rather small project in terms of cost, but it was not small to the people of this small town putting in sewers. Some of the houses were as much as 150 years old with plaster walls and deteriorating frameworks -- that is the most charitable way I can describe the conditions of these houses. Now, in this situation, I was called upon to supervise the pre-blast survey. I believe the people were honest, and we had honest complaints. The reason I believe the people were honest is that this small town is the home of a theological seminary and 95 percent of the people were either ministers or ministerial students. What about these claims? What about these people? Many times, people, in all honesty, make claims that they feel are justified, and they point to cracks in the wall and they point to various other conditions and say "...they were not there before." You have to have a photograph that you can show that "Yes, it was." Or you have to be able to go up and say "Well, look at the paint down in the crack there. Is that really a crack that just formed; when did you paint last?" "Three years ago, and we painted the house green." "But that is blue paint in the crack." "Oh, that was five years ago when we painted blue."

You have a people problem. This is an engineering problem because we are going from a technical problem to dealing with people. The thing that complicates the situation is that the level of vibration that is necessary for a person to perceive it and to be upset by it is about four orders of magnitude smaller than that which will cause any kind of disturbance in a structure. These are some of the "environmental" effects that can be experienced in removing rock. We have seen it from the producers' viewpoint.

I had the occasion one time to be called into court in Jefferson County and asked to testify about the noise and air pollution associated with a crushed stone plant. I was asked by a lawyer if I had ever seen any crushed stone plant that did not make any noise and did not produce a lot of dust. I said I had seen three. I saw a closely watched TVA installation. I saw another installation that was just being built, and I saw the installation under question. Immediately the lawyer faced about and said, "You mean this plant?" I said, "Yes sir." "You didn't see any dust or hear any noise?" "No sir." I thought I really had him. As he was walking back to his table, he finally had an inspiration and whipped around and said, "Was the plant operating?" "No sir," was my reply. These are the only instances that I can give you where I have seen crushed stone plants that really did not make any noise.

We did measure noise levels at this particular plant. We had a strip chart recorder. We found a steady level of noise, but every so often we got a peak in this strip chart recording, this record of the noise. We finally figured out what the peak was -- there was a bluejay in the tree nearby that occasionally let out a squawk. The noise level from that bird was louder than that from the plant. But the people who lived in the subdivision before the plant was built could hear and became very upset about the noise associated with this plant that began operation at 6:30 a.m. They could barely perceive this noise, but they honestly and sincerely became upset about it. These are some of the problems where you get into some real engineering, and we are talking about the heart of engineering in this particular area.

Geologic and Support Considerations for Rock Excavations

by

J. Mahar
University of Illinois
Urbana, Illinois

The design of rock excavations for civil engineering works primarily focuses on the permanent support that is used to maintain these openings and on any structures which go inside the openings. These are important considerations in the aspects of design. Most of the problems, however, occur during construction, and most of the additional costs as well. Thus, an important aspect of design and construction is the temporary support system. Excavation is equally important as is the control of blasting. I will, however, in this presentation, spend most of my time discussing temporary support systems – some of the more commonly used systems and their proper method of placement – which are extremely important in maintaining the stability of openings.

To accomplish this, we will take a close look at four case histories in which, because of construction procedures used, difficulties developed. We will look at both open and underground rock excavation. All case histories which I will discuss are located in Piedmont rock along the east coast of the United States in Washington and New York City. I will briefly describe the engineering geology as it affected these excavations, as it controlled the behavior of the rock. We will start with an open excavation in both soil and rock, then discuss a stability problem that developed in a rectangular-shaped opening, discuss the failure and collapse of an arch section of a tunnel that was supported with steel struts, and then, in the fourth case, we will talk about the near-collapse of a tunnel section – a section that was prevented from collapsing because of some very quick thinking on the part of the contractor and Dr. Deere.

You should note the similarities between the cases, particularly Case Histories 1 and 2, which involve slope stability problems in the underground and in the open excavation; in Cases 3 and 4, difficult ground conditions were encountered. Pay particular attention to the types of temporary supports that were used as well as the method of installation, because they govern the difficulties that developed. We will cover the difficulties experienced in these cases, the remedial measures that were used, and the design changes and construction changes that occurred such that safe, temporary support of openings was developed.

The rock types associated with all of these cases are essentially the same. They consist of metamorphic rocks, primarily schist and gneiss, all of which have a well developed foliation. Each one of the case histories have geologic conditions in common; namely the presence of foliation shear zones. All of the zones were found to be virtually continuous and could be followed in these excavations for distances in excess of 300 feet. The shear zones contained a gouge. The dominant clay mineral in the gouge has been found to be montmorillonite. In all the cases that I will describe, the foliation shears were running subparallel with the long axis of the opening. Similar types of failures can occur in many other different geologic settings where a major structural feature runs subparallel to the long axis of the excavation – such features as fault zones or mylonite seams in sedimentary rocks. In one of the cases, a conjugate shear zone was present. These shear zones strike parallel or subparallel to foliation shear zones but dip in the opposite direction. The shear zones and the conjugate shear zones have essentially the same properties as the foliation shear zones.

In all the areas, very typical of metamorphic rocks, well developed joints were present in the rock masses. There were four sets of joints, two of which, No. 1 and No. 3, strike north-south and dip in an east and west direction. The other two sets, No. 2 and No. 4, strike east-west and dip north-south. Thus, enough sets are present and can combine to isolate individual rock wedges and allow these wedges to move. Rock blocks can be isolated by foliation shear zones, joints, and conjugate joints. All the joints are virtually continuous, which is typical of metamorphic rocks. The joints often contain gouge, very similar to that in the shear zones. For the tunnel cases, we are talking about tunnels that are located 70 to 700 feet below the ground surface.

The first case history involves an open excavation 60 feet wide and 52 feet deep. The upper 10 to 15 feet of excavation was in soil; the remaining was in rock. The essential elements of this excavation involved a center drift, driven in two lifts, and a cushion, to protect the soldier piles. Rakers were installed to hold the soldier piles in place. The cushion was then excavated, the pile extended to the end invert level, and the sidewalls were coated with shotcrete.

This method was quite successful for 1400 feet; however, in the next 100 feet, problems developed. The reason they were so successful initially was that most of the sections that were driven actually were in soil, and the rock had not been encountered. The primary reason for this failure is the presence of foliation shear zones, which dipped out of the wall. This foliation shear zone was extremely continuous and had a strike within about 10° of the long axis of the opening.

The contractor had made the excavation starting at the base of the soldier pile, and working downward, he completed the first stage opening. In doing so, he maintained a closeness, or very close position, of the sidewall. The reason for that was so that the contractor could put the rakers as close to the base of the soldier piles as possible, but also because of the overbreak that developed. One night, the lower level was shot. The next morning, excavation of the material exposed the shear zone, and rock block movements progressed from the sidewall behind the soldier piles. Four of the piles were daylighted, and the support system gradually collapsed. The timber lagging ultimately loosened and the entire mass moved into the excavation.

The primary cause of this failure is related to the presence of the foliation shear zone and the inadequate support of the high, vertical sidewall. After this section was cleaned up and resupported, the contractor, for the remainder of the opening, used a very careful procedure in making the excavations along both sidewalls. Essentially, he cut down the height, length, and width of the lifts. He immediately supported the rock with shotcrete and rock bolts and very successfully drove the remainder of this open excavation, even though many foliation shear zones were encountered.

In the next case, we have an opening, which is really part of a larger opening, that has a width of about 18 feet and a height of about 26 feet with very steep vertical sidewalls. This particular opening is located about 1200 or 1400 feet from the previous failure. The initial support involved a layer of shotcrete, which was sprayed on the crown and on the sidewalls, within about 10 to 15 feet of the invert. The posts were placed next; the beam across the top was placed, and then the rib was brought into contact with the rock using lagging. Rock bolts were then installed around the crown to pre-support the rock in the larger opening which was to be made at a later time. Very early in the driving of this opening, we noticed instability problems in the east sidewall. Primarily, the shotcrete began to crack. Joints opened, blocks moved, and, in many instances, failures occurred in the east sidewall. Failure occurred when the heading was reasonably close to the block and it ended up displacing the blocks about 8 inches into the tunnel.

The contractor went into this area and placed additional shotcrete and rock bolts in the sidewall. Unfortunately, the rock bolts were very short; they were only about 12 feet long. From results of extensometer measurements, they were very ineffective in preventing rock movements such as already occurred. Because of the conditions that existed in this opening, we decided to install extensometers, to take a look at rock block movements back behind the opening. In doing this, we installed the extensometers at the crown of the opening and in both sidewalls. Even though the crown was reasonably well supported, we wanted to include measurements in the crown and to correlate those with movements that were occurring in the sidewall. Results from the extensometers showed that there was a significant amount of movement that was occurring and most of the movement was deep. Most of the movement in the crown extensometer was located 20 to 30 feet above the crown. There is a direct correlation between the movements that were occurring in the sidewall with those that were occurring in the crown, with a slight difference in magnitude. In the tunnel itself you could see very little deformation of the support in the crown. The only evidence, the only indication, of the presence of an instability problem was in the east sidewall. The extensometer results gave us the information that we needed to make construction changes. The contractor shut down his operation, went in and placed very long rock bolts in the sidewalls and additional bolts in the crown, and placed shotcrete in the crown as well as in the sidewalls. Time-displacement data for extensometers located in the crown indicated a very rapid initial rate of movement. With the advance of the tunnel, movement did not tend to level off, but they tended to continue with time even, when the tunnel heading was 40 to 80 feet beyond the extensometer location.

Essentially, the foliation shear zones, which once again were striking subparallel with the opening, dipping into the excavation, were geologic conditions contributing to the instability. Most of the block movements were occurring along these zones. The rock was loosening in the sidewall section. As a result of this loosening and these movements, rock in the crown became mobilized. The interesting thing is that the height of rock that was mobilized, some 20 to 30 feet above the opening, in fact exceeded the lengths of these rock bolts and came very close to the interface between the soil and the rock. Through the remainder of the tunnel, the opening was reduced in height; but, more importantly, rockbolts were placed in the sidewalls on a regular pattern between the steel ribs right at the heading. With the placement of these bolts, other extensometers showed that these bolts stabilized the opening in a very short period of time. So we had a way of checking the effectiveness of the bolting operation. It was the instrumentation that gave us the key to the instability of the ground, indicated that instability was caused by the foliation shear zones, and showed a lack of adequate support of those high sidewalls.

In another section, a very deep tunnel, collapse of the tunnel occurred where steel ribs were used in an attempt to hold the ground. This particular opening is about 600 feet to 650 feet below the ground surface. it is essentially a horseshoe-shaped opening; when completed it will be a circular opening about 28 feet in diameter. The opening itself is primarily driven without support. However, when some of these foliation shear zones were encountered, the contractor elected to use steel-rib support in those sections. In the area of the collapse, he was driving in a south direction. Where the shear zones came from the wall across the heading, the contractor began to set steel. He set the steel ribs, which were 8 WF31 members, on about 4-foot centers through the entire section where the shear zones were present in the face. He continued his mining operation approximately 80 feet beyond this steel-ribbed section. The gound within this section started to work; it started to move; it started to load the lagging. The contractor went in and shut down his operations, went back to the north end of the steel-set section, and put in jump sets, that is, sets which are spaced between the original sets. He advanced these jump sets over a period of three days.

The primary geologic considerations were the foliation shear zones and a conjugate shear zone which formed a very large wedge approximately 18 feet above the opening. These combined with the joints to isolate a very large mass of rock. The temporary support and the additional support that was used to attempt to arrest the movements was not placed fast enough to prevent this failure from occurring. In the jump-set section, collapse did not occur; it was in the section that had not been reached with these jump sets that the failure did take place. The contractor then put in jump sets in the failed section and retimbered all the way up to the top of the "cathedral" that was formed by the intersection of the two shear zones.

In the last case history, the sets started to work and then they started to take load. In this opening, which is approximately 34 feet wide and 23 feet high, the initial support consisted of a very thin layer of shotcrete spread across the arch. The steel ribs were 8 WF 41 members. In the section where the movements occurred, the contractor had to put a dutchman in the crown because the section was enlarging. The cribbing that was placed was not adequate and did not give adequate coverage to bring the ribs into contact with the rock, particularly in the haunch area. After the mucking operation and during the time the contractor was preparing to place shotcrete, fallout occurred in the heading. The fallout involved a block approximately 2 feet by 2 feet by 3 feet. Then the rock started to mobilize along a shear zone and load came onto the last steel set that was placed. The movements themselves progressed towards the portal and involved some 110 feet of tunnel. None of the steel sets collapsed; maximum deflections were about 6 inches.

Once again, foliation shear zoning was one of the key elements in allowing the rock blocks to move. In this case, in contrast to the last case, the joints that were present on the other side of the block were quite discontinuous. The system is such, however, that they permitted rock block movements to occur, allowing failure to develop and load to come onto the steel sets. The contractor, once things settled down, immediately went in and placed posts at the quarter arch to keep the ribs from turning inside out. He was able to place these posts over a distance of 110 feet within about 24 hours after initial movements occurred. At Dr. Deere's suggestion, rock bolts were then installed right behind the posts, and rock bolts were particularly effective in keeping the additional movements to a minimum, such that additional load on the steel ribs could not develop. The contractor, in the last rib section that actually did not move, then started to place shotcrete on the walls and in the haunch area to bring the rib in contact with rock to prevent any type of lateral movement and thereby prevent buckling of the rib. This procedure was carried all the way to the face. The contractor went back in and removed the posts and no additional movement occurred.

In comparing the two cases, one can see that it was the rapidity with which the problem was dealt with and handled that saved this section from additional loads that may have developed because of the joints and because of the discontinuities. The contractor then placed support immediately to prevent the ribs from turning inside out, using a support system which did not allow additional rock block displacements to occur.

I am certain I do not have to tell you this, but it is worthwhile mentioning the importance of bringing the rib into contact with the rock. Most people are well aware that this is a very important requirement in the use of steel ribs so they can fulfill their design functions. However, in many tunnel jobs, these conditions are not satisfied. The most critical area is the area in the haunch, which prevents the outward movement of the rib and in turn prevents the rib from turning inside out.

Questions, Answers, and Comments

Comment by Don U. Deere – When the rock movement started at the Metro station in Washington (discussed by Jim Mahar), the extensometer showed that, for the first two or three days, the movement was at the first block in the crown; i.e., between the surface and the first anchor, which maybe was 5 or 6 feet back. The next week, we found that the movement had progressed and it was now between perhaps the 6-foot anchor and the 12-foot anchor. The next week, it had progressed back to 18 or 20 or 23 feet. This same thing has been noticed on other projects, of course. If you allow the first blocks to start to come, then the whole rock mass tends to loosen up and you build a lot larger load than if this first block had not started to move.

With respect to the failure of the steel sets, I think it is interesting to note there have been three such failures within the last year and a half. We all think that I-beams and wide-flange beams are great things. They certainly are expensive enough now; bidding prices are about $1.00 to $1.50 per pound in place. So one of these steel ribs is about a $5,000 item, and they really do not do a very good job. They are not space frames, and if the blocking gets a little loose, there is so much room for lateral movement that you get into trouble with the hinges or with the connections. They are not very good to resist torsion. So in general, I do not think they do a good job of supporting the rock.

I am much more in favor of rock bolts. It is interesting that many times we have been able to save the steel sets by putting in rock bolts in between. Then we could really take out the steel sets. It is a little more hazardous to put in rock bolts when the rock is a little blocky. That is why the contractor and the workmen feel much happier when they can put up a steel set. I can see us perhaps going toward using very light sections strictly for a canopy for protection; then under this protection, installing rock bolts. With the polyester resin bolts now being used, which you can grout the full length, install them in 3 minutes, and in 5 minutes they are worth 10,000 psi, you can really get a fine bond fast. I believe that now, with the shortage of steel, with the increased prices of steel, we are simply going to have to go to more rock bolts, more shotcrete, and lesser weights in steel where we do use it.

Yesterday, we had a meeting with one of the design firms where we are studying a subway. They have had trouble with this subway system in that the bids are coming in very high. I think an engineer's estimate on one project was $33 million and the bid came in something like $43 million. It was not awarded, but was redesigned and put back out about 6 months later. The bid came in at $46 million; just one bid. In New York City, there is a real problem with tunnels simply because you no longer can make an engineer's estimate. We have all kinds of tunnels projected there. No one is interested in bidding a tunnel in New York, so there is no competition. This is one of the real problems that has hit New York City, and with the additional water tunnels to complete and the new subway system, it is going to continue to be a problem. I do think our meeting yesterday was historic in that the design engineering firm and a couple of transit authorities would like to put together a consulting panel. I was sitting yesterday in a design meeting with the vice president of construction of the underground division of a large contracting firm, who was there as a consultant. Next to him was a representative from another one of the really large contractors, also there as a consultant. Next to him was the chief of the tunnel division of another large contractor. We thus had involved in the design three of our biggest contractors on heavy underground construction in the United States, working side by side with consultants of the engineering design firm. It was one of the most enjoyable meetings I think I have ever attended. The language was a little more colorful, sometimes, and there is no doubt about how uncertain these people are on certain subjects.

Question -- Is it feasible to use shorter bolts initially to keep the small blocks from falling down as happened in the last two cases mentioned by Mr. Mahar?

Don U. Deere -- I think the concept is certainly good and is one which has been used a lot. It works, but it is just as if you were designing a reinforced concrete arch in which you build a plane on which you put axle grease. You just do not have enough shearing resistance to develop the arch action if that plane happens to be at a crucial angle. I think that in the situation we had here, where the planes were so through-going that they were mobilizing such large masses of rock, the short bolts would not have been able to give us a good arching action.

Jim Mahar -- I would agree that this was the case with the sidewall bolts that were used in the case where we have the high vertical sidewall. The sidewall bolts were only about 12 feet long and you can get by with those lengths, which are essentially half or one third of the width, providing you install the bolts at the working face. If you wait to install 15 or 20 or 30 feet behind the face, then you have to go to a much longer bolt to get behind the mass of rock that has already moved. On the contrary, however, those bolts in the crown were part of a larger structure; they were not used just to support the opening you saw. The opening you saw was 18 feet of the 76-foot wide station. For the rock bolts placed in the crown, the 24-foot long bolts were used to presupport the rock in the arch.

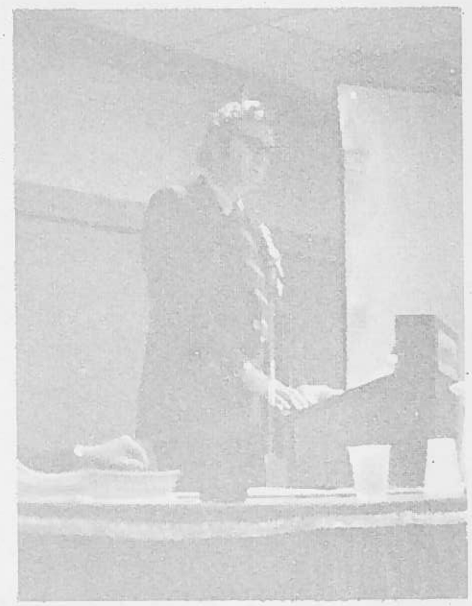
Question -- Is timber getting competitive with steel and is it true that timbers "talks to you," giving indications of forthcoming failure?

Don U. Deere -- I am sure it is true, but we do not have timber men around anymore. In New York City, you had a certain breed of fellow who had this type of experience. They could put down a timbered shaft to rock, whether it was 10, or 20, or 30 feet down. But these people just are not around anymore. This is one of the difficulties with steel -- it takes really good blocking to block against the rock. Contractors block fairly well, and then they blast right in front of it. The blasting knocks out the blocking, particularly on the sides, and especially the lowest one down. And that is the most important one. When the top starts to come down and there is no restrain there, the top opens up and pops the bolts. I would agree with you. I think in some cases you would feel a lot better if you had timber in place and could see what it is doing. However, I am a much greater believer in not letting the rock move even that much, by getting rock bolts in quickly.

Comment by Don U. Deere -- The contractor must go in and muck out, and within an hour or two get the bolt in within two or three feet of the face because then, with the next round of blasting, you get an elastic movement. Forget for a moment about time-dependent behavior and sliding on the shear zones. Just the elastic movement will, of course, cause the bolt to tension itself. With subsequent elastic movement, the rock will have hardly opened up. Yet you can get very high tension loads in the bolt. For example, for a bolt 10 or 12 feet long to be loaded with 30,000 pounds of tension, you can figure from Pl/AE the elongation you are going to get in that bolt; maybe 1/4 of an inch or 3/8 of an inch. That is too much. We do not want the rock to move that much. But this is not quite a true picture either, because it is only at the joint that the rock wants to separate. Since the bolt is grouted throughout, the load is not applied over the full length of 8 or 10 or 12 feet; the load is applied over a length of perhaps 1/2 inch. So the elongation is a few thousands of an inch. You must tension completely at that area where you need it. I have used up to 80-foot-long bolts, grouted in, and measured some elongations up to 2 inches. But it still did a very nice job in holding the rock. I do not have any comparative data, only the fact that we have used a great number of grouted dowels or untensioned bolts and have had great success with them.

Comment by Don U. Deere -- There has been so much money lost in underground construction, so many tunnels have gone out at $10 million and it has taken $12 million to do the job, or it has gone out at $50 million and it has taken $100 million to do it. The water tunnels in New York City were the biggest contract, $350 million, ever let in the United States for civil construction 3 years ago. The contract documents provided for 700,000 pounds of steel for the entire system, 25 miles of tunnels. The job is a little more than 60 percent completed and they have already used more than a million pounds, an overrun of more than 50 percent. Obviously, there is already a large construction claim. This has happened on private jobs, on government jobs, all over this country; it is just too difficult to predict the behavior of the subsurface. The contractors are claim-conscious, many times rightly so, and other times perhaps not. There has been a big tendency in the ASCE and other organizations to look at other contracting methods. One method is the target estimate with a bonus and a penalty clause in which the owner pays for doing the job. If it is done within the cost estimate, he shares 60 percent of any profits. If it overruns, the contractor pays 30 percent on only the overrun. The contractor cannot lose more than his total profit. The contractor is either working for the profit he bids, for nothing, or for a bonus which can be as much as double the profit he bids. We think that this is taking a lot of the gamble out of underground construction bidding. Instead of an adversary relationship between the design engineer and the owner with the contractor, it brings them together. I sold this idea for a big project. Three days ago the bids came in from six contractors that were selected. They were given the presentation, we brought them to the site, we had 2-day meetings with everyone, and now it has gone out. Whether the government can do this or not, we do not know. I doubt it, but I think in the future they are going to have to do this. I have already talked to the Bureau of Reclamation and they are ready to listen because tunnels are just costing too much. But on this new project, it could be in the range of a $100-million job, the owner is going to reimburse the contractor for all the labor it takes to do that job, he is going to pay for all materials, he is going to take care of escalation and other things. The contractor is just going to do the finest work of managing that job and getting his laborers to perform, using only the kind of support that is really needed. If there is a savings, he has got a big hunk of it. The project involves cooling water tunnels under the Atlantic, about 2 miles long, for a nuclear power plant. So there is some risk involved from "ground" water or ocean water, or whatever you want to call it. We are trying to minimize that risk. Contractors so far are very happy. We will not know until the job is completed how well the scheme works. We, as engineers, are very happy with it. The owner had to be sold, and now he is happy with it. And the contractors, to date, seem to be very pleased with this opportunity to win or make themselves $8 or $10 million, keeping their losses down to something that will not put them out of business. The jobs today are just too big for a contractor to take a risk of having a big loss. When jobs are $3 or $4 million, the contractor can take a loss and make it up the next time. He might make a windfall on the next job. Now, if you lose one, you are out.

J. A. Waddell, Martin Marietta, speaking at the 1974 Seminar



H. Thomas, USCEC, at the 1974 Seminar

Presentation by M. W. Palmer, University of Louisville, at the 1974 Seminar

ROCK ENGINEERING ON SOME RECENT PROJECTS

by

Don U. Deere
Consultant in Engineering Geology and Rock Mechanics
Visiting Professor, University of Florida

It is a pleasure to be here and to see again so many old friends and colleagues in this part of the country. I wish to spend about 7 minutes summarizing my thoughts on rock engineering in a technical way, and then perhaps about 25 minutes talking about case histories by using a few color slides. Now, the 7-minute summary that I would like to give regarding rock engineering will include many points which you have already heard in many of the papers presented today. It is a sort of summary, perhaps, not only of my thoughts but of the development of rock mechanics by many different people in many different parts of the world.

"Rock Engineering" is the title of this conference and this evening I will concentrate on construction and design of facilities of different types built in rock or on rock. I will not include the use of rock as a construction material; that is, what type of rock is best for rip-rap or dams, types of concrete aggregate, appropriate tests for building materials, etc. Not that this is not an important part of rock engineering. It is. It is just not one I will mention this evening.

In rock engineering, the engineering properties that we are concerned with are the in-situ properties of a rock mass, not those of the sample that goes into the laboratory. The in-situ properties which are important in project after project are the following four: (1) the compressibility of the rock mass, (2) the shear strength of the rock mass, (3) the permeability of the rock mass, and (4) the in-situ state of stress. I did not formerly include this fourth, the in-situ state of stress, until I was on a couple of jobs where the walls of the excavation kept moving in or the face of the tunnel kept exploding in our face. Then I took the hint. The in-situ properties of rock depend not only on the kind of rock, but I think to a greater extent on the number and the character of the joints, bedding planes, and the other discontinuities in the mass. Therefore, it is the details of the site geology that really dictate in-situ rock properties.

It is axiomatic that no rock mechanics test or rock mechanics analysis should be made until the geologic framework is understood. And for working out the geologic details at a site, it is usually necessary to use a variety of exploratory techniques which may include, as you heard this morning, field geologic mapping, air-photo studies, core borings with undisturbed samples and core samples of the rock, test pits, test tunnels, and test shafts. I would like to emphasize the last three because they are expensive and time consuming and neglected far too often. When you have an important rock job where the possibility of failure is rather drastic in terms of danger, in terms of cost, then you are justified in telling your client, whether he is an architect (who usually will not listen) or a structural designer or the owner, that you must get the information or you cannot do him any good. Too often, consultants sell themselves short by doing only what the client feels is necessary, rather than telling the client what is necessary to get the information. This often means getting the money, the time, to drive a test tunnel into the area, to sink a test shaft through rock perhaps a hundred feet, or to open test trenches, removing 25,000 or 50,000 cubic yards of rock. If the project is important, then it may be necessary to do these things.

Until the geologist can construct geologic cross sections or geologic profiles across the site in different directions which very accurately depict the geologic conditions at the site -- meaning the type of overburden material; the depth to the top of rock; the amount, depth, and irregularity of weathering; the types of geological structure and joints; and the ground water table -- then the geologist does not really understand the site geology. I have found time and time again on projects that the most difficult thing is to get the project geologist to draw a cross section. He likes to make borings, log cores, talk, and do various things; but many times he is very reluctant to draw a cross section, for once the cross section is drawn, he has committed himself. And you know geologists are rather careful about committing themselves. Two geologists were driving down the road and one was commenting, "Oh! I see that herd of sheep there, they have just recently been sheared, haven't they?" The other geologist looked very carefully and said, "Well, at least on this side."

In metamorphic rocks, and we do not have many metamorphic rocks in this area -- all we have to do is go a little farther to the east, Philadelphia, New York, Washington, and Baltimore or Boston, etc. and we find that we have primarily metamorphic rocks to deal with -- one must be very careful because they often contain so-called "mud seams," foliation shear zones. This zone usually is a band of foliated rock, usually high in mica or mica schist, which is a little weaker than the rock on the two sides. So as the mountains have been folded up, the rocks have been brought into their positions many many eons ago. There has been a little differential movement, a little sliding and slipping, only a few inches to a few feet on the weakest materials, and these are invariably in the micaceous zones. This slipping crushes the rock and forms a thin seam of talcum powder or ground up mica which, when wet, is plastic and weak. These are EXTREMELY common and they can be disasterous to a project. These thin zones of foliation shears may only be an inch or so thick but they may continue for hundreds of feet.

In sedimentary rocks which we have in this part of the country, we have a similar situation in which bedding plane shears take place. With the cutting down of the river valleys or with slight folding or faulting, there is a readjustment which allows some of the beds to move with respect to the others. This shearing displacement always takes place in the weakest materials. And what are the weakest materials? The shale zone, the thin coal seam, the lignite seam, the underclay. And many times these have been sheared so that the material has been ground up; we no longer have interlocking rocks but a mud seam. This seam is very weak with respect to the rest of the bedding planes. There are great problems in the Ohio River valley and many other parts of the world with these so-called bedding plane shears, bedding plane faults, or shale mylonites.

You will hear a little bit more about these two most adverse geological features in the following paragraphs; that is, the foliation shear zone and the shale mylonite layer. I think that if all the rest of engineering geology could be thrown out and we would only teach to our students about foliation shear zones, shale mylonite seams, and their physical properties, then we would still turn out excellent engineering geologists.

At one time I had eight projects going in various parts of the world that were confronted with shale mylonite seams. I had eight projects with foliation shear zones. Every little zone was worth a million or two million dollars in redesign or in construction problems. So I have become, over the last 10 years or so, very impressed with the importance of these highly significant features.

Let us return for a moment to the in-situ properties of the rock mass. First, the compressibility ... what is it? Why is it important? How do we determine it? It is simply the modulus of the rock mass. It is a gross value that includes not only the solid rock blocks, the joint blocks, but also the closing of the cracks that are present; also, if we have clay in the some of the joints or weathering along them, the compression of this material as well must be considered. Therefore, it is obvious that the in-situ modulus is much lower than that determined in the laboratory; the rock mass is more deformable than is a core that we would take into the laboratory. I would say, in general, that the modulus of rock in situ is probably only one-half to 1/20 of the modulus of the rock in a laboratory specimen.

Why is this modulus important? Because it determines how much the rock is going to displace when a load is placed on it. When building a bridge pier, a radar tower, or a gravity concrete dam, it is the property that gives us some information as to how much this structure will settle and perhaps how much differential settlement there will be between one part of the dam or one footing and the adjacent footing or the adjacent part of the dam. These differential displacements will result in very high stresses being developed in the concrete structure or steel structure that is being founded on the rock. Dr. D'Appolonia in his paper this morning gave a good illustration of the problem of rock modulus, although their problem there seemed to be a problem of engineering geology definition rather than a rock mechanics problem.

An arch dam is one of the most beautiful structures in the world. Very thin, arched, and with terrific force or hydraulic thrust of a reservoir carried through the arched shape into the abutments. You usually count on the abutments being a fairly solid rock. If one abutment is not very solid rock, then there will be deformation of that abutment; and as the abutment deforms, the arch dam deforms and stresses are set up in that dam for which it was not designed. Cracks develop and out-right failures may occur. This has happened in a few cases with a great number of deaths and property damage. Therefore, the rock mechanics engineer who is called on to determine the in-situ modulus for an arch dam has one of the most important jobs in the world. If he misses, there may be a failure of the project that may have cost $50 million and the lose of the lives of thousands or hundreds of thousands of people, depending on the location of the dam.

In a pressure tunnel going through rock that is not very strong, it is not uncommon that a steel liner is placed inside of the tunnel. As the water pressure builds up and pushes against the lining, the lining expands against the rock. If the rock is of fairly good quality, the rock gives resistance; it keeps the steel liner from expanding and the concrete around it from expanding and you have a stable condition. If through this area of the tunnel there is some place where there is a fault zone, a weathered zone, or different type of rock of low quality or low modulus, in that area the expansion is not prevented and often the steel liner will burst, the water will come flooding out into the mountain, the hillside will get saturated, and you may get a big landslide that may actually take out the power plant below.

How does one determine the modulus? Several ways. One, you make a large-scale, in-situ test. Large scale means a loaded area two feet, three feet, all the way up to perhaps five or six feet in diameter. It can be a load test on the surface, it can be a footing that is jacked down or pulled down by a cable, or it can be a radial jack test, as was noted this morning. A liner can be expanded against the tunnel, but this is fairly expensive. You can also do it in a bore hole, but there you have a scale effect. A bore hole is fairly small, and if you are dealing with rock that consists of big joint blocks, that one little hole and little test in it does not tell you too much. So it can be misleading under certain circumstances. Another way is to do a seismic test. We heard about that this morning. All you have to do is multiply the square of the seismic velocity by the appropriate factors, including Poisson's ratio, and you can get a dynamic modulus. But is that dynamic modulus the same as the static modulus? The answer is NO. It is not. It is much higher. So the dynamic modulus that you get by seismic tests must be reduced. This reduction factor cannot be determined by theory. It can only be determined by tests. Our experience has shown that maybe one-fourth to one-fifth is the correct reduction factor.

Let's talk for a moment about shear strength, the other in-situ property. The shear strength of importance is that along a joint or bedding plane or a weak zone. Because rock in itself is usually stronger than concrete, it is only when you have a crack in it at the wrong angle that things are going to slide. Therefore, it is usually the frictional resistance in a given direction or in a couple of directions that is important. Where we have failures with shear strength of rock, almost invariably the rock has moved along a pre-existing joint; the intact rock did not fracture. Where the joint surface is fairly rough, there is interlocking of the rock on one side of the crack with that on the other side. We may have a little cohesion and we get very good strength values. However, where that joint surface is quite smooth, the friction is very very small (the ϕ angle may be only 10 or 20 degrees). Where we have had failures, it is often because we do have such a condition or we have weathered rock or clay in a joint. Typical shear strength problems in civil engineering include (1) the sliding resistance of a hydraulic structure, e.g., a dam; (2) stability of a rock cut (if we make a deep cut for a highway (and the interstate highways have many deep rock cuts) or for a spillway or in an open pit mine, we can get rather large stability problems with large landslides); and (3) stability of the walls of underground openings, such as large tunnels or large underground power plants (not only the stability of the walls but roof stability depends on shearing resistance).

We like to think that rock is an elastic material, but we know that it is not. At a given depth below the surface, the vertical pressure should be equal to the weight of all the overlying material. Let's say at a depth of a thousand feet, the pressure would be equal to the weight of a thousand feet of rock, or roughly 1 psi per foot of depth, since rock weighs roughly 144 pounds per cubic foot. So at a depth of 1,000 feet, we could expect the vertical stress to be about 1,000 psi (at 500 feet, 500 psi; at 10 feet, 10 psi); by measurement this has been found to be just about right.

However, what is the stress in a horizontal direction? The people in theoretical and applied mechanics use Poisson's ratio and say it is very easy to determine lateral stresses. It is the product of vertical stress and Poisson's ratio over one minus Poisson's ratio, or about 0.3 or 0.4. This means if you are 10 feet deep and you have 10 psi vertical pressure, the horizontal pressure should be about 3.5 to 4 psi. This has been found not to be true.

If we were dealing with water, the horizontal pressure would be equal to the vertical pressure. In some rock, the horizontal pressure is at least equal to the vertical pressure, and in some cases, it is greater. I think this situation is similar to compacting fill with a bulldozer against a retaining wall. As the bulldozer comes along and compacts the fill and pushes out against the retaining wall and then goes on by, that fill does not ever recover its original state. It has been pushed down and wedged in against the retaining wall, so it has built up a high horizontal stress. The bulldozer went by, the horizontal stress is still there. This is what happened in the earth's crust. In many cases, we have horizontal stresses much greater than you can account for simply by the theory of elasticity. It has been shown on many projects that the horizontal stress is between one and a half and perhaps three times the vertical stress. Not always, but it is a fairly good original starting place until you try to determine it by in-situ means.

Now what does this high in-situ stress do? For one thing, it can cause the sides of a cut to move in. For another, if a rock mass has high horizontal stresses and you drive a tunnel into it, you can have pieces of rock popping off the sides of the tunnel with explosive force. In driving a tunnel underneath Welfare Island in New York last year, they ran into a very great problem with explosions of the rock from the walls and from the face of the tunnel to the extent that, for several days, the miners would not go back in. This was a high quality rock; everybody thought everything was going to be great. The rock was too good, it had too much stress, and the granite was essentially fracturing in front and blowing out at you. It can be a very frightening thing. In areas where this stress is even higher such as some of the mines in India and South Africa, while driving a tunnel or drilling a shaft, all of a sudden the high stresses will cause the rock to fracture and the hole is just simply eliminated. Anybody in it is also eliminated. It just simply closes with explosive force. We do not have too much of that on this continent; but we do have some problems.

In summary, rock engineering involves:

1. Determination of the geologic framework and geologic details at the site -- The more I work and the more my colleagues work in applied rock mechanics, I think the more we come to appreciate that engineering geology is more and more important. You cannot do rock mechanics until the engineering geology is worked out.

2. Determination of the pertinent in-situ rock mechanics properties, the ones that are pertinent to your problem -- These can only be determined by in-situ testing, in general. In-situ testing starts at about $5,000 or $10,000. So on a major project where you may have six, eight, or ten tests, you are easily spending $100,000, and $1,000,000 on the bigger projects is not uncommon.

3. Analysis of the anticipated behavior -- This is exactly what Dr. D'Appolonia was talking about this morning. They knew the loads, they knew the stresses were going to come; they were trying to get an idea of the rock properties and then they were able to make a prediction of the behavior. After you have made the prediction comes a most important part.

4. Assessment of the anticipated behavior in terms of its acceptability -- In other words, you can predict how something is going to happen. Then you can walk away from it -- you would be an analytical man, you would be a theoretical and applied mechanics man. But we are not interested in that for our final end product. We are interested in engineering decisions. Is the rock going to settle an inch? Is that acceptable? Is it going to settle 4 inches. Is that acceptable? It is going to have a shear stress of so much, a factor of safety of 1.05. Is that acceptable? This is where the engineering accessment comes in. You have to look at the anticipated behavior, evaluate how well you really know the properties that you used in making your analysis. In other words, how correct is this estimate? What is the probability? Then you have to determine the acceptability as to performance and to cost.

I would like to relate a little episode that I related to my students in the 18 years I was at the University of Illinois. For one of the last jobs I did in Puerto Rico as a foundation engineer in 1955, I was called by the India Brewery Company, which makes a really fine beer. I knew it well. In fact, it was selling so well that they decided to make an extension to their brewery. It was down below the campus of the university, on the flood plain. The architect and the structural engineer said they needed a soil investigation. The owner understood I was in this business and asked if I would take it on. I said, "Yes, fine, we will make some borings, we will do the appropriate testing, we will recommend the foundations." We did all this, and I finished the report and sent it over to him, together with the bill. He asked me to come over, he wanted to discuss the report with me and maybe the bill, also. He said, "I see that you have recommended pile foundations." I said, "Yes, that is right. You are on a flood plain here, and the piles would be 90 feet long to get through this very soft flood plain silt." He said, "How much will the piles cost?" I said, "We have estimated between $90,000 and $100,000." He said, "But this existing building does not have piles, it is just sitting out here on the dirt." I smiled; I thought I had him, and I said, "That's right and look at the cracks in this building" We measured the differential settlements; we do not know what the total settlement was because there were no original level measurements made. But the differential settlement between columns was 6 inches. The concrete was cracked all over. I said, "Look at that crack in the wall behind you." He turned around, and, I tell you that crack in that concrete wall was at least a half inch open. You could look out and see a lot of Puerto Rica through that crack. I smiled and said, "Well, if you don't put it on piles it is going to settle and you are going to get cracks just like this." He turned around, looked at that crack and said, "I can look at a lot of cracks for $100,000." That was heresy. Here I was, a young, eager foundation engineer and my building was going to crack. I really turned it on and finally convinced him that it had to go on piles. We put it on piles and it has not cracked. Yet, as I get older, I keep remembering this and thinking back. I think maybe he was right. He was paying those people to work for him in the brewery. He even let beer come out of the water fountains. So the people liked to work there. They could not have cared less if there were a few cracks in the building; it made it cooler. Here I was, a foundation engineer, a purist, who did not want any cracks in my building. I am convinced to this day that I made a poor engineering assessment of that job. That job could have been $100,000 cheaper, that building would have cracked, and nobody would have been any unhappier. So engineering assessment is not absolute, you must take a lot of things into account.

5. Designing remedial measures to make the behavior a little bit better in case that is what your

analysis shows has to be done – This may mean using long anchors, rock bolts, gunite, or maybe drainage, as you have seen today. Applied rock mechanics is not an exact field. But I would differ with almost everyone who has spoken here today – it is not an art. It is an applied science that is recognized as such. There is a history of the application of rock mechanics to a variety of engineering projects with notable successes. There are some failures, as in many new and developing fields, but I believe it has passed from the art, as it was used by the old engineering geologists many years ago and by the foundation engineers, to an applied science. But one has to get the data and this, of course, is easier to do on a big job than it is on a smaller one.

(Editor's Note: At this point in his presentation, Dr. Deere displayed a number of colored slides of rock engineering projects.)

Those of you who have been looking at the Prudential advertisements probably have seen the other side of this particular rock. But I wonder if the Prudential people have seen this side of it. This is the back of Gibraltar from near the town of Algeciras in Spain. As one turns 90 degrees to the south, you look across an 8-mile expanse of beautiful water and see the mountains of Morocco in North Africa. It was decided that it might be profitable to drive a tunnel across this area in the rock below the water and bring oil from Morocco into Spain, and on into France and Germany. The oil and natural gas would come from Algeria, through pipelines on the land, and across here. The first thought was that a pipeline on the bottom of the ocean could be used. The only trouble was the current; it is very very fast. The water comes pouring in through the Straits of Gibraltar, and there is no possibility that a pipe would remain in place on the bottom. Therefore, it would be necessary to sink shafts in Spain and in Morocco 2,000 feet deep and then drive a tunnel, which would give you a depth of at least 400 or 500 feet of rock and mud between the tunnel and the bottom of the Mediterranean.

The outcrops on the coast of Spain are quite outstanding. Here you see that the beds of siltstone, fine sandstone, and shale are nearly vertical. Very easy to do a geologic study here. Sometimes there are a few contortions, indicating that the rocks have been subjected to some great tectonic forces in the past and have been shoved around a little bit. Some places have been cut with a fault; you can see how the lines of bedding have been cut and offset. As we cross over to Morocco, we find that the beds, instead of being parallel to the coast, are at right angles to the coast. So it is quite apparent something has happened between Spain and Morocco, and it is very difficult to tell what. There are no borings in that area. The current is very fast, boat traffic is very great, and it would cost a million dollars to move out with a boat and even start drilling and probably two million dollars for the first hole. So the original study of the tunnel was strictly surface work on the two coast lines. Then we got a large ship and did sonar soundings on a half mile grid throughout the area, trying to get some idea of the geology in the center of the area to make a feasibility study, at least a first estimate of cost.

It was also considered that it might be worthwhile to go a little bit farther to the west, to Tangier, and to start back from the coast with a long inclined tunnel instead of a 2,000-foot vertical shaft, and then come up in Spain so the gradient would never be more than 4 to 5 percent. Then they could run a train that would be able to take cars from Africa to Europe. But the cost would be very much greater. The length of the tunnel would be 25 miles – although we found a spot where the water was only 800 feet deep so the tunnel would not be quite so deep.

Back in Spain again, we see the beds are vertical but they have also been contorted. You can see a big fold where the rocks have been contorted, and this is representative of the area. I might mention that this is where my son, who was helping me three or four years ago in the field in making the studies of Morocco and Spain decided he would be a geologist. It happens that in southern Spain in the summer time, all of the blonds from Norway and Sweden come down to sun themselves. As we were going up and down the beach there studying the geological formations, I was not always sure he was studying the right formation. He did tell me he thought geology field work was a lot of fun.

Only a couple of hundred miles from Louisville over in Ohio, they are stripping coal in Muskingum County. They decided that, rather than use small shovels, they would manufacture or would buy the world's largest dragline. They could strip away all of the soil to depths of 125 feet with this little instrument and expose the coal. The dragline was designed and built by one of the equipment companies. If it looks big, it is. Two Greyhound buses can get into the bucket, a 225 cubic yard bucket – the world's largest at that time. A link of the chain weighs a couple of tons. The weight of this dragline was 25 million pounds. Knowing that a car weighs about 4,000 pounds and it costs about $4,000, that would show you that machines cost about $1.00 a pound. This machine weighs 25,000,000 pounds, and it cost $25,000,000. One day one of the officials of the company said, "You mean we are going to put $25,000,000 up on the top of that slope AND it is suppose to be digging way down below". They suddenly saw they were putting all their eggs in one basket because, if the operator got about 1 foot too close, there could be a collapse of the high wall and the machine would be down and the company would be out of business. Since the company supplies coal for a power plant, the power plant would also be out of business. So it became of great economic importance to this corporation to determine how close to the edge this machine could work. If it worked close to the edge, it was very efficient, it could reach all the way down. If you made it stay back from the edge 15 or 30 feet, it got less and less efficient. Looking around the area, you could see little slides here and there, not very big. But what if the machine had been there? Here is a joint cutting right behind the rock. What if the machine had been sitting there? Around the county you find some shales and underclays which again have large slope failures, just due to the weight of the material itself. So it became a problem in applied rock mechanics as to how to operate that 25-million dollar piece of equipment. You had to try to decide what would determine the failures – obviously the geology. So then it became a very detailed study of many days trying to determine what were the controlling geological joints, faults, erosions patterns, and weathering.

Now to go to an open-pit mine in Chile. This is a nearly vertical slope with two planes that intersect. This morning you saw photos of two intersecting planes. Here is the same thing; the rock has already fallen down. In Brazil, they have a very rich deposit of iron ore. Blasting is used to break up the ore so that it can be loaded by a shovel into trucks and carried down to the concentrator. The rock face in green is not ore, it is the waste rock, and on top of that are brown-looking soil materials. This iron ore continues down with depth. Right now we are probably 300 or 400 feet deep. In another five years, it will be 1500 feet deep, and you do not want the top of the slope coming down into the lower part of the pit. So this is very real problem in stability. It is very real to the mine operators. They do not want to spend any money, but they want absolutely no failures. So it is a tradeoff between trying to be safe yet as economical as possible. Formerly, there were benches on the upper green slope, which incidentally is green schist. One day there was a failure and eight benches came tumbling off in a massive landslide which closed the pit for nearly a year. They decided they would like to try to design around that and not have that problem in the future. Some of the weathered rock and the soil was sampled and various types of tests were made. Samples were tested in triaxial shear and parafined so we can see how the failures developed. One of the failures followed the schistocity, others cut across the schistocity. In the rock itself, there were layers of thin clay seams, actually little thin foliation shear zones. All these contributed to some failures. The iron ore occurs to the left. Then we have a zone of brown weathered material that contains a lot of clay and some graphite, very weak. Behind it and to the right are the inclined layers of green schists. In the green schists, we have some irregularities in the foliation. These irregularities have been very helpful in giving us a little increased shear strength. It appeared that water from heavy rainfalls in the area were seeping into the cracks in the rock, running down, and building up pressures, particularly beneath that brown weathered layer. There was an attempt to drill a hold through the brown weathered material to encounter the harder green schists to see if indeed there was artesian water in it. There was, and since then several drains have been placed. Drains which discharge only one or two gallons per minute have been sufficient to lower the ground-water level in that mountain by several tens of feet, thereby increasing the stability. One of the problems in the residual soil which is formed from the decay of the rock has been erosion with rainfall. Quartzite is very hard, one of the hardest rocks in the world. Due to the high temperature and the weathering over millions of years, this quartzite has lost its cementation and is nothing more than a pile of sand. It erodes very very badly. The surface of the benches are paved with clay and compacted, and slopes are planted with grass in an attempt to stabilize the upper slopes. We had trouble not only with the upper slopes but with the lower slopes. If you do not pave and put in a good drainage system, you see what happens with rainfall. Water seeps into the slope developing a very bad stability condition. Four or five benches are very rich iron ore, almost 99 percent pure hematite running around 71 or 72 percent iron.

Now I would like to move to another area, in Venezuela. I have been impressed there with their use of gunite, concrete which is sprayed on the surface of their deep cuts, particularly in weathered rock and residual soils. The gunite is tied back with long rock bolts or anchors. As you drive along through the highways or city streets in Caracas and out into the countryside, you see very great use of this type of protection.

Now I would like to move to a dam on the Rio Grande near Del Rio, Texas. You can see the vertical holes, which represent presplitting of the limestone. I want to point out three black layers of shale. The shale has been sheared by past geologic movements and you can see a so-called bedding plane shear or a shale mylonite zone. These are very weak and they caused a great number of of difficulties during excavation and in the design. Looking down from a higher elevation, onto part of the abutment, there was solid limestone. The contractor was supposed to bring the dam over and rest it on this material. However, when he was blasting off a little bit of it, the gas pressure went down a vertical joint about 10 feet and hit one of these black clay seams of shale mylonite. The gas went out along the contact, raised the rock block, and the gas in the vertical joint pushed the block out. So here we had movement on a zone about 10 or 15 feet below the excavation line, and the contractor was as surprised as anybody. He just drilled a few holes and shot it and the whole mountain side moved out. Of course, the design engineers, the Corps of Engineers, doing the work for the Boundary Commission said to the contractor, "You used very bad blasting and construction practice." The contractor looked right back at him and said, "You have very lousy rock." The die is cast, this always goes on. Design engineers say bad construction and contractors say bad design and bad rock. Finally, the design engineer said, "You must take out this rock. You ruined it and you have to take it out without blasting." How are you going to take out a chunk of limestone about three times the size of this room without blasting? By drilling a whole bunch of holes, putting in hydraulic jacks, and trying to break or split it into blocks that you can pick up and lift out. You can imagine how much this costs. The contractor contacted me at the time and said, "I think I have been had and I think we have a claim; I think the owner ought to pay for some of this." But what was the real center of the whole thing? This little shale mylonite zone. It is not only here. It happened at Cannelton, it happened at Uniontown, and it happened at Wheeler Lock, It happened in Greece and almost every country you can think of. They are very weak zones and any blasting creates slide.

Here is a river and a large dam made of dirt and gravel that is going to be built to a height of 300 feet. The reservoir will go back up the valley for 100 miles. Here at the side of the valley is a little level terrace and a hill behind. The design engineers have decided to excavate a level spot, to excavate the channel and to build one gravity dam out of concrete and one out of earth and rock. Through the concrete gravity dam they will have three holes with steel penstocks taking the water down the slope to the power house. There will be several gates which will allow the water to escape down the spillway in times of flood, a rather straightforward design. The design was finished last year. It is now out for bids. The contractor will be selected in a matter of just a month or so. It is down in Argentina. Let us take a look at a cross section through this intake channel and the powerhouse. But look at what we have in the way of geology. Sandstones and shales and zones of weaker shales interbedded -- very similar to the Ohio River Valley. And the possibility that there has been shearing displacement forming at one of these contacts a thin bedding plane zone of mud or shale mylonite. If this is the case, that is just like building on grease. When the water pressure comes up there for the first time, I think the concrete dam, this piece of rock, and the whole thing will slide down. The factors of safety were computed by design engineers according to the formula $S = C + \sigma \tan \phi$. The only thing I object to is a cohesion of ten tons per square meter, essentially 15 psi shearing resistance by cohesion. A friction angle of 20 degrees, as assumed, is reasonable, but cohesion is nonexistent on a mylonite layer. We have some exploration in the area which indicates mylonite zones 4 inches thick extending for hundreds of feet. So this obviously must be redesigned for safety. Their safety factor calculation shows 1.44. If the drains do not work, it is only 1.2. But if you eliminate the cohesion, which I am sure nature eliminated many centuries ago, then the factor of safety is 0.8, which means it will fail.

Here we have a very interesting river. There is a deep canyon and a high ridge. The design engineers looked at this and said, "Well, very good. We will build a big dam here, back the reservoir up the river. Through this ridge we can make a little cut and put in a concrete dam and a spillway so we can control it." What I want to show you is the stability problem with this spillway control structure. We have this ridge, very steep, 600 feet high, very steep in the other direction. The idea is to excavate the ridge, place a dam, and hopefully control the floods. They can open the gates and let the water come down the spillway to the ski jump where the water will go way up in the air, dissipate its energy, and finally hit the river about 100 to 200 feet downstream. The only question is, "Will the dam be there when the water comes up?" There is not very much to hold it. It becomes then a rock mechanics problem in trying to assure yourself that the dam is stable. Tunnels have been driven in the mountain. From these tunnels, grout holes have been drilled and concrete has been injected. We have, hopefully, a curtain of grouted rock which will not allow much water to seep into the hillside. Just in case some does get through, we have a line of drain holes to intersept it, because if water comes out of the downstream slope, the situation is very bad. Some foliation zones have been sheared; we have potential wedges bounded by two planes which could move out under the thrust of the water. The problem was the analysis of this situation. But fortunately the analysis we made indicated that it would be stable. The dam was finished on July 27, it was dedicated by the President of Mexico as well as the President of Columbia, and they started filling the reservoir. The water level in the reservoir is rising rapidly, and in a couple of weeks we will know if we have any more jobs in Columbia. The job was completed with the loss of the lives of 57 men; this will show that it was a very difficult site. A lot of people lost their lives by falling off, by landslides, by avalanches, etc. The stability of the dam depends on the design based on the rock mechanics analysis that was made, taking into account the joint sets.

One of the most beautiful waterfalls in South America is Iguacu Falls between Argentina and Brazil in a series of basalt flows; the lava flows are thousands of feet thick. The river comes over these gorgeous waterfalls. Just upstream where the Iguacu River meets the river of Parana, we have a dam site. This is the largest hydro project ever started. It will cost four billion dollars; I think it will now be the second largest because they have started studies for the James Bay in Canada, which I understand will cost eight billion dollars. Both of them are gigantic projects. This river is considerably larger than the Mississippi and the depth of the water at the site is 150 feet. It is a wide river with high flow, high velocity, and great depth. Drilling in the river was very difficult.

Now closer to home, this is out of Mexico City on a flight to the southern-most state of Mexico, very near the Guatemala border. As one is flying over the area, you see a limestone plateau and suddenly there is the deepest gash I have ever seen; it just goes straight down. This is a most impressive canyon, 3,000 feet deep, and I think it has to be more impressive than the Grand Canyon. A number of these blocks of rock on the side have tumbled down, damming the river for a certain time, probably happened hundreds of years ago. There is going to be a dam just downstream of this canyon. The reservoir will fill the bottom third of this canyon. What happens if one of the great blocks of rock, which would be about 1/2 million cubic yards, falls down into the reservoir? How big is the wave? Will the wave go over the dam, and if it does go over the dam, can the dam withstand it? This is one of the problems that is being studied. It is a really rough area to work in on foot. This is a subsidiary canyon, and I think many of you geomorphologists will recognize that this looks like it could be a fault. It is; it is a very active fault, a very recent fault that has just cut at right angles across the main canyon. This area is still working. It is not stable at all. There are active volcanos and the whole crustal area is moving and is subject to continued movement. The limestone bed is dipping down towards the river. The angle of that limestone is about 20 to 30 degrees. When you study the limestone carefully, you find there are thin layers of mud seams or shale mylonites in it. You wonder how those canyon walls could still be there when they are dipping so steeply. When you build a dam downstream and submerge these with the reservoir, this could soften the clay, give uplift pressures, and maybe the walls and the whole thing would collapse and you would lose the dam. So the question is, "What is the shearing resistance along these layers and will it remain sufficiently high with the reservoir inundating the area?" We have done lots of borings. There have been two or three consulting boards. The final decision we made most recently was to go down at river level, select the weakest layer, and drive an exploration tunnel up the contact some thousand feet. We are talking about probably a million-dollar, or at least a half million-dollar, exploration. There will be cross adits following this. This is sort of fantastic, you cannot normally afford to do this. We did not think so until the government of Mexico made a study and determined that this is such a fine site that, if we can build on it, we can afford to spend 40 million dollars extra just to stabilize the rock and to make the studies. With that kind of money, it sort of gets the attention of rock mechanics people.

I would like to discuss one other canyon which has impressed me, a project in Venezuala. This canyon is some 500 feet deep and is about 20 feet wide at the bottom. Again this has been considered for a dam and construction will probably start quite soon. We have about six tunnels going back into the rock so as to study very carefully the internal structure of this mountainside. It is a very narrow ridge; on the other side there is a river valley. A lot of faulting has occurred in the area and it is only about 20 miles from the biggest fault of South America, equivalent to the San Andres Fault of California which is moving about 1 inch every year. We want a tunnel from the reservoir area, going 20 miles through the mountain, crossing the fault, and coming out in a very dry arid valley that will use the water for reclamation. Two years ago, they could not have undertaken this project. Becaue of your kindness in raising your oil prices, Venezuela now has plenty of money for this project.

And now I want to discuss a beautiful arch dam. I think the civil engineer can build nothing more beautiful than arch dams. But they really put the civil engineer and engineering geologist on the spot because the load of the water goes into that structure and then into the two sides of the valley. If the valley is not of very good rock, it may not be able to take the load without deforming. If it deforms too much, we have trouble. This dam is to be built in Greece. When I went to that area I could not believe it was Greece because I had never in my imagination figured that Greece would look like this. But this is in the northern part near Albania, and it looks a little like the Alps at times. At the site, we have limestone, sandstone, and siltstone, and -- you know what we must always suspect to be present -- our old friend, the mud seam or the shale mylonite.

They are there in thick layers. There must be 40 or 50 such seams. What became very important was the sliding resistance and deformation modulus of these seams. The beds dip down with some folding and faulting. We made a total of about 13 tunnels and 30 in-situ rock mechanics tests. The rock mechanics tests here did cost a million dollars. They went on for 9 months; there must have been 20 people involved. We believe the project is feasible. The dam will be built. This will be the second highest arch dam in Europe, and it will be built on the weakest rock that any arch dam has ever been built on. We could not have done this without an extensive engineering geology study and extensive rock mechanics in-situ testing and very extensive analysis by the structural engineers in close cooperation with the rock mechanics people and the engineering geologists.

Announcements from D. J. Hagerty, University of Louisville, at the 1974 Seminar

J. Mahar, University of Illinois, and J. A. Waddell, Martin Marietta, at the 1974 Seminar

SPEAKERS

1973 SEMINAR

C. T. Gorman and T. C. Hopkins, Kentucky Bureau of Highways, and V. P. Drnevich, University of Kentucky, authors at the 1973 Seminar

W. J. Baker, University of Detroit, at the 1973 Seminar

T. H. Wu, Ohio State University, dinner speaker at the 1973 Seminar

J. D. McNeal, Kansas State Highway Commission, at the 1973 Seminar

Y. H. Huang, University of Kentucky, at the 1973 Seminar