COMMONWEALTH OF KENTUCKY

DEPARTMENT OF TRANSPORTATION

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May 29, 1974

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MEMO TO: J. R. Harbison
State Highway Engineer
Chairman, Research Committee

SUBJECT: Research Report No. 390; "A Rock Evaluation Schema for Transportation Planning in Kentucky;"

Soil mantles and bedrock systems in Kentucky are being well defined pedologically and geologically. Agricultural soils maps, together with topographic and geological quadrangle maps, provide excellent, megascopic information for land-use guidance and site planning. In some respects, the agricultural and geologic technologies have surpassed or by-passed soil-and-rock mechanics -- that is, the engineering technology. During the past several years, the Soil Conservation Service in Kentucky has included engineering data and descriptions furnished by the Research Division in their publications. Also, U.S.G.S. notes some engineering information on the new, geological quadrangle maps. For instance, they note some aggregate sources and fossil slides and instances where highway embankments have shown a history of instability. As mentioned, engineering data on soils has been steadily accumulated during the past 25 years. It is now possible in many areas to indicate the most probable value of some properties and the expected range and variability. Rock data have not been accumulated; and, so, the purpose of the report submitted herewith is to bring into view the possibilities of beginning to assemble and catalog rock data in an orderly way for eventual reporting along with soils data. The Bureau of Highways is the principal source of engineering data for soils and produces extensive footages or rock cores from borings at bridge sites and along highway corridors. The first-stage plan would involve eventual testing and perhaps disposal of a backlog of cores now in storage and a data processing system. Ideally, the on-going plan would be a joint effort of the Research and Materials Divisions. The Division of Materials' quarry logs and test reports are unexcelled. Strength data and other engineering properties are needed. Some necessary equipment has already been acquired. A pilot project is anticipated but has not been initiated.

Respectfully submitted,

Jas. H. Havens

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Director of Research

JHH:gd
Attachment
CC's: Research Committee

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16. Abstract

The initial goal was to devise an engineering calssification system for intact rock samples based on simple index tests which could be used to categorize Kentucky surface and near-surface rock types and assist Kentucky Department of Transportation personnel in planning for transportation facilities. While conducting the literature survey, several facts become 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 from a careful study of existing classification systems and index testing procedures that developing yet another "specialized" classification system with associated index tests would not be a significant contribution. It was decided, therefore, to concentrate on development of an overall rock evaluation schema which, while useful for a specific purpose, 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 stuides and cooperation of many individuals and organizations. Such a program, properly developed and used, would substantially contribute to an advancement and a delineation of the schema and guidelines for its implementation would be a worthy goal.

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Research Report
390

A ROCK EVALUATION SCHEMA FOR TRANSPORTATION PLANNING IN KENTUCKY

KYP-64-13, HPR-1(9), Part III

by

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CHAPTER I

INTRODUCTION

STUDY RATIONALE

The occurrence of rock-related failures and(or) features affecting transportation facility planning, construction, and maintenance is a continual source of concern for highway officials. Several of these problems are illustrated in Figures 1 through 6. A critical assessment of problem areas to deduce methods for remedial action and improved design requires an extensive as well as reliable data base upon which to found such evaluation.

A first logical step in approaching rock-related problems is the development of a systematic approach to 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 most cases, overconservative. Only in rare instances are tests actually performed. Lack of a systematic approach for recording, cataloging, and storing data results in duplication of effort and loss of valuable information to the engineering community. It also contributes to the lack of communication between practitioners and those involved in research.

The International Society for Rock Mechanics (ISRM) recognized the need for standardization of testing methods and data collection. The commission on "Definition of the Most Promising Lines of Research" made the following recommendation in 1971:

> *"There is a need for a better documentation and correlation of geological and petrographic data, and corresponding mechanical property data obtained from both laboratory specimens and(or) rock masses, together with operating experience in the same rock mass or the subsequent performance of structure in the rock mass created by excavation."*

It seems reasonable that a similar line of approach be applied to data and experience collection in Kentucky.

A second step toward solutions is the development of a method of presenting collected data in a form convenient for a variety of uses. In a discussion of "Descriptive Classification of Cross Stratification" (Jacob, 1973), Spearing commented, "A classification scheme is not an end in itself, but provides the means to organize existing knowledge, enhance observations, and facilitate interpretations." Unfortunately existing classification systems alone do not embody characteristics suggested by Spearing to a degree sufficient for practical application. A method of further quantifying classification parameters is needed.

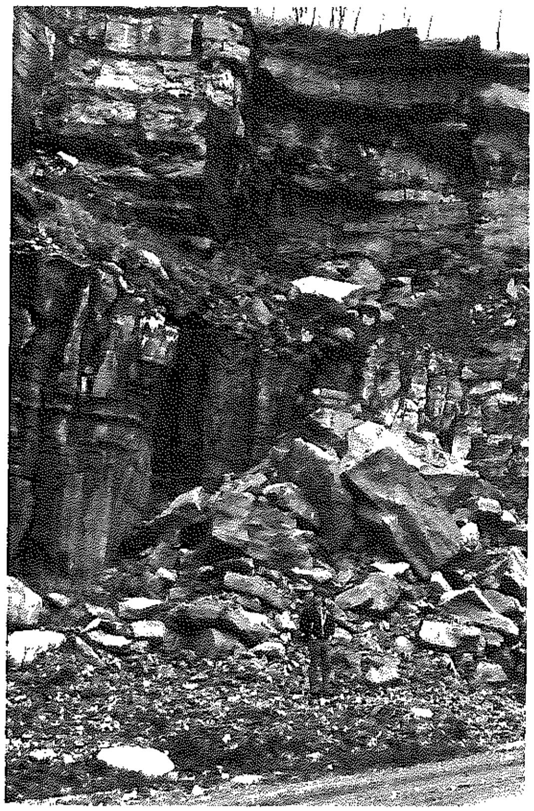
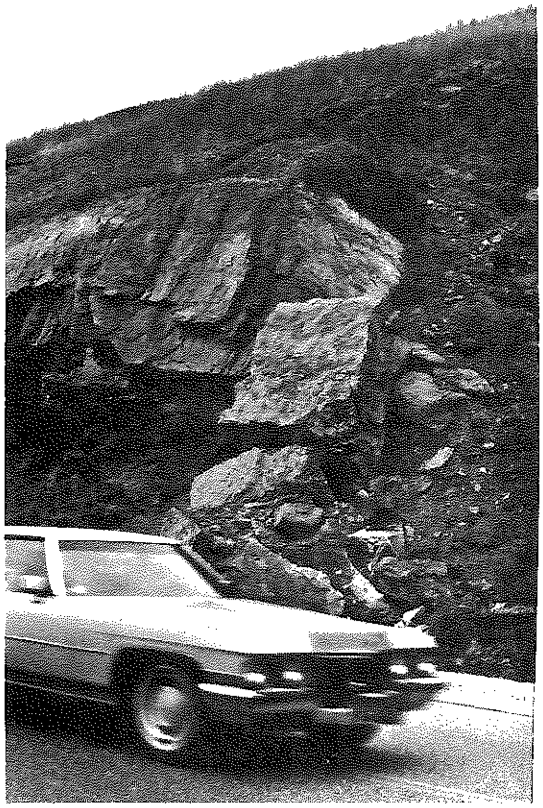


Figure 1. Poor Excavation Technique.

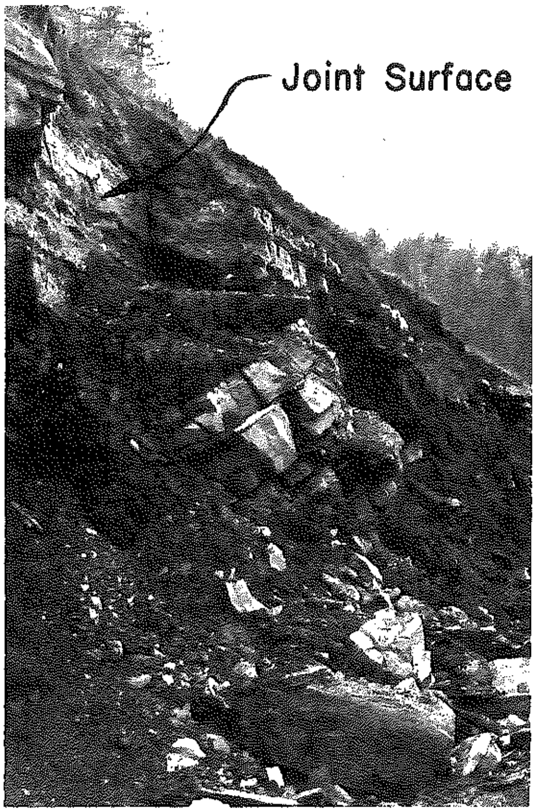
Figure 2. (a) Failure of Large Blocks along Natural Joints.
(b) Same Failure From a Different Prospective.



(a)

(b)

Figure 3. Failure along a Stress Relief Joint.



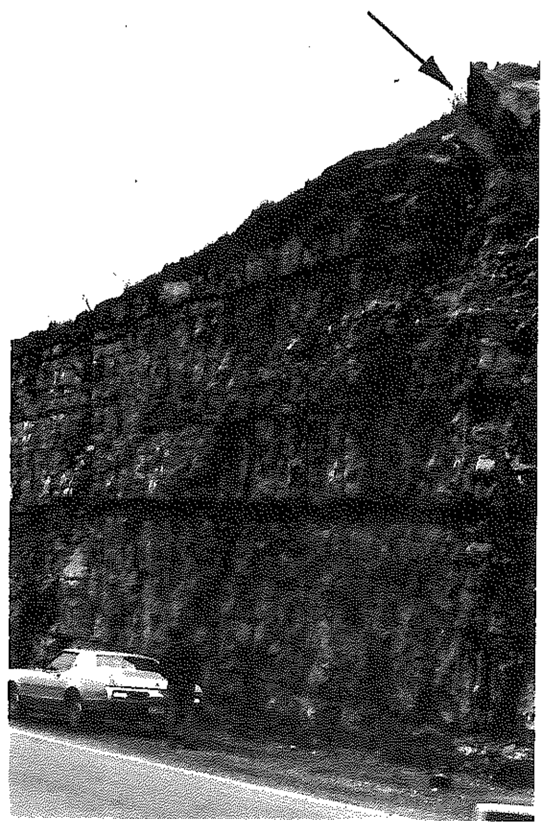


Figure 4. Potential Hazard from Outfall.

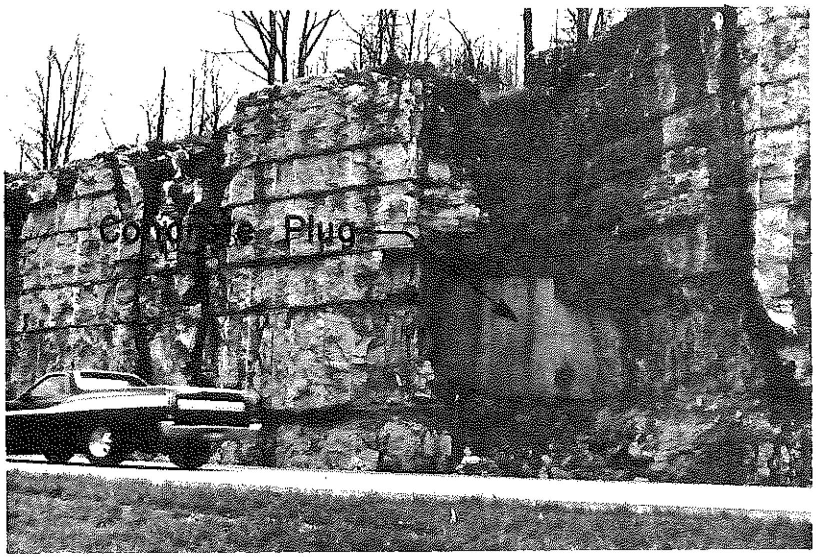


Figure 5. Failure Possibility Due to Differential Weathering.

Figure 6. Extensive Solutioning Requiring "Dental Work".

SCOPE

The task of completely delineating, testing, and implementing a rock evaluation 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 evaluation program and provide sufficient guidance for eventual implementation.

APPROACH

The formulation of a viable rock evaluation program required in-depth study. First, the subject material (rock) must 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 "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 fillers.

Having defined the subject matter, it is important to describe its variation and distribution over the area of specific concern, in this case, Kentucky. A brief summary of the geologic history, structural features, and distribution of rock types of Kentucky is presented in Chapter II.

Second, to make a critical determination of the most suitable methods for collection, storage, and use of data and experience, a comprehensive state-of-the-art review of classifications, both intact and in situ, and associated indexing parameters must be conducted. Such a study and conclusions as to the best available system are presented in Chapter III. Additional detailed information is available in the appendices.

Based on information presented in Chapter II and Chapter III, a proposed rock evaluation system has been developed; it is described in Chapter IV. Basically, it consists of two segments. The acquisition segment consisting of a test sequence, monitoring option, and data bank which permits systematic storage and convenient retrieval of rock data and experience information. It is designed to use standardized tests, where possible, to retain universal applicability and is regional only in the character of the input data. The application segment is composed of a classification system and a "use table". This table fills the void in translating test results into practical use. This segment is versatile in that several classification and use table combinations can be devised for different purposes and used interchangeably with the acquisition segment. Plans for the initiation and implementation of the program and recommendations for studies of related topics are presented in Chapter V.

CHAPTER II

GEOLOGICAL CONSIDERATIONS

INTRODUCTION

In any study involving rock, the need for a familiarity with geology is evident. The reason was well expressed by Deere (1969):

> "The role of geology is immediately clear; the materials involved are all rock masses that exist in a geological environment, or have been extracted from a geological environment. The materials possess certain physical characteristics which are a function of their mode of origin and of the subsequent geologic processes that have acted upon them. The sum total of these events in the geologic history of a given area leads to a particular **lithology**, to a particular set of **geological structures**, and to a particular **in situ state-of-stress.**"

To adequately devise a rock evaluation program which will be useful and practical in highway engineering practice, it is essential to know the location of the major structural features in a study area, the distribution of rock types, and the lithologies which have been created during the geologic history of the area. Additionally, a knowledge of local geologic nomenclature 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

a) ensure that index tests selected for classification purposes are compatible with the range of rock types to be encountered,

b) locate potential trouble areas which are associated with particular types of geologic structures,

c) identify those formations which have exhibited undesirable characteristics (i.e., swelling, solution cavaties, rapid weathering, etc.),

d) evaluate the probable in-situ stresses that have developed during geologic history, and

e) provide a means to delineate the nature and extent of the testing program to be used for a particular project at a particular site.

An abbreviated description of the geologic history, major structural features, and distribution of rock types in the region being considered provides an adequate basis from which to plan an overall rock evaluation program. However, the possibility of localized facies differences or structural anomalies cannot be overlooked. It is necessary, therefore, to obtain more detailed information early in the planning stages about sites being considered for particular projects.

While it is intended to make the methods presented in this discussion applicable, with certain modifications, to a variety of localities and purposes, the primary objective is the development of a rock evaluation program for Kentucky highway engineers. As a basis, therefore, a brief review of the geologic history, structure, and rock types is provided for the state. The material presented is based primarily on McFarlan's **Geology of Kentucky**. More recent work done by the Kentucky Geological Survey (KGS), the US Geological Survey (USGS), and others has been included where significant new interpretations, correlations, or nomenclature changes have taken place. Additional information, including a brief lithologic description of some of the more important formations, a geologic column and time table, and a glossary of geologic terms, is provided in APPENDIX A.

REVIEW OF KENTUCKY GEOLOGY

The state of Kentucky extends into three major physiographic provinces (Thornbury, 1967). The portion of Kentucky west of the Tennessee River lies in the East Gulf Costal Plain of the Atlantic Plains Province. The large central portion of the state between the Tennessee River and the Pottsville Escarpment (see Figure 7) includes divisions of the Interior Low Plateaus Province. The area east of the Pottsville Escarpment consists of divisions of the Appalachian Highlands Province.

Geologic regions of the state have been delineated so that they are approximately bounded by the outcrops of the various geologic age groups (see Figure 8). These regions provide more convenient reference areas for discussion of the many facets of Kentucky geology.

The outcrop patterns in Kentucky were established primarily by the formation (mid-Paleozoic) and subsequent erosion of a large north-south trending structural arch, the Cincinnati Arch, through the central portion of the state. Minor influences are also exerted by local structural features; e.g., Pine Mountain, various fault zones, etc. (Figures 9 and 10).

The age of outcropping formations in Kentucky varies from mid-Ordovician (exposed at the high point of the Cincinnati Arch, Jessamine Dome) to Quaternary (exposed in the Jackson Purchase region). The majority of the outcrop rocks are, however, Paleozoic. Pleistocene and Holocene alluvial deposits occur only along streams and rivers.

Sedimentary rocks dominate Kentucky's surficial geology. Only in the western Kentucky counties of Caldwell, Crittenden, and Livingston, and in Elliott County in eastern Kentucky, do igneous rocks (peridotite dikes) occur (McFarlan, 1961; Helton, 1964). Metamorphic rocks do not outcrop extensively in Kentucky (Helton, 1964). The range of competency (strength, hardness, and durability) of Kentucky rock types extends from high competency (limestones, dolomites, and sandstones) to very low competency (weakly compacted shales).

GEOLOGIC HISTORY

The present geology of Kentucky is the result of a diverse series of events. The early Paleozoic record (Lower Cambrian) indicates erosion of the Pre-Cambrian System over most of the state. Subsidence

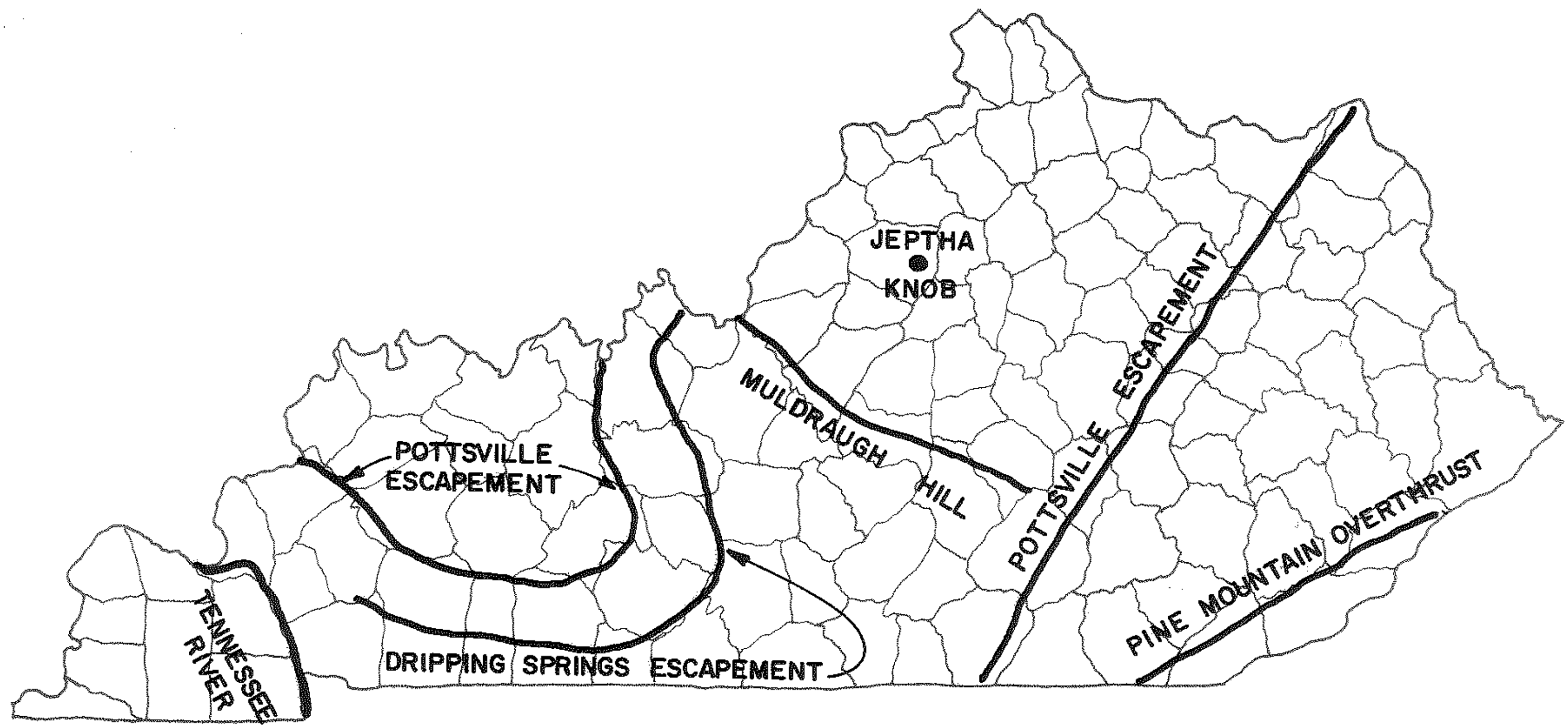


Figure 7. Major Topographic Features.

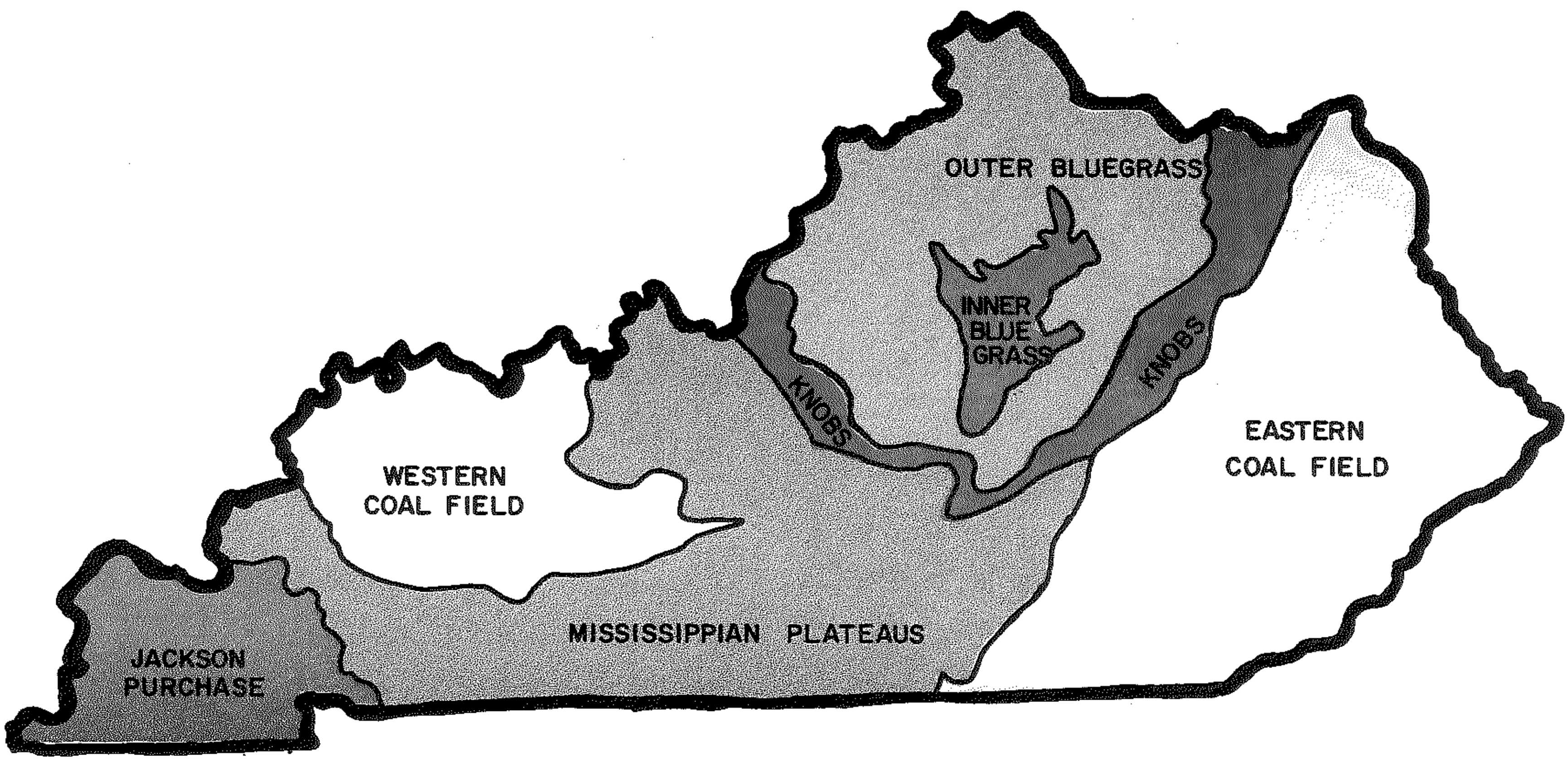


Figure 8. Physiographic Regions of Kentucky.

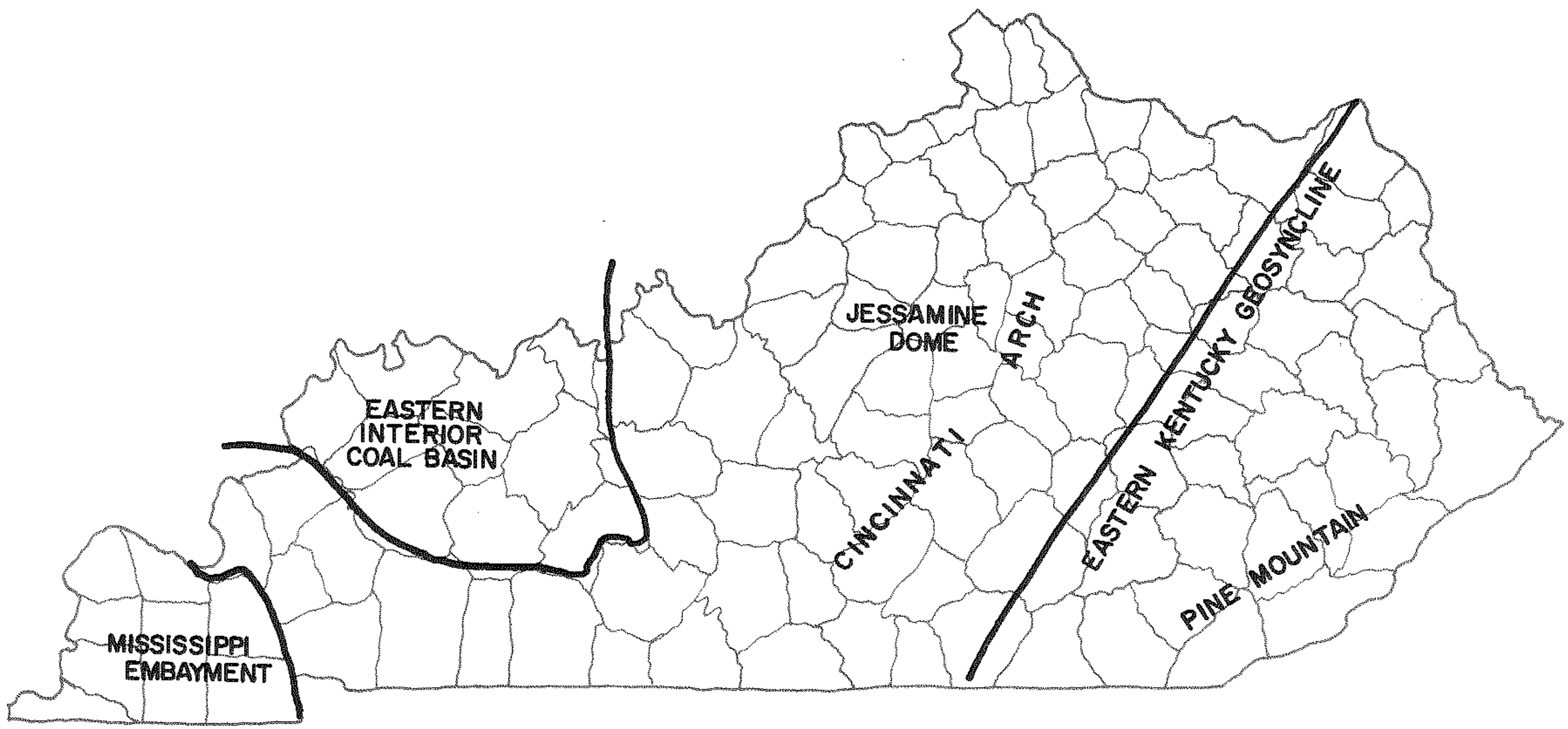


Figure 9. Major Structural Features.

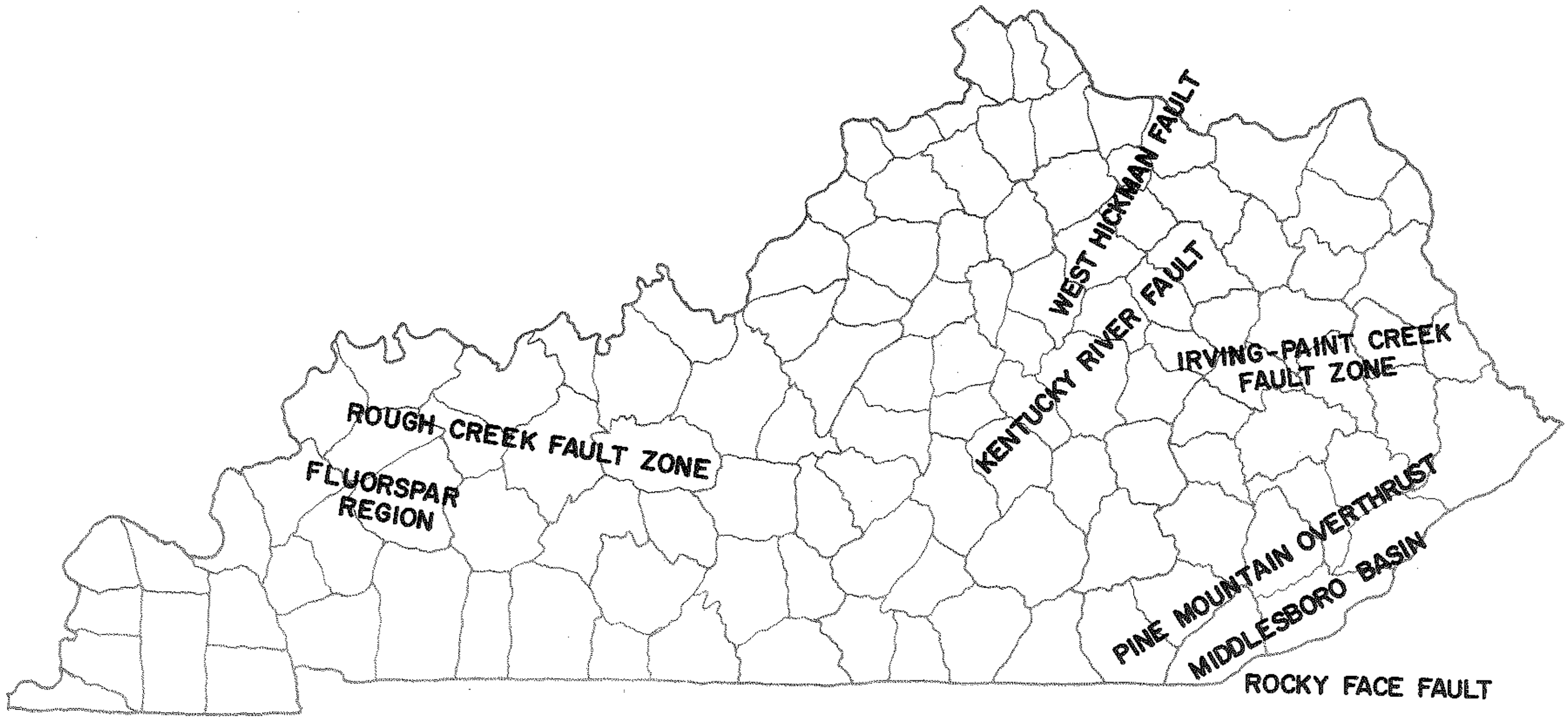


Figure 10. Major Fault Systems.

occurred during Middle and Upper Cambrian, causing submergence. The state remained submerged to receive marine lime and some mud deposits through Ordovician and Silurian times (Renfro, et al., 1970).

The Devonian Period saw the first stages of development of the Cincinnati Arch. Uplift was great enough to cause erosion of the Silurian System and part of the Ordovician System along the axis of the arch. Devonian deposition was predominantly lime west of this uplift and a mixture of lime and mud on the east (Renfro, et al., 1970).

The Mississippian Period was again a time of marine lime and mud deposition. The Mississippian-Pennsylvanian contact marks a significant hiatus in sedimentation. The unconformity evident between the two age groups indicates that the Mississippian formations were eroded significantly before the Pennsylvanian deposition. This is an indication that significant uplift had occurred. Additional evidence of this is the change from predominantly marine Mississippian to predominantly non-marine Pennsylvanian sediments (McFarlan, 1961).

The end of the Paleozoic Era was marked by the Appalachian Revolution, the time of the formation of the original Appalachian Mountains. It was during this time that the major structural features of Kentucky were formed (or completed in the case of the arch) (McFarlan, 1961). The formation of the Cincinnati Arch was augmented by the creation of minor geosynclinal structures on its flanks through subsidence of thick Pennsylvanian sediments.

The Mesozoic Era was a time of erosion over the entire state with the exception of the extreme western portion. It was during this time that the physiographic regions as they are known today were formed. Large sections of Paleozoic deposits were removed from the uplifted axis of the Cincinnati Arch. In the downwarped areas on the flanks of the arch, the Pennsylvanian deposits were preserved and appear today in the "coal fields."

Erosion in the western part of the state during the Cretaceous Period removed all post-Mississippian deposits and provided a basin for later deposition of great depths of unconsolidated materials. Additional uplift occurred early in the Cenozoic Era. The uplift was greatest in the southeastern portion of the state. This rejuvenation of crustal upsurge established the present drainage patterns in the state and caused the peneplanation of some of the weaker rock formations. The physiographic structure of the Blue Grass and Knobs regions, begun during earlier Mesozoic erosion, were completed during this uplift.

Pleistocene glaciation had little effect on Kentucky. The Illinoian ice sheet touched parts of North Central Kentucky and left drift from Oldham County to Bracken County along the Ohio River. The most significant effects produced by the glaciation were accelerated localized erosion, creation of limestone caverns, and formation of the present Ohio River in an alluvium-choked bedrock valley.

Eolian silts in Kentucky are predominantly associated with the Mississippi Loessal Uplands which extend along the east bank of the flood plains of the Mississippi River from New Orleans to the mouth of the Ohio River. These windblown silts have been deposited by the prevailing westerlies and may be as thick as 100 feet along the Mississippi River and thin out over a distance of some 40 or 50 miles east of the river. Topography of the loess in the Mississippi River Valley is distinctly hilly along the western edge where it is the deepest. Where the material becomes much thinner to the east, the surface topography assumes the character of the underlying materials which are undulating to flat. Limited areas of exposure of the windblown silt similar to that observed in the Mississippi River Valley have also been observed in the lower reaches of the Ohio River Valley. A thin surface mantle of silt-sized material is also found over extensive areas of the Western Coal Field. These deposits thin rapidly to the east and south.

IMPORTANT STRUCTURAL FEATURES

Locations of the major structural features in Kentucky, which may indicate potential problem areas in rock engineering, are indicated in Figure 9. There are many minor faults associated with the major structural systems. The location and orientation of known faults are indicated in Figure 10.

The Cincinnati Arch extends from Ohio to Tennessee through Central Kentucky. The Jessamine Dome is the high point of the arch in Kentucky. It is centered over Jessamine County. The average east-west dip of the limbs of the dome is 20 to 30 feet per mile (3.6 to 5.4 m/km). North-south dips along the axis of the arch measure about 10 feet per mile (1.8 m/km). To the east, downwarping of arch limbs has been increased by subsidence of the Pennsylvanian sediments. This area has been termed the Eastern Kentucky Geosyncline, even though it is not of the magnitude generally attributed to a geosynclinal structure. The west flank of the arch extends a great distance before an extension of the Eastern Interior Coal Basin causes additional downwarping due to subsidence.

A direct result of Appalachian Mountain building was the formation of the Pine Mountain Overthrust in extreme Southeastern Kentucky. Associated with this structure are several minor fault systems. The Middlesboro Basin, located between Pineville and Cumberland Gap, is thought to be the result of erosion accelerated by the crushing of the local rock during the formation of a major discontinuity in the area, the Rocky Face Fault.

An east-west anticline with adjacent normal faulting extends from the vicinity of Irvine to Paintsville and Martin County. This structure is associated with the Irvine-Paint Creek Fault. The Rough Creek Fault zone is another series of east-west continuities, which extend from Grayson County to Webster and Union Counties, caused by a complex structural uplift with reversed faulting accompanied by en echelon normal faulting. The amount of uplift varies from a minimum of 100 feet (30.5 m) to a maximum of 2000 feet (610 m). There are places where underlying Mississippian formations are brought back to outcrop.

In the Blue Grass region, the major fault zones are the Kentucky River Fault zone, which extends from Lincoln County to Montgomery County, and the West Hickman Fault zone, which intersects the Kentucky River Fault in Fayette County and extends northeastward to Maysville. They are both zones of en echelon normal faulting with maximum displacements of 600 feet (183 m). The fluorspar region of Caldwell, Crittenden, and Livingston Counties is an area of profuse faulting believed to be the result of igneous intrusions as evidenced by the presence of numerous peridotite dikes.

Smaller localized structural features are found in various parts of the state. For the most part, these have been noted on USGS geologic quadrangle maps. The most unusual of these is Jeptha Knob. It is suggested that this isolated hillock in Shelby County is the result of a meteor impact or an igneous intrusion (McFarlan, 1961; Seeger, 1968).

STRATIGRAPHY AND LITHOLOGY

Because of the large number of stratigraphic units of varying extent found in Kentucky, it is not practical to discuss them individually. It is necessary, however, to have an idea of the locations, names, and composition of the more important groups. A generalized geologic columnar section of the state is presented in Figure 11. Abbreviated lithologic descriptions of the important formations are presented in APPENDIX A. This section is devoted to a brief review of the names and geographic locations of the significant surface and near-surface formations. Inherent characteristics which are important in engineering are mentioned.

The geologic nomenclature used in Kentucky varies a great deal since early terms and names have been revised or eliminated. Revisions are constantly being made as more detailed work is done. The best nomenclature source at present is the indentification system used for the Kentucky Areal Geological Mapping Program conducted by USGS and KGS personnel. It is used in this report. However, since the nomenclature used by McFarlan (1961) is well known, it is also indicated in the geologic column.

It is not difficult to establish a basic understanding of Kentucky stratigraphy if the relationships among the structural features, the physiographic regions (see Figure 8), and the outcrop patterns of the geologic systems (ages) of rock are established. The geologic systems (ages) outcrop in chronological order in a more or less concentric fashion around the Jessamine Dome; the oldest outcropping formation, the mid-Ordivician High Bridge Series, is found at the summit of the dome. Boundaries of the physiographic regions coincide roughly with materials of certain age. It is convenient to describe the progressively younger

Figure 11. Major Surface and Near-Surface Geologic Formations of Kentucky.

ERA	SYSTEM	SERIES	McFARLAN'S NOMENCLATURE	FORMATION	PREDOMINATE ROCK TYPE	PHYSIOGRAPHIC REGION OF OUTCROPPING
Cenozoic	Quaternary	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 Claiborne 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	Upper	Monongahela Conemaugh	Western Coal Field: Henshaw-Dixon, Lisman Eastern Coal Field: Absent, Conemaugh (Boyd Co.)	Western Coal Field: Sandstones, Shales, and Coals; Shales, Sandstones, Limestones, Coals Eastern Coal Field: Sandstones, Shales, Limestones, Coals	Eastern and Western Coal Fields
		Middle	Allegheny	Western Coal Field: Carbondale Eastern Coal Field: Allegheny (Boyd Co.)	Western Coal Field: Shales, Sandstones, Coals, Underclays Eastern Coal Field: Sandstones, Shales, Clays	
		Lower	Pottsville	Western Coal Field: Tradewater, Caseyville Eastern Coal Field: Breathitt, Lee	Western Coal Field: Shales, Sandstones, Coals, Underclays; Sandstones Eastern Coal Field: Interbedded Shales, Sandstones, Siltstones, and Coals; Sandstones and Conglomerates	Knobs Region
	Mississippian	Chesterian – Upper	Kinkaid Degonia Clore Palestine Menard Waltersburg Vienna Tar Springs	Fluorspar Region: Kinkaid, Degonia, Clore, Palestine, Menard, Waltersburg, Vienna, Tar Springs West of Arch: Leitchfield (Buffalo Wallow) East of Arch: Pennington	Fluorspar Region: Limestones, Sandstones, Limestones, Sandstones, Limestones, Sandstones, Limestones, Sandstones West of Arch: Interbedded Shales, Limestones, and Sandstones East of Arch: Shales	Mississippian Plateaus Region
		Chesterian – Middle	Glen Dean Hardinsburg Golconda Cypress	Fluorspar Region: Glen Dean, Hardinsburg, Golconda, Cypress West of Arch: Glen Dean, Hardinsburg, Golconda East of Arch: Bangor, Hartselle	Fluorspar Region: Limestones, Sandstones, Limestones, Sandstones West of Arch: Limestones, Sandstones, Limestones, Sandstones East of Arch: Limestones, Sandstones	
		Chesterian – Lower	Paint Creek Bethel Renault Aux Vases	Fluorspar Region: Paint Creek, Bethel, Renault, Aux Vases West of Arch: Elwren, Reelsville, Sample, Beaver Bend, Paoli East of Arch: Monteagle (Newman)	Fluorspar Region: Limestones, Sandstones, Limestones, Sandstones West of Arch: Shales, Limestones, Sandstones, Limestones East of Arch: 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	Ashlock Grant Lake Grant Lake Calloway Creek Fairview	Limestones Limestones Limestones Limestones	
			Eden	Garrard Kope Clays Ferry Clays Ferry	Siltstones Shales Shales Shales	
		Middle	Cynthiana Lexington	Lexington Limestone	Limestones	Inner Blue Grass Region
			High Bridge	High Bridge	Limestones	

outcrops as they appear using the physiographic regions as references.

Inner Blue Grass -- The Inner Blue Grass region is basically an area of interbedded shales and limestones. The High Bridge Series is predominantly limestone. Overlying the High Bridge Series and surrounding its exposed portions are the limestones and shales of the Lexington Limestone Series. Again, these are predominantly limestones. Associated with Lexington limestones are minor solutioning problems. The outer margin of the region is covered by the Clays Ferry Formation (Eden), predominantly shale. Beds are thin and overall strength is poor. This shale is one of the most troublesome materials in Kentucky from a rock engineering viewpoint.

Outer Blue Grass -- Formations found in the Outer Blue Grass region range in age from Upper Ordivician at the Inner Blue Grass border to Lower Devonian near the Knobs region. This Outer Blue Grass region is basically an area of limestone and shale outcrops. The particular formations vary considerably throughout the area. The geologic columnar section in Figure 11 is typical; but locally, some formations may be absent. The outer boundary of the region crosses the geologic age systems. In areas where the Silurian and Devonian Systems predominanly consist of limestones, they are included in the Outer Blue Grass. Where they are mostly shales, they are included in the Knobs.

The area surrounding the Inner Blue Grass is a continuation of the Clays Ferry Formation (Eden). To the south and west, the Clays Ferry Formation is covered successively by varying thicknesses of the Calloway Creek, Grant Lake, and Ashlock Formations, all of which are predominantly limestones. To the north and east, the Clays Ferry gives way to the Kope Formation. The Kope also is predominantly shale and is a noted source of landslide problems (Deen and Havens, 1968). The progressively younger formations in this direction, the Fairview, the Grant Lake, and the Bull Fork, are again interbedded limestones and shales.

Along the eastern and western margins of the Outer Blue Grass, members of the Drake Formation, primarily dolomitic limestones, are present. In the west where the members primarily are limestones, the Silurian and Devonian Systems are included. The basic problems in these areas are sinkholes and solution cavities in the limestones and a tendency toward slope instability in the Silurian Osgood Formation (Deen and Havens, 1968).

Knobs -- The Knobs region is a rugged zone comprised of the erosional reminants of the shales that separate the limestones of the Mississippian Plateau on the south and the sandstones of the Cumberland Plateau on the east from the limestones of the Outer Blue Grass. In areas close to the uplands from which they have been carved, where the limestone or sandstone cap rock is still intact, the Knobs are flat-topped. Farther toward the lowland of the Blue Grass, in areas where erosion has more thoroughly dissected the formations and removed the cap rock, the Knobs assume the conical form from which they derive their name.

In the west, the geologic systems included range from Upper Devonian to Lower Mississippian. The Silurian System in this border area consists primarily of limestones and therefore is placed in the Outer Blue Grass, as are the limestones of the Lower Devonian.

The Knobs mainly consist of the Devonian New Albany (Ohio) Shale and the Mississippian Borden Formation. The cap rock in this area is St. Louis Limestone.

In the southern sector, where the Knobs region passes over the Cincinnati Arch, the Silurian System is absent. The Devonian Boyle Dolomite rests unconformably on the Ordovician Ashlock Formation. The Knobs again are formed in the New Albany Shale and members of the Borden Formation. The St. Louis Limestone, and in some areas the Salem Limestone, provides the cap rock.

The eastern sector is somewhat different. The Silurian System east of the arch is composed primarily of the members of the Crab Orchard Formation. The Devonian predominantly consists of New Albany Shale. Two additional shale formations are present under the Borden Formation. The Pennsylvanian Lee Formation unconformably overlies the Borden in this area and provides the cap rock with its resistant sandstone conglomerate.

Formations in the Knobs region which have been the source of engineering problems are the Crab Orchard and the New Providence member of the Borden Group (Deen and Havens, 1968). Slope stability problems and swelling can be expected routinely in the softer shales.

Mississippian Plateaus – The Mississippian Plateaus region is the principal locale in which the Mississippian System outcrops. This region consists of two distinct plateaus, the Pennyroyal and the Mammoth Cave, separated by the Dripping Springs Escarpment (McFarlan, 1961).

The lower plateau, the Pennyroyal, extends from Muldraugh Hill, which forms the boundary with the Knobs regions, westward to the Dripping Springs Escarpment and southward along the axis of the Cincinnati Arch. It also extends westward along the southern edge of the Dripping Springs Escarpment to the Jackson Purchase region. It is bounded on the east by the Pottsville Escarpment of the Eastern Coal Field. The Pennyroyal is typically developed on St. Louis Limestone. Along the axis of the Cincinnati Arch, however, the Ste. Genevieve and St. Louis has been removed by erosion to expose the older Salem (Warsaw) Limestone and members of the Osagian Series. The higher hills are capped with Ste. Genevieve Limestone. There is a significant facies change in the Osagian Series exposed along the arch from the predominantly shale Borden Formation in the north to the predominantly limestone Ft. Payne in the south. This change has been referred to as the Borden Front (Hagan, 1972).

Along the Cumberland River in the south, the Ft. Payne Formation has been eroded to expose the New Albany (Chattanooga) Shale and members of the Ordovician System. Characteristics of the strata in this area are much the same as those of the same formations exposed in the Knobs and Blue Grass regions.

The transition from the Pennyroyal Plateau to the Mammoth Cave Plateau is marked by Ste. Genevieve and Lower Chester Limestone knobs capped by the resistant Cypress Sandstone which forms the Dripping Springs Escarpment. Similar knobs are developed in the vicinity of the Pottsville Escarpment in the east. Engineering problems inherent in the formations of the Pennyroyal are confined to those resulting from the karst topography of the region.

The Mammouth Cave Plateau comprises the region between the Dripping Springs Escarpment and the Pottsville outcrop which surrounds the Western Coal Field. The extensive cave systems of Kentucky are located in this region near the Green River. Formations exposed in this region range from the Middle Chester, of which the Cypress Sandstone is the base, through progressively younger strata of sandstones and limestones to members of the Upper Chester. Included in the Upper Chester are several shale members and an occasional coal seam. The percentage of shale in the Upper Chester increases as the boundary of the Western Coal Field is approached. The primary engineering concerns associated with the Mammoth Cave Plateau formations are the extensive solution cavities and the low shear strengths of the underclays associated with the coal layers found in these formations.

Eastern Coal Field – The Eastern Coal Field includes all of the state east of the Pottsville Escarpment. The surface and near-surface formations range from the Lower Pennsylvanian (Lee Formation of the Pottsville Group) to members of the Conemaugh Group of the Upper Pennsylvanian. There is an exception to this along the Pine Mountain Overthrust where the Devonian (New Albany (Chattanooga) Shale) and Mississipian Systems outcrop on the fault scarp.

The massive Rockcastle Conglomerate of the Lee Formation provides the cap rock for most of the Pottsville Escarpment. To the east, the alternating sandstones, shales, and coal layers of the Breathitt Formation cover most of the region. The differential weathering associated with this combination of rock types, coupled with the uplift and accelerated erosion that occurred in the Tertiary Period, has developed the rugged terrain of the region.

The Lee Formation also provides the cap rock of the rock sequence exposed by the Pine Mountain Overthrust. The fault scarp marks the division between the Cumberland Plateau region to the north and the Cumberland Mountain section to the south. Formations in the Pine Mountain area are identical with strata of the same name found elsewhere, but because of the additional uplift they have experienced, local relief is greater than in other localities.

The main sources of engineering difficulties inherent in these formations are the low strength and durability of the underclay layers of the Breathitt Formation (Deen and Havens, 1968). The possiblity of solution cavities also exists in marginal areas, such as the Carter Caves area or the Pine Mountain area, where Mississippian limestones outcrop.

Western Coal Field -- The Western Coal Field is a topographic and structural basin in which the Pennsylvanian System is preserved in Western Kentucky. The Caseyville Sandstone provides a rugged outer rim around the region resembling the Pottsville Escarpment of the Eastern Coal Field, but on a smaller scale.

Exposed formations are progressively younger from the rim to the center of the basin. The predominantly shale of the Tradewater; the alternating sandstone, shale, and coal of the Carbondale; and the soft sandstones, weak shales, and occasional limestone of the Lisman Formation cover most of the region. The youngest Pennsylvanian formation, the Dixon, occurs only in Webster and Hopkins Counties.

Topography in the Western Coal Field is not as rugged as that of the Eastern Coal Field. This is partially because of the absence of significant subsequent uplift in the region. The soft shales generally weather to form rolling hills and broad valleys filled with thick deposits of alluvium. Ridges have formed where the massive sandstones outcrop.

The Tradewater Formation has been noted as the source of slope stability problems (Deen and Havens, 1968). This is primarily because of the soft shales and the underclays associated with coal layers found in the formation.

Jackson Purchase Region -- The Jackson Purchase Region is covered primarily with unconsolidated Mesozoic and Cenozoic deposits. On the eastern side of the Purchase, the area between the Tennessee and Cumberland Rivers is sometimes included in the region. In this area, some members of the Mississippian System are exposed in stream valleys. The ridges are carved in Cretaceous sediments. Farther west, the Mississippian System and older Paleozoics dip southward and westward toward the Reelfoot Basin in Tennessee and are covered by as much as 2000 feet (610 m) of Mesozoic and Cenozoic deposits (Schwalb, 1969). Consolidated rock masses, therefore, are not of concern in surface or near-surface engineering works in the region west of the Tennessee River.

SOURCES OF ADDITIONAL INFORMATION

Detailed geological information about an area is available from a variety of sources. The geological survey of the state in which the area is located and the United States Geological Survey are the best places to initiate a search. These offices serve as clearinghouses for an assortment of geological data. They publish lists of geologic data (literature, maps, drilling logs, well logs, etc.) available. The personnel of these organizations are, in general, well versed on the geology of the state in question and can be quite helpful.

University and municipal libraries usually have on file a large portion of the geologic maps and literature published by governmental agencies. They also have numerous publications (texts, conference proceedings, periodicals, etc.) from which information of a more general nature may be obtained. Information pertaining to the performance of particular formations can be obtained from federal, state, and local governmental agencies (U.S. Army Corps of Engineers, highway departments, utility companies, city engineers, etc.), local contractors, and consulting firms. Personnel of the geology, mining, and engineering departments of universities in the area can be of assistance in obtaining desired information and should not be overlooked.

Specific information on Kentucky geology is available through the Kentucky Geological Survey located at the University of Kentucky in Lexington. The Survey publishes a pamphlet entitled "List of Publications" listing materials available. At present, the most complete, single reference for information pertaining to a specific area is the geological quadrangle map of the region. Shown on it are the outcroping formations, lithological details, locations of known faults, and other useful information.

General information about geology can be obtained from any of a number of excellent texts published on the subject. Several of these are listed in the references.

SUMMARY

This chapter provides the geologic background necessary for the formulation of an engineering classification of rock in Kentucky. This classification will be used as part of an overall rock evaluation program for Kentucky highway engineers. The outline of Kentucky geologic history, major structural features, and stratigraphy provide an intuitive appreciation for the materials being classified. This appreciation is essential to the formulation of a competent, reasonable, and useable program. Additional information has been given to provide a guide to more detailed sources of information for use in specific projects.

The following considerations are pertinent to the development of an engineering, rock classification system:

a) the age of surface or near-surface rock outcrop in Kentucky ranges from mid-Ordovician to Pleistocene,

b) the majority of units which would be encountered in highway work are Paleozoic sedimentary rocks (i.e., limestones, sandstones, shales),

c) the range of competency of rock units extends from very competent (high strength limestones and sandstones) to very weak (compact shales), and

d) there are several regions of previous geologic disturbance which may affect the properties of the exposed rocks in those regions (i.e., areas of uplift with subsequent erosion causing stress relief, fault zones, etc.).

CHAPTER III

ROCK CLASSIFICATION AND INDEX PROPERTIES

INTRODUCTION

The term "rock mechanics" may be defined as the study of basic processes of rock behavior and their technological significance (Fairhurst, 1963). The time scale for these basic processes ranges from millions of years to microseconds, from orogenesis to blasting. The complex influence on mechanical properties include the 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. Theories abound, but the engineer remains faced with problems of building in, around, above, and through rock formations.

The essential purpose of rock classification systems is to facilitate transfer of rock engineering information from the laboratory to field operations and from both laboratory and field to design office. Testing of rock in its native environment naturally would be the best approach to determination of mechanical properties used in the design of structures. The expense of such an approach in obtaining necessary parameters is 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. Concepts of index properties and index tests encompasses these indirect determinations of significant rock mechanical properties. In testing a rock specimen in the laboratory, limits are set upon such mechanical properties as strength, deformability, weatherability, and permeability. These limits allow design parameters to be established and alert the field engineer to potential problems on the construction site.

INDEX PROPERTIES

Even the most common rock types are composites of highly variable materials: sandstones may be cemented with silica or calcite; shales may contain smectite (montmorillonite) or illite, variations in which would drastically change the shale's physical and chemical characteristics. Intact rock may be considered generally to be a solid consisting of a matrix aggregate of minerals, the properties of which are a function of mechanical properties of aggregate constituents and nature of bonding between the aggregate constituents. Intact rock may be sampled and specimens devoid of large scale structural features can be tested. However, in-situ rock masses 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 mass. Any 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 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 (Miller and Deere, 1966; Obert and Duvall, 1967; Stagg and Zienkiewicz, 1969). In addition to understanding this size problem, the practioner should be fully aware of test objectives before measuring rock properties which may or may not be relevant to the problem at hand. "If a mechanical process involves pieces of rock whose dimensions are less that those of...(a discontinuous unit)...as in crushing or grinding tests, the mechanical properties of the pieces from the rock should relate to the process under examination. On the other hand, if the investigation is concerned with phenomena occurring at a scale greater than the rock unit size, as in blasting studies or in the evaluation of rock structures, the mechanical properties of specimens cut from a unit may poorly approximate the properties of the megascopic rock" (Obert and Duvall, 1967). It is necessary to develop and use simple, inexpensive, replicable 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.

Complexities involved in even the most superficial overview of rock geognosy require extreme simplification because of physical and mathematical continuity considerations (Jaeger and Cook, 1969):

a) 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;

b) rock discontinuities and inhomogeneities play a dominant role in terms of rock deformation and failure for both intact and in-situ conditions;

c) "constants" incorporated within simplified mathematical models are statistical functions of these discontinuities and heterogeneities; and

d) 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.

INDEX PROPERTY TESTS

Unfortunately, except in certain specialized applications, there are no standards to guide the rock engineer in selecting appropriate indicator tests. "It is dangerous to regard a particular test as suitable because of long-established usage, or as irrelevant because of novelty, since rock classification procedures are at an early stage in their development" (Cottiss, Dowell, and Franklin, 1971). Of course, classification tests should be chosen so that, regardless of geologic origin, specimens with similar index properties should exhibit similar mechanical behavior (Peck, Hanson, and Thornburn, 1953). 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" providing information for design. Design tests 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 (Pomeroy, 1957; Deere, 1963):

a) the test property must be an index of a material (mechanical) property which the design engineer can use effectively;

b) the test should be simple, inexpensive, and rapidly performed (minimum sample preparation); and

c) 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 an index property which would delineate "rock" from "soil" or "rock-like" from "soil-like" materials.

The variety of index properties relevant to the mechanical quality of rock masses include (McMahon, 1968; Cottiss et al., 1971; Mesri and Gibala, 1972):

anisotropy
apparent specific gravity
brittleness
brokenness
core recovery
deformation modulus
relative absorption
residual shear strength
resilience
secant modulus
slake durability
swelling

degree of alteration
dilatational wave velocity
fracture frequency
hardness (rebound and indentation)
joint extension
modified core recovery (RQD)
moisture content
Poisson's ratio
porosity
tangent modulus
tensile strength
toughness
uniaxial compressive strength
unit weight
void index
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, for instance. Packer pumping tests, in which zones in a borehole are isolated by means of expanding "packers" and water under pressure is applied to the isolated rock zones, are extremely useful in preparing permeability logs for grout-take evaluation and drainage; borehole sonic velocity, electrical resistivity, and gamma ray emission logs are useful for stratigraphic and mechanical correlations.

As Morgenstern (1969) indicated, "Either local or overall displacements limit the utility of the engineering structure and are therefore the fundamental design criteria". It is apparent, therefore, that index tests and(or) properties that are indicative of compressibility or displacements should be included in classification systems. However, measures of deformation moduli or mass compressibilities are extremely difficult to obtain and involve serious complexities which are yet to be resolved. For instance, it would be necessary to know, to some degree, the initial state of stress in a rock sample to evaluate its response to imposed stresses during index testing or, in fact, construction processes.

There are three basic approaches to the development of a rock classification system based on inherent rock characteristics; geologic designations, physical characterists of intact samples, and gross characteristics of the in-situ mass.

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:

a) igneous rocks -- rocks formed by cooling of molten magmas or by the recrystallization of older rocks under the action of heat and pressure of such magnitude as to render them fluid;
b) sedimentary rocks -- rocks formed as products of deposition of plant and animal remains, from materials formed by chemical decomposition, and fromproducts of the physical disintegration of pre-existing rocks; and

c) 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 (Pirsson and Knopf, 1926). 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 (Wahlstrom, 1973).

Classifications of sedimentary rocks notably group the rocks into origin, texture, and particle size or composition categories (Wentworth, 1922; Putnam, 1964; Leet and Judson, 1971; Wahlstrom, 1973); 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 constitutents -- 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 properites in terms of a classification system are texture, structure, and mineralogical composition. "In an indirect way, the magnitude of strength and the nature of deformation properties can be deduced from such analysis" (Coates, 1964). 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 (Spock, 1953). Structure includes distribution and grouping of minerals, which are constituents of rock (Huang, 1962). As Franklin (1970) suggested, 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 (Coates, 1970). Megascopic fabrics in rocks also have been classified with respect to isotropy and anisotropy (Wahlstrom, 1973); 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 in percent by weight. It should be noted, however, it is impossible to determine 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 (Spock, 1953). Chemical classifications may be of little use to engineers interested only in physical rock properties.

All of these descriptive indicators -- genesis, petrography, texture, mineralogy, and chemical composition -- give only vague information concerning the engineering behavior and capabilities of the rock. Limestones may vary in compressive strength from 6,000 psi (41 MPa) to 36,000 psi (248 MPa). Granite, although geologically a hard coherent rock, is also extremely variable in strength from location to location depending upon environmental conditions to which it has been subjected. Sandstones may vary in compressive strength from less than 5,000 psi (34 MPa) to over 30,000 psi (206 MPa). Geologic classification systems do not give comprehensive information as to rock properties in terms of mechanical behavior of the in-situ rock masses. Much detailed information obtained from geologic studies is not suitable for classification purposes.

Geological mapping, for the most part, is based upon rock classification systems incorporating geologic observations which reflect the genesis of rock instead of rock engineering properties or mechanical characteristics. 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.

In many regions, the topographic relief is sufficiently characteristic to be indicative of the geology of the bedrock, even though very few rock exposures may be present. Wahlstrom (1973) has presented a classification of landforms as they relate to errosional or depositional history and subsurface geology. Utilizing aerial photographs, topographic maps, and drainage patterns, an assessment can be made of subsurface geology and the structure of bedrock. Brink and Partridge (1967) have devised a system of classification in which "land systems" are defined on the basis of a limited number of constituent facets (mapping units) which occur in specific combinations. After defining a land system, data for any recurrent facet within it can be stored and readily retrieved. The authors refer to the Kyalami Land System in terms of slope form (quantitative), description of soils, materials and hydrology, tone, relative texture, structural pattern, steroscopic appearance, and associated characteristics. The physiographic classification of terrain data of Brink and Partridge has proven to be a great aid in location, planning, design, and construction of roads in the Kyalami Land System north of Johannesburg, South Africa.

An interesting exception to the qualitative approach of most geological mapping surveys is the Pattern-Unit-Component-Evaluation (P.U.C.E.) by which a methodology of terrain description and quantification has been introduced and applied to a region of more than 200,000 square miles (518

Gm^2) in Australia (Aitchison and Grant, 1967). 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 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 ususally both visual observations and mechanical tests are required to provide data for design calculations. Significant mechanical properties of both the rock material and the rock mass must be recognized and the appropriate information obtained to specify an initial appraisal of potential problems (Coates, 1964). 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. A general summary of the most widely known intact sample classification systems appears as Figure 12. Contents of this summary are further described in APPENDIX C, INTACT SAMPLE CLASSIFICATION SYSTEMS and APPENDIX E, CORRELATION PARAMETERS. There exists a certain amount of overlap in terms of the parameters designated. For example, texture, toughness, and hardness are sometimes difficult to distinguish within a classification system; rock durability is measured by both swelling and slake tests. Also, there are parameters or indices which may be required to categorize a rock specimen; for example, Miller and Deere (1966) used unit weight to differentiate rocks with the same range of hardness values. But unit weight was not a category in their engineering rock classification system, instead it was an index test.

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

a) strength,

b) deformability or pre-failure deformation characteristics,

c) lithology,

d) gross heterogeneity or anisotropy,

e) durability or failure characteristics, and

f) 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 (see APPENDIX E), should include the following properties: strength (tensile), lithology, specimen anisotropy, and durability.

Tensile Strength – Since, obviously, 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 (see Figure 12), 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 (Protodyakonov, 1960; Hobbs, 1963; Hobbs, 1968; Reichmuth, 1968; Franklin, 1970; Broch, 1970). The point load strength index, I_s, as standardized by the ISRM (Franklin, 1972) provides a measure of tensile strength, and empirical results show excellent correlation between this index and unconfined compression strength (Franklin, Broch, and Walton, 1971).

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 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.

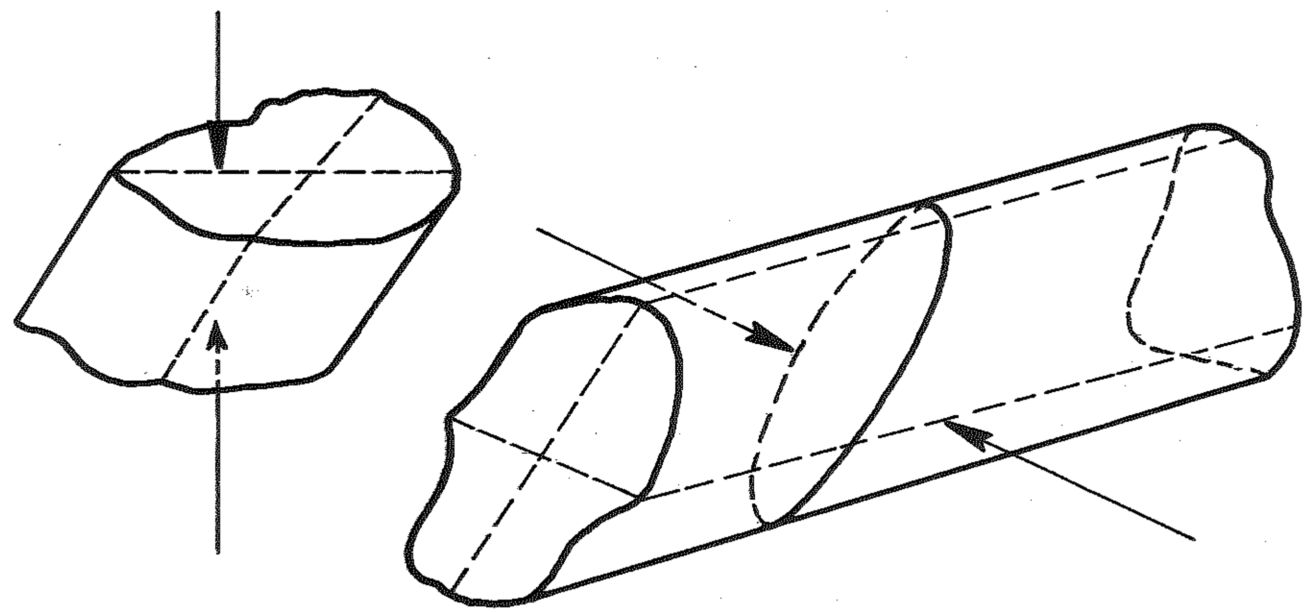
Specimen Anisotropy -- In general, most rock is anisotropic (measured mechanical properties are a function of specimen orientation). Most elastic sedimentary rocks are slightly to strongly anisotropic in such mechanical properties as thermal conductivity, velocity of elastic waves, electrical conductivity, and fluid permeability. Permeability, which has been reported to be the most sensitive indicator of relative anisotropy (Somerton, Masonheimer, and Singhal, 1970) and the point load test has been applied successfully in the logging of cores (Franklin, Broch, and Walton, 1971). The point load test is used to define the "strength anisotropy index", I_a, as the ratio between the maximum and minimum strength indices (see Figure 13). Figure 13 shows how the load should be applied in relation to the planes of weakness; first, diametrically parallel to the planes of weakness, and second, perpendicular to these planes. Whenever possible, the diametrical test is arranged to break the core into discs of equal length and diameter (the optimum shape for axial testing).

Durability -- Durability refers to the extent (variation) of alteration a rock will exhibit under various environmental conditions. Short-term weathering of rock has been measured with various degrees of success

INTACT SAMPLE CLASSIFICATION SYSTEM	PARAMETERS IN SYSTEMS																					
	ANISOTROPY	LITHOLOGY	SLAKE DURABILITY	TENSILE STRENGTH	COMPRESSIVE STRENGTH	DENSITY	DRILLABILITY	DRY SPECIFIC GRAVITY	FAILURE CHARACTERISTICS	HARDNESS	HYSTERESIS	MOISTURE CONTENT	PETROFABRICS*	POROSITY	SEISMIC VELOCITY	SHEAR	SWELLING	TANGENT MODULUS	TEXTURE	TOUGHNESS	UNIT WEIGHT	WEATHERABILITY
Woolfe, 1951		q								X			q						q			
Harley, 1926		q					X			q										q	X	
Head, 1951							X															
Panama Canal Company, 1959										q												
Wegehaupt, 1960		q			X								X									
Rollow, 1962							X															
Kasmanovic and Lancof, 1962		X														X						
Coates, 1964		Xq			X				q		X											
Miller and Deere, 1966					X													X				
Coates and Parsons, 1966		q			X				X		X											
Obert and Duvall, 1967		q									q		q									
Stapledon, 1968		q			X																	
Duncan and Jennings, 1968					X					Xq												
Duncan, 1969		Xq						X				X	q				X		q			
Coates, 1970		Xq			X						X											
Franklin, 1970		q	X	X		X				q			Xq	X					q			X
van der Vlis, 1970		q								X												
Cottiss, Dowell, and Franklin, 1971			X	X				X		X				X	X							X

Notes: * -- Includes mineralogy; q -- Qualitative; X -- Quantitative

Figure 12. Summary of Intact Sample Rock Classification Systems.



(A) MAXIMUM STRENGTH INDEX

(B) MINIMUM STRENGTH INDEX

Figure 13. Anisotropy Testing.

by such tests as the Los Angeles Rattler Test (California State Standards 2ll-C), the Durability Index Test (California State Standards 229-E), Sodium Sulfate Soundness Test (California State Standards 214-D), Durability Absorption Ratio, Slake Test, and Swelling Test (Smith, McCauley, and Mearns, 1963; Cottiss et al., 1971). Rock has been classified using degree of weathering (Hamrol, 1961) and degree of competency in terms of support necessary for safe tunnel design (Wahlstrom, 1973).

Probably the best methods for a measure of durability from an engineering standpoint are the swell test and(or) the slake-durability test (Franklin, 1972). With regard to transitional material (see APPENDIX B), the swell test should be performed first to eliminate as much of the ' soil-like" material as possible from rock testing as soon as possible. Thereafter, a slake-durability test should be performed on the "rock-like" material. Franklin and Chandra (1971) have classified material into six groups according to slake-test results (see Figure 14). Unequal subdivisions of durability are used for the more durable rocks since most rocks have "extremely high" slake durability; thus smaller subdivisions are needed to reflect differences in resistance to breakdown.

The intact sample classification system (see Figure 15) segment of the proposed rock evaluation schema is very similar to the system of core logging presented by Franklin, Broach, and Walton (1971). Tentative values for indices have been indicated to quantify expected ranges. As will be discussed in Chapter IV, ranges of the indices should be determined for specific use; that is, values for a weak rock would be different for a highway classification system as opposed to a tunnel classification system (which would also require slightly different indices to measure rock quality). The scope of tests which have been standardized for rock classification and characterization (see Figure 16) include those for intact sample and in-situ systems. The ISRM has also indicated a series of possible engineering design tests. Whenever possible, it is advisable to utilize these tests to obtain values and nomenclature.

There still remains the problem of soil-rock differentiation. At the very least, this differentiation is important in terms of laboratory procedures. Several methods for separating compacted (soil-like) materials from cemented (rock-like) materials have been published. Both Duncan (1969a) and Jaeger (1972) proposed using a free swell test (see APPENDIX C). Duncan (1969a) also suggested a plot of dry apparent specific gravity versus saturation moisture content (i_s percent, log scale). This graph (see Figure 17) would delineate weak rock and soil materials from "rock-like" cemented and compact rock materials. Stapledon (1968) offered 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. This approach is susceptible to operator bias. While studying the stability of natural slopes, Skempton and Hutchinson (1969) proposed that "clay-shales" or "hard shales" have an undrained shear strength above 4,000 psf (192 MPa) and material with strengths below that value be considered soil or "soil-like". The

SLAKE DURABILITY I_d (%)	CLASSIFICATION
0 - 25	Very Low } Soil
25 - 50	Low } Soil
50 - 75	Medium } Transitional Material
75 - 90	High } Transitional Material
90 - 95	Very High } Rock
95 - 100	Extremely High } Rock

Figure 14. Slake Durability Classification.

TENSILE STRENGTH		LITHOLOGY		ANISOTROPY	DURABILITY
POINT LOAD INDEX, I_s				STRENGTH ANISOTROPY INDEX, I_a $I_a = \frac{\text{Maximum Strength}}{\text{Minimum Strength}}$	SLAKE DURABILITY INDEX, I_d PERCENT SLAKE
CLASS	MPa				
Very Strong (1)	> 10	SS	Sandstone	1.0 - 1.2 (1) Isotropic	< 5 (1) Very Durable
Strong (2)	3 - 10			1.2 - 1.5 (2) Slightly Anisotropic	5 - 10 (2) Durable
Medium (3)	1 - 3	SH	Shale	1.5 - 5 (3) Moderately Anisotropic	10 - 25 (3) Moderately Altered
Weak (4)	0.3 - 1			5 - 20 (4) Anisotropic	25 - 50 (4) Highly Altered
Very Weak (5)	< 0.3	LS	Limestone	> 20 (5) Very Anisotropic	50 - 100 (5) Decomposed

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

Figure 15. Intact Sample Classification System.

Category 1 -- Classification and characterization

A Intact rock material

(1) Density, water content, porosity, absorption
(2). Strength and deformability modulus in uniaxial compression; point load strength
(3) Hardness abrasion, attrition, and drillability (Schmidt hardness, Shore scleroscope, indentation, Los Angeles test, Deval test, etc.)
(4) Swelling and slake durability
(5) Sound velocity; pulse and resonance (Lab)
(6) Permeability (Lab)
(7) Micro-petrographic description for engineering purposes (emphasis on mechanically important features)
(8) Anisotropy indexes

B In-situ mass

(1) Joint systems; orientation, spacing, openness, roughness geometry, filling, and alteration
(2) Core recovery rock quality designation and fracture spacing
(3) Seismic tests for mapping and for rock quality index purposes
(4) Geophysical logging of boreholes

Category 2 -- Engineering design tests

A Laboratory tests

(1) Determination of strength envelope and elastic properties (triaxial, biaxial, and uniaxial compression and tensile tests)(direct shear tests)
(2) Strength of joints and planes of weakness
(3) Time dependent and plastic properites

B In-situ tests

(1) Deformability tests
(2) Direct shear tests (intact material, joints, rock-concrete interface)
(3) Field permeability, piezometric levels, and ground-water flow
(4) Stress measurements
(5) Rock movement monitoring; rock noise monitoring; blast and groundmotions monitoring
(6) Uniaxial, biaxial, and triaxial compressive strength

Category 3 -- Research

It was decided that research tests, including many of the rock physics tests are beyond the scope of standardization.

Figure 16. Standard Tests for Classification and Characterization.

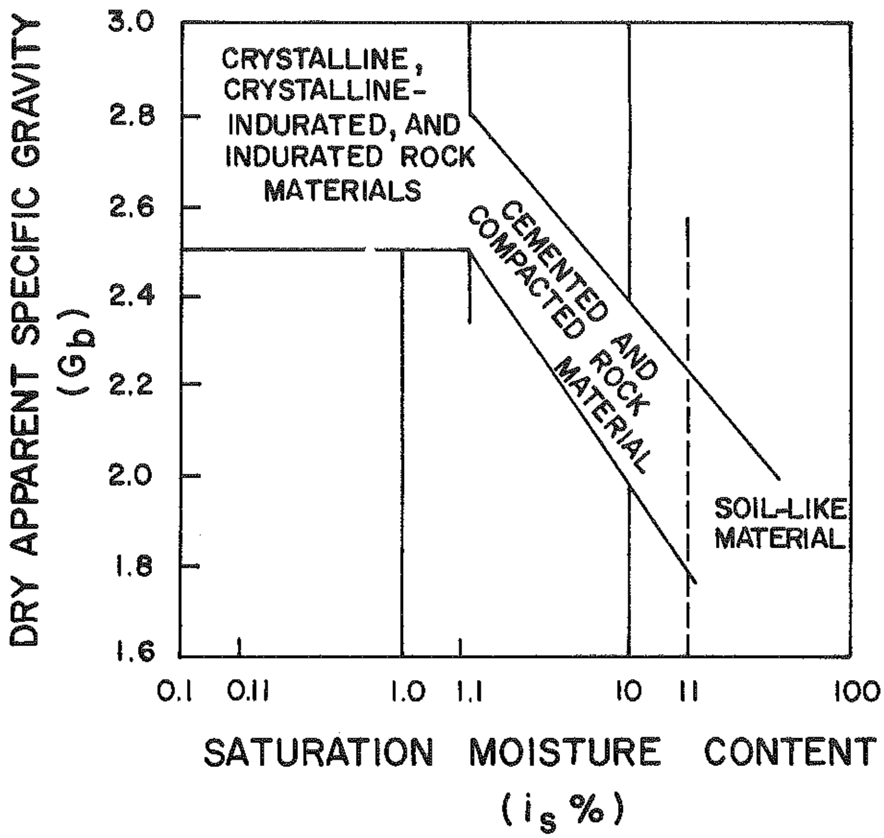


Figure 17. Soil-Rock Differentiation.

use of wet-dry cyclic weathering to distinguish among transitional materials has been proposed by many investigators (Philbrick, 1950; Underwood, 1967; Fleming, Spencer, and Banks, 1970). Thus far, the best method of soil-rock differentiation appears to be Gamble's (1971) durability-plasticity classification (see APPENDIX B).

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 rock state and second through prediction of the consequences of anthropogenic activities which may occur (Jovanovic, 1970), 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 (Franklin, 1970).

In-Situ Classification Systems

Significant engineering properties 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, packer pumping tests, and geophysical tests). Exploratory tests measure properties of rock which can be related to engineering properties (Coon, 1968). These correlations are the basis for an engineering classification of in-situ rock.

A brief survey of in-situ classification systems (see Figure 18) revealed several interesting facts:

a) there are relatively few general in-situ classification systems;

b) 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;

c) major concerns in existing systems have been rock quality (bedding character, joint frequency, and weathering or alteration), lithology, deformation characteristics, and velocity ratio;

d) some systems utilize laboratory measurements on intact specimens such as unconfined uniaxial compression strength, static modulus, and static sonic velocity;

e) in-situ tests utilized to a significant degree included seismic velocity, plate jacking, permeability, modified RQD, and borehole analysis tests.

SYSTEM	ROCK QUALITY: BEDDING CHARACTER	ROCK QUALITY: JOINT FREQUENCY	ROCK QUALITY: WEATHERABILITY OR ALTERATION	LITHOLOGY	DEFORMATION CHARACTERISTICS	VELOCITY RATIO	INTACT SAMPLE TESTS	IN-SITU TESTS	ENGINEERING PROPERTIES	PURPOSE OF CLASSIFICATION
Terzaghi, 1946	q	q	q							Tunnelling
Talobre, 1957			q							Weathering
U. S. Bureau of Mines, 1962	q				q					Underground Openings
John, 1962		X	q				Uniaxial Compression			General
Onodera, 1962	q	q	q	q		X	Sonic	Seismic		Dam Foundations
Deere, 1963	XB	XB	qB	qB						
Lane, 1964		XB		qB		X	Saturated Sonic	Seismic Plate Jacking		Site Investigation
Deere, 1964	qB	XB*	qB	qB						General
Coates, 1964	q	X			q		Uniaxial Compression			General
Knill and Jones (gneiss), 1965			XB	qB				Permeability	Slope Stability Powder Factor	Blasting (Dam)
Knill and Jones (shale), 1965	qB	qB		qB					Slope Stability	Blasting (Dam)
Deere, Hendron, Patton, and Cording, 1966		XB*	XB*			X	Static Modulus	Plate Jacking		General
Ege, 1967		XB*					Sonic	Seismic		Borehole
Obert and Duvall, 1967	qB	qB		q	q					Underground Openings
Scott and Carroll, 1967	q	X	X	q				Seismic		Tunnelling
Merritt, 1968		XB*	XB*			X	Sonic	Seismic		
+Franklin, Broch, and Walton, 1971	q	XB	Xq	q			Point Loading Slake			Blasting (Excavation)

Notes: X – Quantitative; q – Qualitative; * – RQD; B – Information obtained from core; + – Not included in APPENDIX D

Figure 18. Summary of In-Situ Rock Classification Systems.

Strength and deformation characteristics 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 at a potential construction site, 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.

Properties of interest in the proposed engineering in-situ rock classification system include strength, deformability, lithology, gross heterogeneity, durability, and rock continuity (which is related both to strength and deformability). A synthesis of these properties appears in Figure 19 as an in-situ rock classification system. Strength, deformability, and continuity appear together and are described by bedding spacing, joint spacing, joint frequency, and infiltration material. Lithology is retained from the intact sample rock classification system once again to obtain a "feeling" for the mass and to gain a geologic appreciation of the formation. Gross heterogeneity is measured by the mass permeability, the property most sensitive to anisotropic influences. The last parameter, the "intact - in-situ correlation" allows for a relatively easy connection between intact and in-situ strengths. In particular, the velocity ratio is a function of in-situ rock quality and intact homogeneity.

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: constituent rock material, joints and faults as potential planes of structural weakness, and environmental conditions before, during, and after project 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 the procedure by which data are collected to construct the rock mass model (Duncan, 1969b). The description of joints in terms of joint classification, degree of continuity, orientation, and surface description and the use of such techniques as impressographs, coefficient of joint volume decrease, and joint log sheets are all beyond the scope of this research; but the concepts of joint surveys should be subject to implementation with a rock evaluation program.

SUMMARY

In any practical investigation in rock mechanics, the first stage is a geologic and geophysical investigation. The outcome of such an investigation is to establish the lithology and boundaries of the rock types involved. The second stage is establishment of the detailed pattern of discontinuities by means of drilling or exploratory excavations. Also, at this stage, mechanical and petrological properties of rocks

STRENGTH AND DEFORMABILITY ROCK QUALITY (CONTINUITY)								LITHOLOGY	GROSS HETEROGENEITY		*INTACT - IN-SITU STRENGTH CORRELATION REDUCTION FACTOR	
BEDDING (THICKNESS) CHARACTERISTICS		JOINT SPACING		JOINT FREQUENCY (PER 3 METERS)		*JOINT INFILTRATION MATERIAL			PERMEABILITY ($x\ 10^{-5}$ cm/sec)		VELOCITY RATIO	
Class	cm	Class	cm	Class		Filler	Code No.		Class	k	Degree of Correlation	Range
Very Thin	< 1	Very Close	< 1	Very Low	< 1	Air	1	SS Sandstone	Very Low	< 0.01	Excellent	0.8 - 1
Thin	1 - 5	Close	1 - 5	Low	1 - 3	Water	2		Low	0.01 - 0.10	Good	0.6 - 0.8
Medium	5 - 30	Moderately Close	5 - 30	Medium	3 - 5	Cohesionless Soil	3	SH Shale	Medium	0.10 - 1.0	Fair	0.4 - 0.6
Thick	30 - 150	Wide	30 - 150	High	5 - 10	Inactive Clays	4	LS Limestone	High	1 - 10	Poor	0.2 - 0.4
Very Thick	> 150	Very Wide	> 150	Very High	> 10	Active Clays	5		Very High	> 10	Very Poor	0.0 - 0.2

*Use of these parameters for reduction of intact strength subject to modification with further testing.

Figure 19. In-Situ Rock Classification System.

are determined from intact samples. If possible within the framework of a construction project, the final stage would be to measure stresses present in the unexcavated rock (Jaeger and Cook, 1969). A major goal in rock mechanics is to develop procedures that will permit accurate evaluation of the mechanical properties of rock so that the scientific community can make quantitative predictions concerning the response of these mechanical properties to variable forces, both natural and man-made (Rock-Mechanics Research, 1966; Franklin, 1972).

CHAPTER IV

PROPOSED ROCK EVALUATION SCHEMA

INTRODUCTION

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.

In terms of transportation facility planning, a program must provide highway 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.

PROPOSED SCHEMA

The proposed rock evaluation schema consists of two segments (see Figure 20). 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 (i.e., transportation facility planning). 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.

Computer programming will be used to facilitate storage, retrieval, and use of acquired information. The exact programming format must be developed in subsequent research. Herein the evaluation schema is discussed in descriptive terms only.

ACQUISITION SEGMENT

Data Bank Format

The data bank consists of a system of computer files arranged in three categories which allow systematic storage and convenient retrieval of accumulated information (see Figure 21). Category 1 contains information pertinent to the location, identification, and natural environment from which the

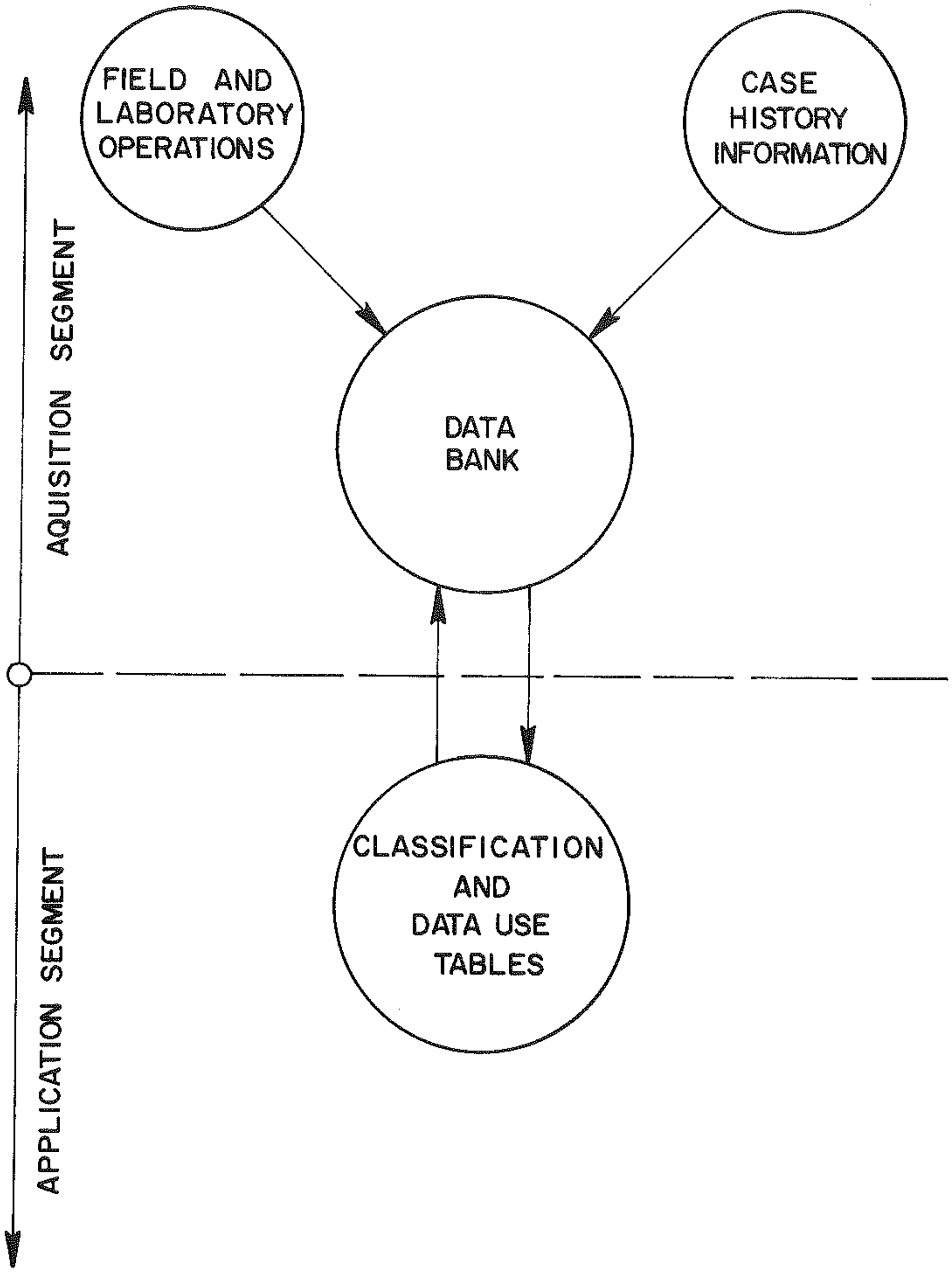


Figure 20. Schematic Diagram of the Proposed Rock Evaluation Program.

LOCATION SAMPLE IDENTIFICATION FORMATION ROCK TYPE GROUND ELEVATION SAMPLE ELEVATION WATER TABLE ELEVATION SAMPLE ORIENTATION			CATEGORY 1
COLOR TEXTURE MINERALOGY GRAIN SIZE STRUCTURE	VISUAL	INTACT	CATEGORY 2
FREE SWELL SLAKE DURABILITY POINT LOAD STRENGTH SONIC VELOCITY NATURAL MOISTURE CONTENT SATURATION MOISTURE CONTENT DRY APPARENT SPECIFIC GRAVITY UNIT WEIGHT POROSITY TANGENT MODULUS (E_{50})	INDEXING	INTACT	CATEGORY 2
DIRECT SHEAR TRIAXIAL SHEAR	SPECIFIC	INTACT	CATEGORY 2
JOINT SPACING BEDDING THICKNESS JOINT FREQUENCY INFILTRATION MATERIAL	VISUAL	IN SITU	CATEGORY 2
SEISMIC VELOCITY MODIFIED RQD PERMEABILITY (PACKER PUMPING TEST)	INDEXING	IN SITU	CATEGORY 2
PLATE JACKING TEST PRESSURE TUNNEL FIELD SHEAR	LARGE SCALE	IN SITU	CATEGORY 2
PREVIOUS EXPERIENCE CONSTRUCTION PERFORMANCE MONITORING			CATEGORY 3

Figure 21. Data Bank Format.

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 space for case history reports of previous experience, contemporary construction experience, and information to be derived from rock monitoring programs.

Field and Laboratory Operations

There is a degree of overlap between field and laboratory methods used to obtain data for Categories 1 and 2 of the data bank. It is therefore desirable to discuss both simultaneously. Information for Category 1 is acquired in the field. A sample identification sheet (see Figure 22) has been prepared to illustrate information required. Exact methods of sample selection, acquisition, preparation, and testing will be the subject of a separate study. Sample selection will be based on geological considerations (Chapter II) and availability. Based on the literature and preliminary work by the authors, the most suitable samples appear to be NX size cores or blocks from which cores may be obtained in the laboratory.

Ideally, samples should be tested at the site immediately after removal from the core barrel (Franklin, Broch, and Walton, 1971). 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 moisture 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. Additional tests and observations as indicated in the visual and indexing sections of the intact and in-situ portions of Category 2 are performed. Parameters obtained from these observations and tests were selected based on considerations presented in Chapter III.

More refined laboratory (direct shear, triaxial, etc.) or large scale in-situ (packer 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, solutions, 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

SAMPLE IDENTIFICATION SHEET

1. Sample Location ______________________________

2. Sample I. D. ______________________________

3. Geological Formation from which Sample Was Taken ______

____

4. Rock Type ______________________________

5. Ground Elevation ______________________________
   ☐ measured ☐ estimated

6. Elevation of Sample ______________________________
   ☐ measured ☐ estimated from ground surface

7. Elevation of Water Table ______________________________
   ☐ measured ☐ estimated from ground surface

8. Orientation of Sample with respect to Ground Surface
   ☐ G.S. ☐ G.S. ☐ G.S. θ= ____

9. Orientation of Sample with respect to Major Bedding Planes
   ☐ B.P. ☐ B.P. ☐ B.P. θ= ____

10. Method Used to Obtain Sample
    ☐ NX Core ☐ Block ☐ Quarry sawn
    ☐ Loosened with hand tools
    ☐ Other -- explain

____

11. Comments ______________________________

____

____

INSTRUCTIONS

1. List sample location by county and quadrangle number.
2. Sample I. D. will be quadrangle coordinates followed by sequential numbers for each site.
3. Enter the geological formation name, if known. If questionable, follow name with a question mark. If unknown, leave blank.
4. Generic term (i.e. limestone, sandstone, shale, granite, etc.).
5. Indicate elevation to nearest foot. Mark whether measured or estimated from a map.
6. Indicate sample elevation to nearest foot. Mark whether measured from ground surface or estimated.
7. Indicate water elevation, if determinable.

8 - 9. Sample should be marked with a vertical arrow ( ) to indicate the top surface. Mark the appropriate block which relates this arrow to the surface in question. If on skew, indicate the approximate angle.

10. Check proper box. If other, explain briefly.
11. Include additional information which may be significant, i.e. general condition of rock at site (weathered, fractured, extensive joint systems, joint filling, solutioning, water seepage, etc.).

Figure 22. Sample Identification Sheet and Instructions.

reference file. The code and identification of the site and(or) formation will be entered in the data bank (Category 3) 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 preliminary steps. First, the classification system proposed in Chapter III (Figure 15) must be adapted (ranges of properties for each parameter or the parameters themselves changed) depending on the intended use. Second, a use table encompassing uses relevant to the intended purpose must be developed and appropriate ranges of the index parameters determined for each (see Figure 23). 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. To use the program, one has simply to follow procedures outlined in Figure 24. 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.

There are, of course, other uses of the data, such as a basis for research. The important concept is that the program value depends upon the amount and quality of information which is fed into it. Information gained during and after (construction and monitoring) use should be fed back into the data bank for retention and future reference. In this way, the program becomes self perpetuating.

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

Figure 23. Typical Data Use Table.

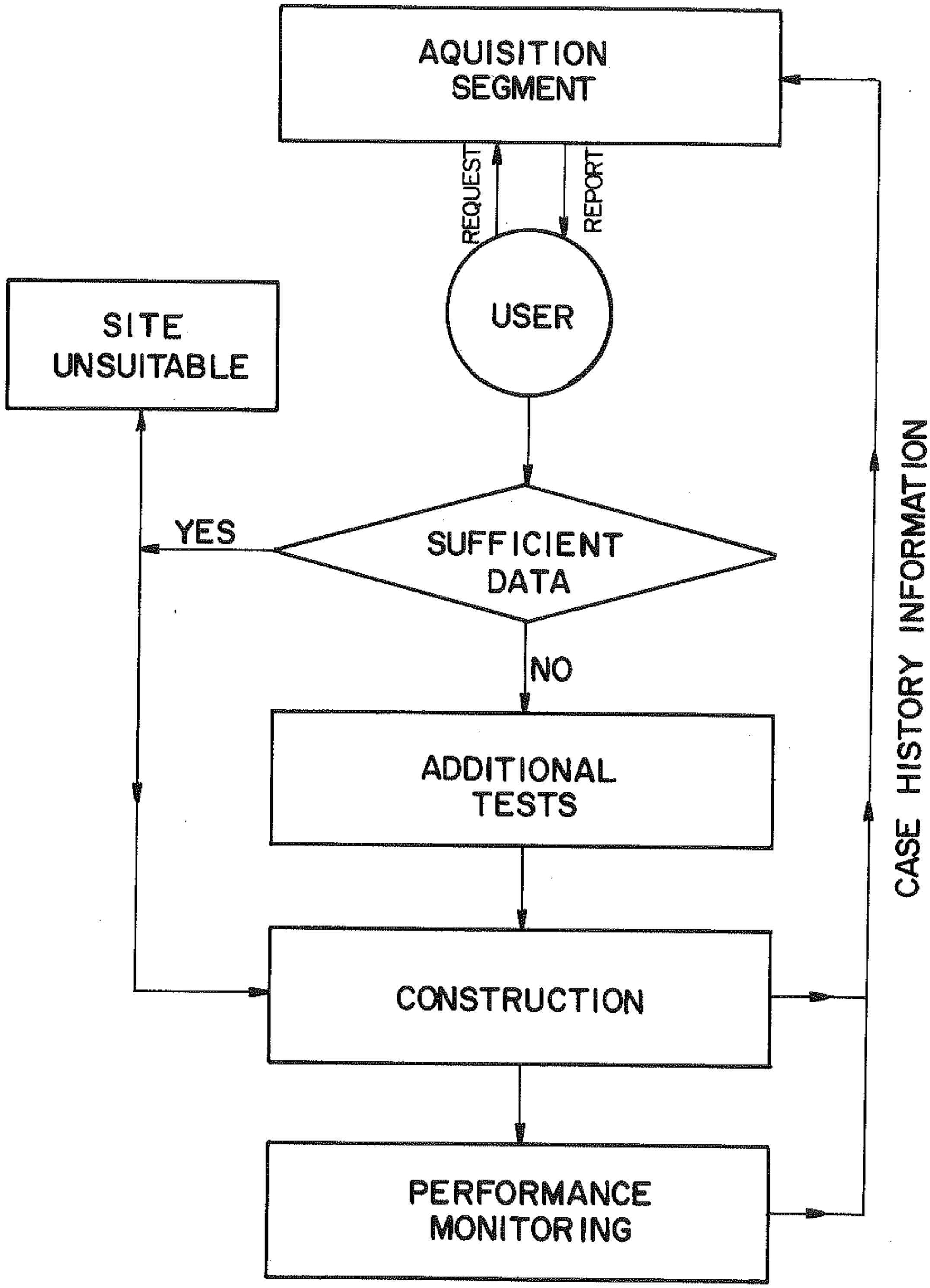


Figure 24. Schematic Diagram of the Use of the Applications Segment.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

The scope of rock engineering encompasses at least three major concepts: engineering 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 (John, 1962). 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 program 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 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 the rock as engineering construction material has been suggested.

A next stage in the implementation of the proposed rock evaluation program would be to computerize the systematic input, storage, and retrieval of the data. Similarily, development and utilization of statistical analysis must be accomplished for rock correlation and prediction of engineering behavior of rock, both intact and in situ. Computerization of the system would require definition of routine sample acquisition and laboratory testing procedures. Finally, a means of cataloging experience from past construction activities (successful and unsuccessful) must be established.

FURTHER STUDIES

For complete development of a functional rock evaluation program, a further series of studies must be undertaken. These would include:

a) since the quantitative ranges of the qualitative descriptive terms within the classification systems are tentative (strong intact rock having a range of 69 MPa to 170 MPa, etc.), it is necessary to verify or adjust these numerical ranges;

b) through further study, it may be determined that different parameters are needed within the suggested classification systems;

c) the most economical means possible should be sought to accomplish laboratory testing and acquisition of data for Kentucky rock types;

d) delineation of ranges of individual classification parameters for suggested use tables is necessary to successful utilization of rock as a construction material (aggregate, lower base course, etc.), to effect rock removal operations, and for reliable performance prediction;

e) further study of Kentucky's transitional materials is necessary to define the exact acquisition procedures and testing routines necessary to characterize material as "soil-like" or "rock-like" and to obtain design parameters and ranges to be included in use tables;

f) case histories from Kentucky should be studied and added, where appropriate, to the data bank; and

g) a format for a rock monitoring system (subprogram) should be designed so design analyses may be checked regarding environmental effects and probable failure conditions in situ.

RELATED STUDIES

There are numerous studies which may be incorporated within the framework of the proposed rock evaluation program for transportation facility planning. Some of these studies include:

a) development of a terrain classification system -- the system could be established for Kentucky to delineate construction problems of water table locations, rock forms present, etc., to aid in the initial construction bids for a particular proposal;

b) use of geophysical methods for subsurface exploration -- a routine for indirect subsurface exploration is needed to aid field investigations in Kentucky (Wahlstrom, 1973);

c) use of model studies in blasting -- the use of laboratory-scale blasting experiments (Johnson, 1962) to control variables and reduce costs of studying blast techniques relevant to Kentucky rock;

d) increased use of airphoto interpretation -- an indexing system is available (Holden, 1967) which classifies areas according to geology, altitude, climate, topography, drainage pattern, erosion cross section, and vegetation (this information can form the basis for interpretation of soil types, sources of construction materials, etc. (Caiger, 1967));

e) use of infared imagery -- preliminary studies have shown that it is possible to correlate soil temperature with soil moisture. (There are also indications that landslides are well defined on thermal infared imagery and specific zones of active water seepage can be identified (Greeley, Blanchard, and Gelnett, 1974). It may be possible to monitor by means of airborne infared imagery large areas for potential landslides.)

Implementation of these procedures would aid greatly in evaluation of the engineering capabilities of surficial earth materials.

KGS-USGS GEOLOGIC QUADRANGLE REFERENCES

The following is a listing of authors and geologic quadrangle map numbers used to compile Figure 11 of this paper:

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APPENDIX A

ADDITIONAL GEOLOGICAL INFORMATION

GEOLOGIC COLUMN AND TIMETABLE
(After Leet and Judson, 1971)

ERA (ERA)[a]	PERIOD (SYSTEM)	EPOCH (SERIES)	TIME DURATION (YEARS x 10^{-6})	TIME BEFORE PRESENT (YEARS x 10^{-6})
Cenozoic	Quaternary	Holocene	.01	
		Pleistocene	1.5 - 2	
	Tertiary	Pliocene	5 - 5.5	
		Miocene	19	
		Oligocene	11 - 12	
		Eocene	15 - 17	
		Paleocene	11 - 12	65
Mesozoic	Cretaceous		71	
	Jurassic		54 - 59	
	Triassic		30 - 35	225
Paleozoic	Permian		55	
	Pennsylvanian		45	
	Mississippian		20	
	Devonian		50	
	Siluvian		35 - 45	
	Ordovician		60 - 70	
	Cambrian		70	570
Precambrian	Precambrian			

[a]Geologic time units are referred to first, followed by the rock unit in parenthesis.

LITHOLOGY OF MAJOR GEOLOGIC FORMATIONS

The following abbreviated lithologic descriptions of the major Kentucky surface and near-surface geologic formations have been compiled from McFarlan (1961) and geological quadrangle maps published by the Kentucky Geological Survey. The descriptions are typical of average characteristics prevalent in the formations. However, in light of the variable nature of lithology, the geological quadrangles and all other available sources of information should be consulted when information on specific sites is required. The formations are listed by geologic age and are further subdivided by geographic location where necessary.

CENOZOIC ERA

QUARTERNARY SYSTEM

Holocene Series

Alluvium: sands, silts, clays, and gravels: in varying proportions occurring as unconsolidated deposits in stream beds.

Pleistocene Series

Loess: silts: occurs predominantly in the western portion of the state and along the bluffs east of and adjacent to the Ohio River.

Continental Deposits: unconsolidated gravels, sands, silts, and clays: occur predominantly in Western Kentucky.

TERTIARY SYSTEM

Eocene Series

Jackson Formation: sands and clays: predominantly sand, occurs in the Jackson Purchase Region.

Claiborne Formation: sands, clays, and silts: unconsolidated deposits occurring in the Jackson Purchase Region.

Wilcox Formation: sands, silts, and clays: unconsolidated. Two members are recognized, the Grenada (upper) and the Holly Springs (lower).

Paleocene Series

Porter's Creek Formation: clay, silt, and sand: occurs in the Jackson Purchase Region. The upper member of the Midway Group.

MESOZOIC ERA

CRETACEOUS SYSTEM

Upper Cretaceous Series

Eutaw Formation: sands: some layers slightly cemented by iron oxides.

Tuscaloosa Formation: gravel and clay: the gravel consists mainly of chert, some cobbles as large as 1 foot (0.3 m).

PALEOZOIC ERA

PENNSYLVANIAN SYSTEM -- Western Coal Field

Upper Pennsylvanian Series

Henshaw-Dixon (Monongahela) Formation: sandstone, shale, and thin seams of coal and limestone. Sandstone: medium- to fine-grained, massive. Shale: dark gray to green in color, with thin seams of coal and limestone, thinly bedded. Outcrops only in Webster and Hopkins Counties.

Lisman (Conemaugh) Formation: sandstone, shale, siltstone, coal, underclay, and limestone. Sandstone: massive to thinly layered, medium gray to yellow-brown. Shale and siltstone: very thinly bedded, light gray to black. Limestone: dense, massive (occurs as one bed), light to dark gray.

Middle Pennsylvanian Series

Carbondale (Allegheny) Formation: shale, coal, sandstone, underclay, and limestone: predominant coal-bearing formation of the Western Coal Field. Shale: dark gray to black, thinly bedded. Sandstone: massive to thinly bedded, light gray to yellow-brown. Limestone: massive, medium gray.

Tradewater (Pottsville) Formation: sandstone, shale, siltstone, coal, limestone, and underclay: predominantly a shale formation. Sandstone: white to light gray where fresh, stained brown to red, fine-grained, massive to thinly bedded. Shale: predominantly gray, some white near limestone, some black above coal layers, thinly bedded as is the siltstone. Underclay: associated with coal layers.

Lower Pennsylvanian Series

Caseyville (Pottsville) Formation: sandstone: massive cliff-forming beds, some 100 feet (30 m) or more thick; conglomeritic, similar to the Lee Formation of the Eastern Coal Field. Outcrops mainly on the border of the Western Coal Field.

PENNSYLVANIAN SYSTEM - Eastern Coal Field

Upper Pennsylvanian Series

Conemaugh Formation: sandstone, shale, and limestone. Sandstone: massive, cliff-forming, forms most of the lower part of the formation. The upper part is mostly shale; purple, red, and green. Limestone: occurs in thin beds. Outcrops only in Boyd County and northern Lawrence County.

Middle Pennsylvanian Series

Allegheny Formation: shale, sandstone, coal, and limestone. Shale: light gray to black, thinly bedded. Sandstone: massive, cliff-forming. Limestone: occurs as thin beds. Outcrops mainly in Boyd County and vicinity.

Breathitt (upper portion) Formation: shale, sandstone, coal, and underclay. Shale: medium to dark gray, silty, and interbedded with siltstone or fine-grained sandstone. Sandstone: light gray, micaceous, fine-grained, massive to thinly bedded. Underclay: occurs in thin beds beneath coal layers.

Lower Pennsylvanian Series

Breathitt (lower portion) Formation: essentially identicial to upper portion, described above.

Lee Formation: sandstone and shale. Sandstone: white to medium gray, locally conglomeritic. Rockcastle member provides the massive, cliff-forming cap rock of most of the Pottsville Escarpment. Shale members: medium to dark gray, thinly bedded.

MISSISSIPPIAN SYSTEM

Upper Chesterian Series -- Fluorspar Region

Kinkaid Formation: limestone: light to dark gray, hard, dense, beds 1 to 2 feet (0.3 to 0.6 m) thick.

Degonia Formation: shale and siltstone. Shale: gray, silty. Siltstone: white to gray, thinly bedded, locally massive.

Clore Formation: shale and limestone. Shale: dark gray, thinly bedded. Limestone: medium to dark gray, hard and compact.

Palestine Formation: sandstone and shale. Sandstone: fine-grained, massive to thinly bedded. Shale: dark gray, silty, thinly bedded.

Menard Formation: limestone and shale. Limestone: medium to dark gray, massive to thinly bedded. Shale: greenish-gray to dark gray, calcareous; contains dolomite seams. Present over most of Western Kentucky.

Waltersburg Formation: shale and sandstone. Sandstone: soft, broken. Shale: dark brown to yellow, fissile.

Vienna Formation: limestone: medium to dark gray, thin but uniform.

Tar Springs Formation: shale and sandstone: dark gray shale with sandstone lenses.

Middle Chesterian Series -- Fluorspar Region

Glen Dean Formation: limestone and shale. Limestone: blue to gray, moderately to coarsely crystalline, argillaceous; contains shale beds 1 to 2 feet (0.3 to 0.6 m) thick. Shale: varies from black to green.

Hardinsburg Formation: sandstone and shale. Sandstone: white to light gray, massive to thinly bedded, dolomitic. Shale: dark gray, contains sandstone and siltstone lenses.

Golconda Formation: shale and limestone. Shale: dark gray, calcareous. Limestone: medium to dark gray, argillaceous, massive, highly soluble.

Cypress Formation: sandstone, shale, and coal. Sandstone: light gray, fine-grained, massive. Shale: sandy. Formation forms the cap rock for the Dripping Springs Escarpment.

Lower Chesterian Series -- Fluorspar Region

Paint Creek Formation: sandstone and shale. Sandstone: light gray, fine-grained, thinly bedded. Shale: black to gray, fissile; some limestone lenses interbedded.

Bethel Formation: sandstone: light gray, medium- to fine-grained, massive, cliff-forming.

Renault Formation: limestone and shale. Limestone: light to medium gray, thickly bedded, moderately crystalline. Shale: greenish-gray to gray, calcareous.

Aux Vases Formation: sandstone: very similar to the Cypress Sandstone. Does not appear in surface outcrop.

Upper Chesterian Series -- West of Cincinnati Arch

Leitchfield (Buffalo Wallow) Formation: shale, limestone, and sandstone. Shale: greenish-gray to maroon; comprises the bulk of the formation. Limestone: equivalent of Vienna Formation, argillaceous. Sandstone: yellow-brown to gray, thinly bedded.

Middle Chesterian Series -- West of Cincinnati Arch

Glen Dean Formation: same as in Fluorspar Region.

Hardinsburg Formation: same as in Fluorspar Region.

Golconda Formation: sandstone, limestone, and shale. Sandstone (Big Clifty Member): pale orange to brown, massive to thinly bedded. Limestone: light gray to olive, cherty, highly soluble. Shale: gray, thinly bedded.

Lower Chesterian Series -- West of Cincinnati Arch

Elwren Formation: shale and sandstone. Shale: olive gray to brown, thinly bedded. Sandstone: yellowish-gray, fine-grained, argillaceous.

Reelsville Formation: limestone: olive-gray to light gray, fine-grained, massive.

Sample Formation: sandstone, siltstone, and shale. Sandstone: yellow-brown, thinly to thickly bedded. Siltstone: light brown, argillaceous, thinly bedded. Shale: gray to green, thinly bedded.

Beaver Bend Formation: limestone: light gray to olive, fine- to medium-grained, medium bedded; weathers to platy rubble.

Paoli Formation: limestone: yellow-gray to light gray, fine-grained, beds 1 to 2 feet (0.3 to 0.6 m) thick.

Upper Chesterian Series -- East of Cincinnati Arch

Pennington Formation: shale, sandstone, and limestone: equivalent of the Leitchfield Formation west of the Cincinnati Arch. Shale: light to dark gray, forms the main part of the formation.

Middle Chesterian Series -- East of Cincinnati Arch

Bangor Formation: limestone: dark to medium gray, argillaceous.

Hartselle Formation: sandstone: light yellow to brown, massive to thinly bedded, quartzose.

Lower Chesterian Series - East of Cincinnati Arch

Monteagle (Newman) Formation: limestone: medium to dark gray, thickly to thinly bedded, lenses of calcareous siltstone.

Meramecian Series

Ste. Genevieve Formation: limestone: light gray to olive-gray, clastic, medium-grained, zones of chert and silicified limestone. Three members commonly recognized: Levias, Rosiclare, and Fredonia.

St. Louis Formation: limestone: gray to dark gray, thickly bedded, cherty, some thin shales present.

Salem Formation: limestone and shale. Limestone: gray to bluish-gray, massive to thinly bedded; argillaceous, fine- to coarse-grained. Shale: calcareous, brownish-gray, thinly bedded.

Warsaw (Harrodsburg) Formation: limestone: similar to the Salem Formation; chert locally present.

Osagean Series

Borden Formation: interbedded shale, siltstone, and cherty limestone; formation occurs only in the northern portions of Kentucky; several members are recognized:

Cowbell: siltstone and shale.

Muldraugh: siltstone and chert: light gray, medium to thickly bedded, contains shale partings.

Halls Gap: siltstone, shale, and chert.

Nancy: shale: silty, light to dark gray.

New Providence: clay shale and claystone: pale green to grayish, fissile.

Ft. Payne Formation: shale and limestone. Shale: greenish-gray in lower one-third, medium to dark gray in remainder, numerous siliceous geodes. Limestone: greenish-gray to light gray, silicified. Occurs only in the southern portion of Kentucky.

Kinderhookian Series

Sunbury Formation: shale: black, fissile; occurs in Northeastern Kentucky.

Berea Formation: sandstone: gray, thickly bedded, fine-grained, loosely cemented; noted for ripple-marked bed surfaces; occurs along the Ohio River in Eastern Kentucky.

DEVONIAN SYSTEM

Upper Devonian Series

New Albany (Ohio, Chattanooga) Formation: shale: dark gray to black, carbonaceous, dense when fresh, fissile after weathering, phosphate nodules present.

Middle Devonian Series -- West of Cincinnati Arch

Sellersburg Formation: limestone: dolomitic, argillaceous, locally cherty; divided into three members:

Casey: thickly bedded, gray, fine-grained, cherty.

Beechwood: thickly bedded, gray, coarse-grained.

Silver Creek: thickly bedded, dark gray, fine-grained, cherty.

Jeffersonville Formation: limestone: olive-gray, brownish-gray, or light gray, fine- to very coarse-grained, locally dolomitic, some chert in unconformity at base.

Middle Devonian Series - East of Cincinnati Arch

Boyle Formation: dolomite: olive-gray to brownish-gray, fine- to medium-grained, massive; upper portions contain chert.

SILURIAN SYSTEM -- West of Cincinnati Arch

Middle Silurian Series

Louisville Formation: limestone: light gray, fine-grained, massive, locally dolomitic.

Waldron Formation: shale: dark greenish-gray, calcareous and magnesian, non-fissile, breaks into irregular pieces.

Laurel Formation: dolomite: bluish- to light gray, medium- to fine-grained, massive to regularly bedded.

Osgood Formation: dolomite and dolomitic shale. Shale: greenish-gray, soft, fissile. Dolomite: fine-grained, gray, shaly.

Lower Silurian Series

Brassfield Formation: limestone: grayish-orange to yellowish-brown, micrograined, dolomitic, massive below, thinly bedded above.

SILURIAN SYSTEM -- East of Cincinnati Arch

Middle Silurian Series

Bisher (Peebles, Lilley) Formation: dolomite: gray, fine- to coarse-grained, massive to medium-bedded.

Crab Orchard Formation: shale with interbedded limestone. Formation consists of several recognized members:

Ribolt: shale: very thinly bedded.

Estill Clay: shale: blue, soft.

Waco: limestone; dolomitic, thinly bedded.

Lulbegrud Clay: shale: blue, soft.

Oldham: limestone: dolomitic, interbedded with blue shale.

Plum Creek: shale: interbedded with limestone.

Lower Silurian Series

Brassfield Formation: limestone: identical with formation west of the Cincinnati Arch.

ORDOVICIAN SYSTEM

Upper Ordovician Series -- Southwest Blue Grass

Drakes Formation: interbedded dolomite and shale. Dolomite: gray to yellowish-gray, microcrystalline to moderately crystalline. Shale: dolomitic, greenish-gray.

Ashlock Formation: interbedded dolomite, limestone, and shale. Dolomite: silty, gray to greenish-gray, micrograined, thinly bedded. Limestone: light to brownish-gray, thinly bedded.

Grant Lake Formation: limestone and shale. Limestone: medium gray, thinly bedded, rubbly appearance. Shale: occurs as partings between limestone blocks.

Calloway Creek Formation: limestone and shale. Limestone: medium gray, medium- to coarse-grained, thinly bedded. Shale: light gray to olive-gray, very thinly bedded.

Garrard Formation: siltstone, shale, and limestone. Siltstone: calcareous, olive-gray to greenish-gray, thinly bedded. Shale: light gray to olive, contains siltstone lenses, blocky. Limestone: thinly bedded.

Clays Ferry Formation: limestone and shale. Limestone: medium gray to olive-gray, uniform texture, interbedded with shale. Shale: calcareous, gray, weathers to plates and blocks.

Upper Ordovician Series - Northeast Blue Grass

Drakes Formation: same as in southwest Blue Grass

Bull Fork Formation: shale and limestone. Shale: medium gray to grayish-green, thinly bedded, fissile, calcareous, plastic when wet. Limestone: light gray to bluish-gray, thin to medium thick beds, fossiliferous.

Grant Lake Formation: same as in southwest Blue Grass.

Fairview Formation: limestone, siltstone, and shale. Limestone: predominant member, light bluish-gray to olive-gray, thin beds of equal thickness. Siltstone: light olive-gray, thinly to medium bedded, calcareous. Shale: olive-gray, fissile, calcareous.

Kope Formation: shale, siltstone, and limestone. Shale: medium gray, fissile, calcareous, fossiliferous. Siltstone: medium gray, calcareous, thinly to thickly bedded. Limestone: medium gray, coarse- to fine-grained, silty, very thinly bedded.

Clays Ferry Formation: same as in southwest Blue Grass.

Middle Ordovician Series

Lexington Formation: includes several members, predominantly limestone with some interbedded shales. Limestone: light to dark gray, medium to thinly bedded. Shale: medium gray, fissile, calcareous.

High Bridge Formation: limestone: gray to cream-colored, massive, cliff-forming, dolomitic. Formation consists of three members:

Tyrone: light gray, calcite and chert present.

Oregon (Kentucky River Marble).

Camp Nelson: surface weathers to honeycombed structure.

GLOSSARY OF GEOLOGIC TERMS

Definitions in this glossary have been derived from Leet and Judson (1971) and Webster's Third International Dictionary.

ALLUVIUM -- Deposits resulting from deposition of sediments by streams and rivers.

ANTICLINE -- A configuration of folded rock strata in which the rocks dip in two directions away from a crest known as the axis.

ARCH -- Anticline; rock folded into a configuration that resembles an arch.

BEDDING -- (1) A term used to signify the existence of layers (beds) in sedimentary rocks. (2) Sometimes synonymous with bedding plane.

BEDDING PLANE -- Surface separating the layers of sedimentary rock. Each plane marks the termination of one deposit and the beginning of another.

COMPETENCY -- A measure of the strength and soundness of a rock.

CONGLOMERATE -- A rock formed from rounded, water-worn pebbles of various sizes held together in a matrix of finer materials.

CONTACT -- A plane separating two rock units.

DETRITUS -- Loose material formed from the erosional or weathering products of other rock types.

DIKE -- A tabular-shaped igneous intrusion formed when molten magma is forced upward along joints in the overlying rock mass.

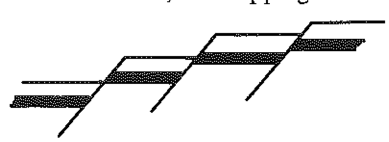
DIP -- The acute angle that a rock surface makes with a horizontal plane.

DOLOMITE -- A rock composed mainly of calcium magnesium carbonate. A magnesian limestone.

DOME -- An anticlinal fold, without a clearly developed linearity of crest, so that the beds involved dip in all directions from a central area.

DRIFT (glacial) -- Any material laid down directly by ice or deposited in lakes, oceans, or streams as a result of glacial activity.

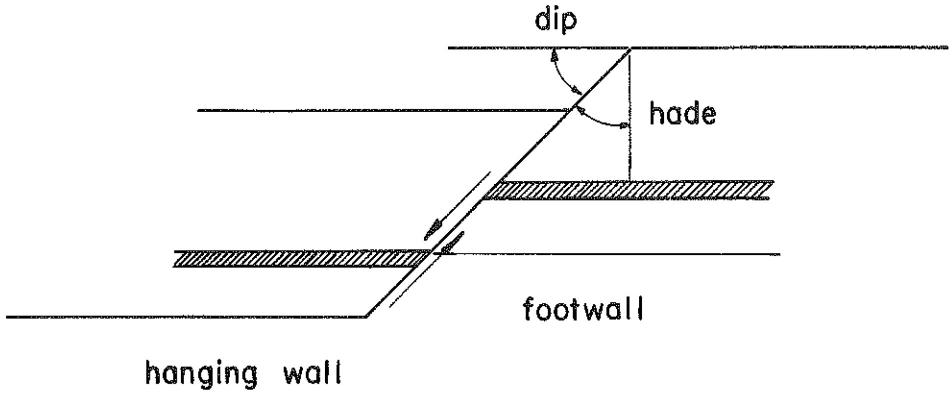
EN ECHELON FAULTING -- Series of short, overlapping faults.



ESCARPMENT -- A relatively steep slope developed by erosion or faulting, may range in height from a few feet (meters) to thousands of feet (meters).

FACIES -- An assemblage of mineral, rock, or fossil features reflecting the environment in which a rock was formed.

FAULT -- A surface of rock rupture along which there has been differential movement. Terminology associated with faults is illustrated in the accompanying diagram.



FISSILITY -- A property of splitting along closely spaced planes more or less parallel to the bedding.

FLUORSPAR (FLUORITE) -- A mineral composed of calcium fluoride. It is used as a flux in making steel.

GEOLOGIC COLUMN -- A chronologic arrangement of rock units in columnar form with the oldest units at the bottom and the youngest at the top.

GEOSYNCLINE -- A basin in which thousands of feet (meters) of sediments have accumulated with accompanying progressive sinking of the basin floor.

GLACIER -- A mass of ice, formed by the recrystallization of snow, that flows forward, or has flowed at some time in the past, under the influence of gravity.

HIATUS -- An interruption or lapse in deposition.

IGNEOUS INTRUSIONS -- Rock in various forms which has resulted from the invasion of the earth's crust by molten magma which never reached the surface.

INTERBEDDED -- Alternating beds of two or more types of rock.

KARST TOPOGRAPHY -- Irregular topography characterized by sinkholes, streamless valleys, and streams that disappear underground, all developed by the action of surface and underground water in soluble rock such as limestone.

LIMESTONE -- A sedimentary rock composed largely of the mineral calcite, $CaCO_3$.

LITHOLOGY -- The character of a rock in terms of its structure, mineral composition, color, and texture.

NORMAL FAULT -- Gravity Fault; fault in which the hanging wall (block above the plane of rupture) has moved downward relative to the foootwall (block below the plane of rupture) (see Fault).

OUTCROP -- The exposure of a formation at the surface of the earth.

OVERTHRUST FAULT -- Very low angle reverse fault. The angle of dip is less than 10 degrees (0.175 rad).

PENEPLANATION -- The reduction of the topography of an area to a flat, low elevation plain by erosion.

PERIODOTITE -- A coarse-grained igneous rock dominated by dark colored minerals, consisting of about 75 percent ferromagnesian silicates and the balance plagioclase feldspars.

PHYSIOGRAPHIC PROVINCE -- A region having a particular pattern of landforms that differs significantly from adjacent regions.

RELIEF -- The difference in elevation between hilltops and valley bottoms in an area.

REVERSE FAULT -- Fault in which the hanging wall (block above the plane of rupture) has moved upward with respect to the footwall (block below the plane of rupture). Sometimes referred to as a thrust fault if the angle of dip is less than 45 degrees (0.785 rad).

SANDSTONE -- A detrital, sedimentary rock formed by the cementation of grains of sand-sized particles, usually quartz.

SHALE -- A fine-grained detrital, sedimentary rock composed of silt- and clay-sized particles, predominantly clay minerals but others present also. Bonding ranges from compacted shales to indurated hard shale. Fissility is always exhibited.

SINKHOLE -- Surface depression resulting from the collapse of a subsurface solution cavity.

SOLUTIONING -- Phenomenon associated with rocks formed of the mineral calcite, $CaCO_3$, primarily limestones. Solutioning occurs as a result of the dissolving of the calcite by an acid formed from water and carbon dioxide (carbonic acid).

STRATIGRAPHIC UNIT -- A particular bed of sediments that exhibits the same lithology and can therefore be considered as a unit.

STRATIGRAPHY -- That phase of geology treating the sequence in which formations have been deposited.

SYNCLINE -- A configuration of folded stratified rocks in which the rocks dip downward from opposite directions to form a trough. The reverse of an anticline.

TECTONIC -- A term which refers to forces and movements inherent in the earth's crust.

THRUST FAULT -- A reverse fault with an angle of dip less than 45 degrees (0.785 rad).

UNCONFORMITY -- A buried erosion surface separating two rock units, the older of which was exposed to erosion before deposition of the younger. If the older rocks were deformed before erosion and were not horizontal at the time of subsequent deposition, the contact is known as an *angular unconformity*. If the older rocks remain horizontal, the contact is called a *disconformity*.

UNDERCLAY -- A thin layer of clay found beneath a coal layer.

WEATHERING -- The changing of minerals or rocks in response to mechanical or chemical stimuli.

APPENDIX B

TRANSITIONAL MATERIALS
Clay Shale, Shale, Mudstone, Claystone,
and Other Argillaceous Sediments

TRANSITIONAL MATERIALS
Clay Shale, Shale, Mudstone, Claystone, and other Argillaceous Sediments

INTRODUCTION

There are many earth materials which are not readily classified as either soil or rock. These materials, herein designated transitional materials, are comprised primarily of clay- and silt-sized particles. On the basis of observer bias, particle-size distribution, mineralogy, and type and degree of bonding between grains, these materials have been assigned several names -- clay shale, shale, siltstone, mudstone, claystone, and marl are but a few of these names. The study of these transitional materials is of two-fold importance. First, argillacious (clayey) materials comprise 50 to 75 percent of the sedimentary rock in the earth's crust (Leet and Judson, 1971). Second, a high percentage of rock engineering problems (slope stability, settlement, bearing capacity failure, etc.) occur in transitional argillaceous materials (Gamble, 1971).

Several individuals and organizations have expended considerable effort to organize existing data and to test and classify materials which fall in the transitional category. Underwood (1967, 1969); Fleming, Spencer, and Banks (1970a); and Gamble (1971) present excellent comprehensive reviews of previous work.

BACKGROUND INFORMATION

Early attempts to classify transitional materials were based on geologic considerations. Parameters such as particle size, mineralogy, type and degree of bonding, and breaking characteristics were used in various combinations to categorize the materials.

One such system, based on particle size, was proposed by Wentworth in 1922 (Putnam, 1964):

Classification of Clastic Sedimentary Rocks
(After Wentworth, 1922)

	Sediment	Particle Size (mm)	Rock
	Boulder		
		256	
	Cobble		
Gravel		64	Conglomerate
	Pebble		
		4	
	Granule		
		2	
	Very Coarse Sand		
		1	
	Coarse Sand		
		1/2	
Sand	Medium Sand		Sandstone
		1/4	
	Fine Sand		
		1/8	
	Very Fine Sand		
		1/16	
	Silt		
Mud		1/256	Shale, Mudstone
	Clay		

This system provided an arbitrary division between the argillaceous materials (shale or mudstone) and the remaining clastic (fragmental) sedimentary rocks.

Transitional materials were further subdivided by Twenhofel in 1937 (Underwood, 1967):

Classification of Shales and Related Rocks
(After Twenhofel, 1937)

Unindurated	Indurated	After Incipient Metamorphism	Metamorphic Equivalent
	Mudstone		
Silt + Water = Mud Clay	Siltstone + Fissility = Shale Claystone	Argillite	Slate Phyllite Schist

Twenhofel's classification left unresolved the distinction between those transitional materials which behave primarily as soils (e.g., the "stiff-fissured clays" of Terzaghi (1936)) and those which exhibit rock-like characteristics.

Mead (cf. 1938) addressed this problem by proposing a classification which differentiated compacted ("soil-like") materials which have been consolidated by the weight of overlying sediments from cemented ("rock-like") materials on the basis of slake resistance (deterioration during wet-dry cycles) (Underwood, 1967):

Classification of Shale
(After Mead, 1938)

Shale

Soil-Like (Slake with Weathering Cycle)	Rock-Like (Not Slake with Weathering Cycle)
Clayey Shale	Calcareous Shale
Silty Shale	Siliceous Shale
Sandy Shale	Ferruginous Shale
Black Shale	Carbonaceous Shale
	Clay-Bonded Shale

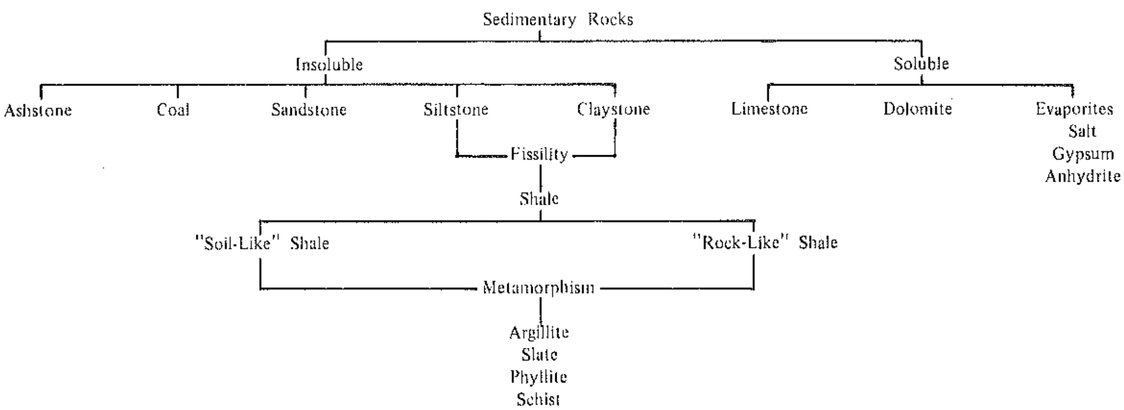
Clayey -- 50 percent or more clay-sized particles which may or may not be true clay minerals.
Silty -- 25 to 45 percent silt-sized particles (may be layered).
Sandy -- 25 to 45 percent sand-sized particles (may be layered).
Black -- organic-rich, splits into thin semi-flexible sheets.

Calcareous -- 20 to 35 percent $CaCO_3$.
Siliceous -- 70 to 85 percent amorphous silica, SiO_2.
Ferruginous -- 25 to 35 percent Fe_2O_3.
Carbonaceous -- 3 to 15 percent carbonaceous matter.
Clay-Bonded -- welded by recrystallization of clay minerals or other diagenetic bonds.

This system takes into account bonding in addition to particle size.

Recognizing the importance of rock solubility in engineering works, Philbrick (cf. 1950) divided sedimentary rocks into soluble and insoluble categories and combined these with a classification of argillaceous members similar to those of Twenhofel and Meade to obtain the following classification (Underwood, 1967):

Classification of Sedimentary Rocks
(After Philbrick, 1950)



Philbrick was somewhat more positive in his approach to the separation of compacted and cemented shales. He proposed a simple test in which the sample was subjected to five cycles of wetting and drying with a 100 N solution of ammonium oxalate or water. Those samples which reduce to individual grains are considered compacted; those unaffected or reduced only to flakes are considered cemented.

Attempts have been made to classify materials solely on chemical or mineralogical composition. Chemical composition alone is insufficient because the transitional materials are very similar in chemical content. Useful information can be obtained, however, from a knowledge of the clay minerals present in the material. In general, a high percentage of illite and smectite correspond to materials of high swell potential and low shear strength. Conversely, low percentages of the above-mentioned clay minerals or high percentages of kaolinite or chlorite indicate material of greater reliability (Underwood, 1967). Factors such as high test cost, time expenditure, and lack of standardized procedures have militated against use of mineralogical studies for classification.

Ingram (cf. 1953) published several conclusions with regard to breaking characteristics (fissility) which have been used for classification and identification purposes (Underwood, 1967):

a) three dominant types of breaking characteristics in shales:
   1) massive -- no preferred cleaving direction,
   2) flaggy -- breaks into fragments of varying thicknesses but with the width and length many times greater than the thickness, and
   3) flaky -- splits along irregular surfaces parallel to the bedding into uneven flakes;
b) fissility is associated with a parallel orientation of clay particles;
c) existence of organic matter in the rock tends to increase the tendency toward parallel orientation of clay particles;
d) most cementing agents cause a decrease in fissility; and
e) moderate weathering increases the fissility of a shale while intense weathering produces a soft massive clay.

Fissility alone is not of much value in classification since materials in the same stratum exhibit this phenomena to differing degrees.

ENGINEERING CLASSIFICATIONS

It is apparent the foregoing geologically-oriented classifications, while offering qualitative information, are somewhat ambiguous and do not provide quantitative information required for engineering purposes. Several recent works have been written in which the authors attempted to establish standardized terminology and proposed classifications based on engineering or mechanical properties of transitional materials. Underwood (1967) delineated several significant engineering properties and related probable ranges of values for in-situ behavior as indicated in the chart for engineering evaluation of shales.

AN ENGINEERING EVALUATION OF SHALES

Physical Properties			Probable In-situ Behavior						
Laboratory tests and in-situ observations	Average range of values: Unfavorable	Favorable	High pore Pressure	Low bearing capacity	Tendency to rebound	Slope stability problems	Rapid slaking	Rapid erosion	Tunnel support problems
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Compressive strength, in pounds per square inch	50 to 300	300-5000	√	√					
Modulus of elasticity, in pounds per square inch	20,000 to 200,000	200,000 to 2×10^6		√					√
Cohesive strength, in pounds per square inch	5 to 100	100 to >1500			√	√			√
Angle of internal friction, in degrees	10 to 20	20 to 65			√	√			√
Dry density, in pounds per cubic foot	70 to 110	110 to 160	√					√(?)	
Potential swell, in percentage	3 to 15	1 to 3			√	√		√	√
Natural Moisture content, in percentage	20 to 35	5-15	√			√			
Coefficient of permeability, in centimeters per second	10^{-5} to 10^{-10}	$>10^{-5}$	√			√	√		
Predominant clay minerals	Montmorillonite or Illite	Kaolinite & Chlorite	√			√			
Activity ratio = Plasticity index / Clay content	0.75 to >2.0	0.35 to 0.75				√			
Wetting and drying cycles	Reduces to grain sizes	Reduces to Flakes					√	√	
Spacing of rock defects	Closely Spaced	Widely Spaced		√		√		√(?)	√
Orientation of rock defects	Adversely Oriented	Favorably Oriented		√		√			√
State of stress	>Existing Over-burden Load	≅ Over-burden Load			√	√			√

Underwood concluded that test results available at the time of his writing (1967) were not sufficiently replicable for detailed evaluation. He therefore did not describe test procedures to be used to obtain property values but presented values in broad ranges which could be narrowed as more consistent test results became available.

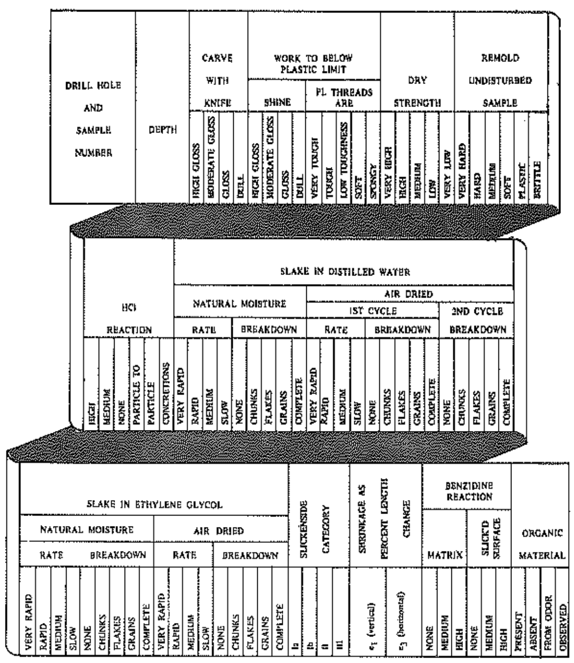
In a discussion of Underwood's paper, Philbrick (1969) pointed out that the system appeared to be aimed at compacted ("soil-like") materials while neglecting cemented ("rock-like") groups. Philbrick suggested the need for a distinction between the two kinds of material, perhaps based on particle size. Underwood (1969) concurred and suggested separate tables for the compacted and cemented types might be used to advantage. He suggested the addition of Atterberg limits to the compacted material table and a measure of observed slope angles with relation to slope height for the cemented materials. In this latter discussion, Underwood also pointed out the importance of moisture content: "One of the most significant indicators of the probable engineering behavior of shale is its in situ moisture content." Plots of moisture content versus depth can be used to indicate potential trouble zones.

The term "clay shale" has been used by several authors to describe the compacted transitional materials. Bjerrum (1967) referred to "overconsolidated clays and clay shales" in his paper on progressive slope failure. Fleming, Spencer, and Banks (1970a) also used "clay shale" in reference to compacted materials. They applied the terms "claystone" and "siltstone" to cemented materials composed primarily of clay-sized and silt-sized particles.

While not proposing a formal classification system, personnel of the Waterways Experiment Station (WES) have presented empirical evidence pertaining to the behavior of transitional materials. Conclusions drawn and methods of approach used to collect and retain data are useful and informative. Results of one such study on the behavior of transitional material at five locations in the upper Missouri Basin (Fleming, Spencer, and Bank's, 1970a; 1970b) lead to the following conclusions:

a) the principal features determining the engineering behavior of clay shales are the degree of overconsolidation and the lithology, both reflections of geologic history;
b) overconsolidation is related to undesirable engineering behavior such as swelling, high lateral residual stresses, and fissure development;
c) important features of lithology are mineral composition (especially clay minerals), mechanical composition (particularly the clay-size fraction), presence or absence of any cementing agent, and degree of homogeneity; and
d) other important factors including local geologic structure (the presence of relatively stronger or weaker strata may favorable or unfavorably affect the mass), water conditions (materials stable at low moisture contents may be unstable when saturated), and time (progressive failure may occur as a result of bond deterioration).

Laboratory testing associated with this project was extensive and well documented. A typical sheet presenting some of the results is included here. The authors concluded from the large range of measured values within the same material that only gross differences in behavior can be obtained through a testing program. They recommended design based on empirical evidence (local site geologic and hydrologic conditions and examination of nearby natural slopes in identical materials).



Sample Identification and Examination Summary.

A similar study of slopes in transitional material along the Panama Canal (Lutton and Banks, 1970) was initiated in 1968. Initially, the study was conducted in approximately the manner previously described and the testing program was identical. The conclusions reached were basically the same; however, a program of instrumentation and monitoring was also incorporated, the results of which were to be reported at a later date (approximately 1975).

Methods used by WES personnel are applicable primarily to the compacted category of transitional materials. No distinction is (or must be) made between "soil-like" or "rock-like" materials. It would be convenient, however, when working with index tests and classification systems for rock to establish a limit of sorts, admittedly arbitrary, below which transitional materials would be subjected to index tests applicable to soils and above which such testing would not be required. The limit should be somewhat more definitive than those of Mead or Philbrick. In this regard, the work of Gamble (1971) and Franklin, Deere, and others associated with the Commission on Standardization of Laboratory and Field Tests of the International Society for Rock Mechanics (Franklin, et al., 1972) is very helpful.

After a thorough review of past experience dealing with transitional materials, Gambel contributed the following:

a) To standardize the prevailing geologic terminology, he proposed the following geologic classification for argillaceous materials:

Unindurated Group	Indurated Group		After Incipient Metamorphism	Metamorphic Equivalents
	Mudrocks (Shales or Mudstones)			
	Breaking Characteristics			
	Massive	Fissile or Shaly		
Silt →	Siltstone	Silty Shale		
Mud[a] →	Mudstone	Shale	→ Argillite →	Slate, Phyllite, or Schist
Clay →	Claystone	Clayey Shale		

[a]Mixture of undetermined amounts of silt and clay with minor amount of sand

Definition of terms:

Indurated -- Rock hardened by pressure, cementation, or heat; includes both compacted and cemented hardened materials.

Massive -- Non-fissile or non-shaly material, breaks in apparently random directions in blocky or irregular shapes.

Fissile -- Splits along approximately parallel surfaces, parallel to bedding.

Shaly -- Splits or breaks into flakes, chips, or thin flat pieces approximately parallel to bedding.

Siltstone -- Massive, indurated rock composed predominantly of silt. Often contains small amounts of fine sand, is grittier and usually harder than adjacent claystones or mudstones.

Claystone -- Massive, indurated rock composed predominantly of clay. Smooth to touch.

Mudstone -- Massive, indurated mixture of undetermined amounts of silt and clay, with possible minor amounts of sand.

Silty Shale -- Fissile, shaly, or laminated, indurated rock composed predominantly of silt.

Clayey Shale -- Fissile, shaly, or laminated, indurated rock composed predominantly of clay.

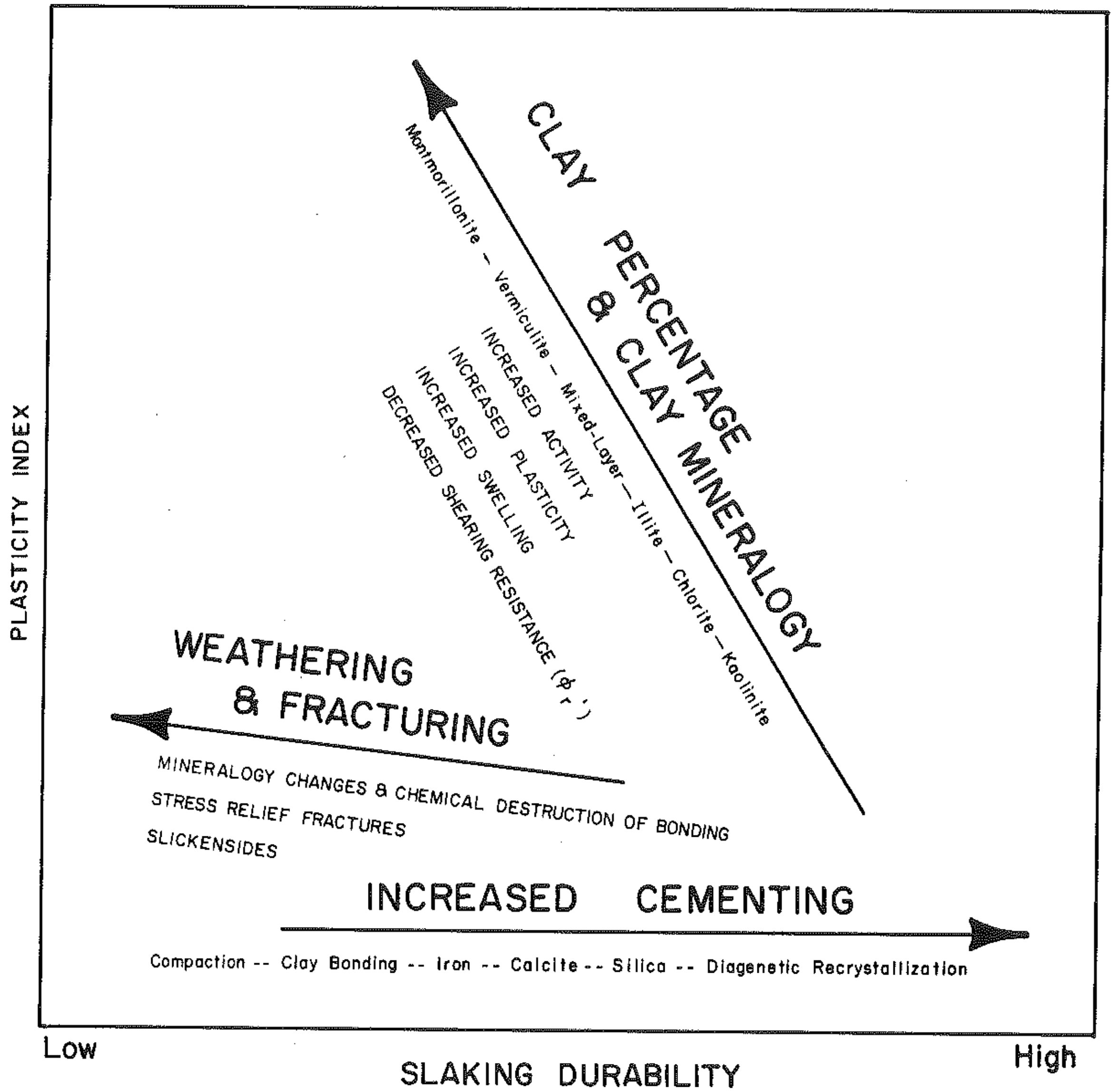
Shale -- Fissile, shaly, or laminated, indurated mixture of undetermined amounts of silt and clay with possible minor amounts of sand.

b) The major engineering problems associated with transitional materials are:

1) low durability -- rapid weathering or slaking in open excavation, differential weathering of slopes and cuts, and slaking or slabbing in tunnels and other underground excavations;

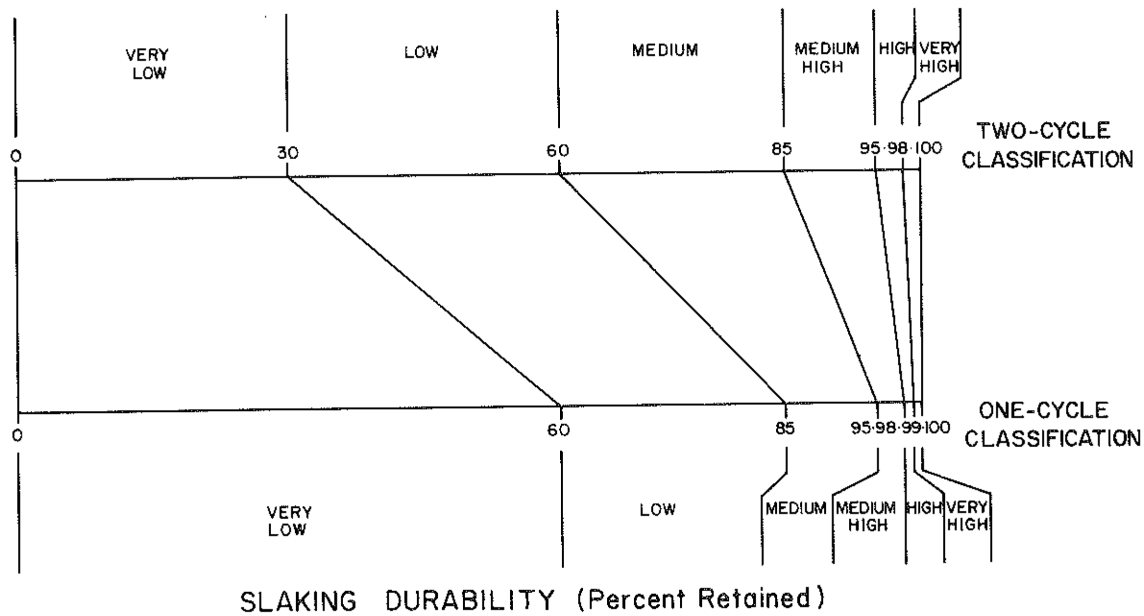
2) swelling, rebound, or stress relief -- common in smectitic clayey shales; caused by relief of overburden pressure, clay mineral hydration, or oxidation reactions or iron sulfides with accompanying volume increase.
3) low shear strength -- problem in slope stability and foundations, discontinuities are often responsible for low strength zones.

An informative chart relating variables that affect behavior was also presented by Gamble (1971) and is reproduced herein.

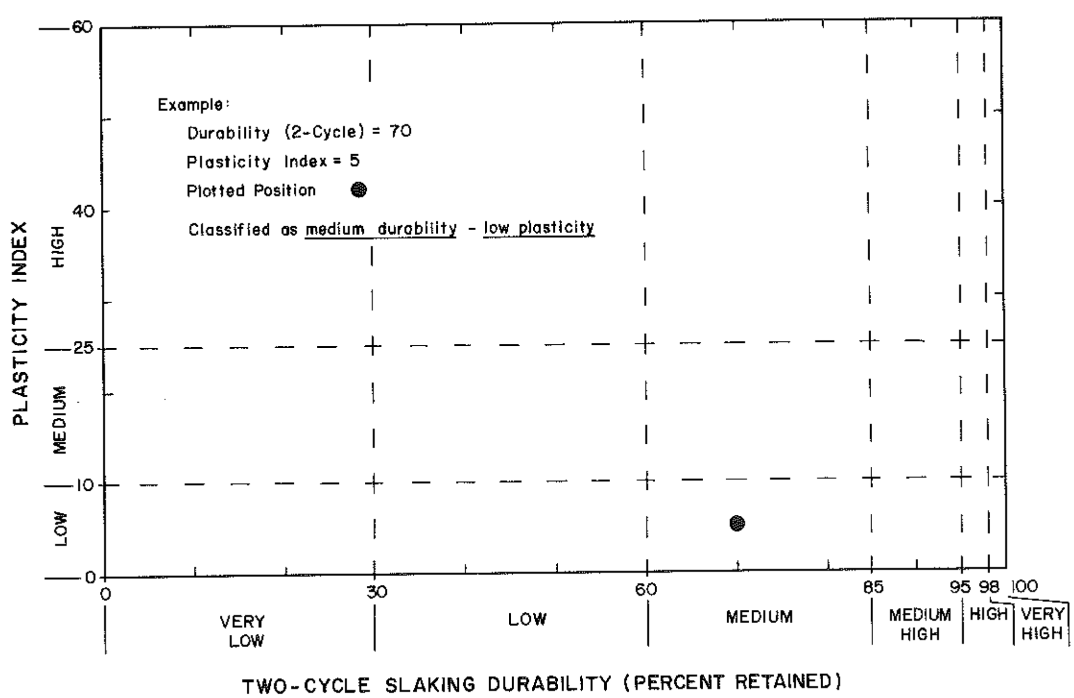


Variables to Consider in Engineering Classifications of Shales and Other Mudrocks (Gamble, 1971).

c) Using apparatus developed by Franklin (1970), Gamble tested numerous samples and proposed a classification based on a two-cycle durability test. The test procedure and apparatus are described elsewhere in this work (APPENDIX E, WEATHERING). Results of Gamble's tests are presented in the slaking durability classification chart included here.



Slaking Durability Classification for One- and Two-Slaking Cycles (Gamble, 1971).



Durability-Plasticity CLassification for Shales and Other Argillaceous Rocks (Gamble, 1971).

d) From correlations of durability index with other properties (water content, liquid limit, dry unit weight, plasticity index, and activity ratio), Gamble concluded that a chart showing the relation between plasticity index and durability index provided the best correlation to use as a basis for classification. Rock samples which have low slake-durability values should be subjected to soils classification tests (Atterberg limits or sedimentation-size analysis).

It appears that transitional materials which fall into the low plasticity range and high or very high strength ranges could be safely designated as rock-like (cemented) material and not subjected to soil mechanics tests. This would provide the distinction necessary to assign a sample to the appropriate testing program.

APPENDIX C

INTACT ROCK CLASSIFICATION SYSTEMS

Woolf, 1951
Harley, 1926
Head, 1951
Panama Canal Company, 1959
Wegehaupt, 1960
Rollow, 1962
Krsmanović and Langof, 1963
Coates, 1964
Miller and Deere, 1966
Coates and Parsons, 1966
Obert and Duvall, 1967
Stapledon, 1968
Duncan and Jennings, 1968
Duncan, 1969
Coates, 1970
Franklin, 1970
van der Vlis, 1970
Cottiss, Dowell, and Franklin, 1971

INTACT ROCK CLASSIFICATION SYSTEMS

ROCK IDENTIFICATION GUIDE (Woolf, 1951)

This identification system is based on the engineer's need to assign to a rock a name of geological significance. The method is intended for use in the field and consequently requires very little apparatus. The rock is placed first into one of five general categories:

I. Glass (wholly or partly),
II. Not Glassy (dull or stoney; homogenous -- fine-grained, grains not visible to naked eye),
III. Distinctly Granular,
IV. Distinctly Foliated (no effervescence with acid), or
V. Clearly Fragmental.

The investigator then categorizes the rock by simple physical or chemical tests. After selecting the proper major group in which the particular rock belongs, further physical and(or) chemical tests enable the engineer to name the rock.

I. Glassy

a) Glassy lūster; hard; conchoidal fracture; colorless, white, or smoky gray.	Quartz
b) Solid glass; brilliant vitreous luster; generally black.	Obsidian
c) Cellular or frothy glass.	Pumice

II. Not Glassy (hardness distinction)

II. A. Softer than Steel	
a) Grains imperceptible; clay odor; laminated; no effervescence.	Shale
b) Brisk effervescence with cold acid.	Limestone
c) Brisk effervescence with heated acid.	Dolomite
d) Soapy feel; translucent edges; green to black.	Serpentine
II. B. Harder than Steel	
a) Light to gray; scratched by quartz.	Felsite
b) Very hard; conchoidal fracture; waxy luster; dark gray to brown.	Chert, Flint
c) Heavy; dark color.	Basalt

III. Granular Rocks (function of hardness and grain uniformity)

III. A. Softer than Steel	
a) Brisk effervescence with cold acid.	Limestone, Marble
b) Brisk effervescence with heated acid.	Dolomitic Marble
III. B. Harder than Steel (grains approximately same size)	
a) Mainly quartz and feldspar; usually light colored.	Granite
b) Mainly feldspar; little quartz; light colored.	Syenite
c) Feldspar and ferromagnesian (dark) minerals:	
1) Mainly feldspar; medium color.	Diorite
2) Ferromagnesian minerals predominant; dark color.	
i. Grains just visible to naked eye.	Diabase
ii. Coarse-grained.	Gabbro
d) Mainly quartz:	
1) Fracture around grains.	Sandstone
2) Fracture through grains.	Quartzite
III. C. Harder than Steel (large distinct crystals in fine-grained matrix) Rocks similar to III. B.	Porphyry

IV. Foliated Rocks (function of grain fineness; break more or less readily along one plane; degree of foliation perfection)

a)	Medium- to coarse-grained; roughly foliated.	Gneiss
b)	Finer grained and foliated.	Schist
c)	Very fine-grained; splits easily into thin slabs.	Slate
V.	**Fragmental Rocks**	
a)	Pebbles embedded in cementing matrix.	Conglomerate
b)	Angular fragments embedded in cementing matrix.	Breccia
c)	Quartz grains, rounded or angular, cemented together; glassy luster; frequently transluscent.	Sandstone

In practice, there has been some difficulty in utilizing this system for the identification of intrusive or coarse-grained igneous rocks. This is probably due to an over-emphasis of the feldspar content of those rocks and not enough emphasis on other mineral constitutents. From the viewpoint of petrographic analysis of intrusive rocks, granite will be the only rock with appreciable amounts of quartz.

GRINDING RESISTANCE (Harley, 1926)

Harley proposed a system of rock classification which considered the following characteristics:

a) unit weight,
b) degree of hardness,
c) degree of toughness, and
d) occurrence of slips (or joints) in the rock mass.

This system is based upon the energy required to drill one cubic inch (16.4 cm^3) of rock, which has previously been correlated to a grinding resistance obtained by a small grinding machine:

	Grinding Resistance Factor	Classification
Hardest Rock	1.0	A^+
Softest Rock	0.1	D^+

This system never gained widespread acceptance.

DRILLABILITY CLASSIFICATION (Head, 1951)

A system essentially similar to that of Harley's (1926) was proposed by Head. This classification of rock formations was based on the relative efficiency with which the formations could be drilled with a small rolling-cutter type test bit. This micro-bit was approximately 2 inches (5.0 cm) in diameter and consisted of two rolling cutters approximately one inch (2.5 cm) in diameter mounted on opposite ends of a shaft at a slight angle with respect to the axis of the shaft. The test bit was designed to facilitate the replacement of the rolling cutters after each drillability test. "The Drillability Classification Number (DCN) was obtained by mounting the micro-bit in a lathe and measuring the time interval, in seconds, required to drill a 1/16-inch depth into a sample of each formation" (Miller and Deere, 1966). For 15 common rock formations encountered in drilling, the DCN ranged from 1.9 for Wilcox Sandstone to 555.7 for Hosston Quartzite. Through empirical evidence, Head concluded that any type of rolling-cutter bit would drill into all formations for which a DCN has been established in the same succession as the test bit if chipping action occurs.

ROCK HARDNESS (Panama Canal Company, 1959)

In 1947, the Panama Canal Company adopted a hardness test applicable to rock material. The scale of rock hardness values was slightly modified and utilized for the classification of rock during the construction of the Balboa Bridge in 1959. The concept is simple; the scale is dependent upon the relative ease or difficulty with which intact rock can be broken.

Rock Hardness

Code	Relative Hardness	Description
RH-1	Soft	Crumbled easily by hand; clay-shales, uncemented sandstones
RH-2	Medium Soft	Not crumbled between fingers, easily picked by light blows of a geology hammer; shales, slightly cemented sandstones
RH-3	Medium Hard	Picked with moderate blows of a geology hammer, cut by a knife
RH-4	Hard	Not picked with a geology hammer, chipped by moderate blows of a geology hammer
RH-5	Very Hard	Chips can be broken off only by heavy blows of a geology hammer

CLASSIFICATION OF CLASTIC ROCKS (Wegehaupt, 1960)

Wegehaupt developed a classification system which divided clastic rocks into three main groupings contingent upon sand content. The study was performed on Carboniferous rocks from the Ruhr District of Germany. An approximately linear relationship was found for sandstones between the sand content and compressive strength:

$$W = 940\ (1 + 0.00332\ s)$$

where W = cube compressive strength, kg/cm^2 and
s = sand content as a decimal fraction.

There was insufficient data for empirical equations similiar to the one above for shales and sandy shales. A strength loss was noted for saturated sandstones and saturated shales, attributable to the presence of clay which tends to swell (Dreyer, 1972).

Classification of Carboniferous Rocks

Rock Type	Sand Content by Weight	Approximate Cubical Compressive Strength (psi)	Approximate Cubical Compressive Strength (MPa)
Shales (ST)	ST≤33.3%	< 14100	< 97
Sandy Shales (SR)	33.3%<SR≤66.7%	14100 - 16200	97 - 112
Sandstones (SN)	66.7%<SN≤100.0%	16200 - 18500	112 - 127

DRILLABILITY CLASSIFICATION (Rollow, 1962)

An empirical drillability classification chart utilizing micro-bit data from samples of the same rock formations on which full-sized bits had been used was developed at the Hughes Tool Company (Rollow, 1963). The laboratory micro-bit test provides a basis for predicting rock drillability from representative samples; but obviously the micro-bit test is valid only to the extent that the micro-specimen is representative of the in-situ rock mass. There is no empirical correction relative to the wear and life of the bit. Probably the major limitation to this rock classification system is "its specified relation to particular types of equipment and procedures for specific rock formations" (Miller and Deere, 1966).

SHEAR STRENGTH CLASSIFICATION (Krsmanovič and Langof, 1963)

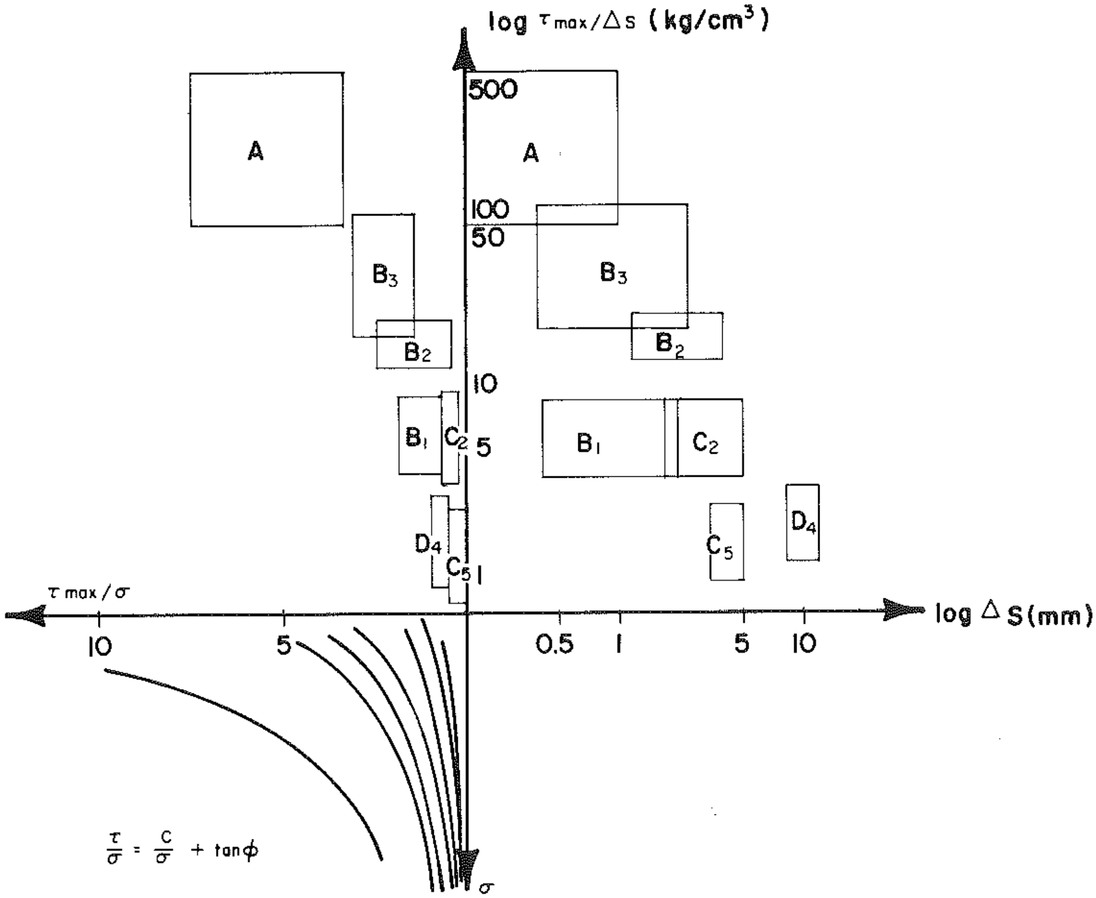
With regard to shear strength, the ratio $\tau_{max}/\Delta S_1$ has proven to be of great significance. Here, τ_{max} is the maximum shear strength and ΔS_1 is the shear strain necessary to obtain the maximum shear resistance. Krsmanovič and Langof suggested plotting

a) $\tau_{max}/\Delta S_1$ versus ΔS_1,

b) $\tau_{max}/\Delta S_1$ versus $\tau_{max}/\epsilon = (C/\sigma) + \tan \phi$, and

c) σ versus τ_{max}/σ.

There is a tendency for different types of rocks to occupy different areas on plots of the above variables. This localization of rock types allows for some rock classification, as can be seen below.



INTACT AND IN-SITU ROCK CLASSIFICATION (Coates, 1964)

Coates listed five rock characteristics which he considered to be the most important with respect to engineering applications. With these properties in mind, he proposed a classification system which encompassed intact and in situ physical characteristics.

Intact Samples

a) Uniaxial Compressive Strength, Q_u
   1) Weak < 5000 psi (< 34 MPa)
   2) Strong 5000 - 25000 psi (34 - 172 MPa)
   3) Very Strong > 25000 psi (> 172 MPa)

b) Pre-failure Deformation (indicates time dependent properties to be expected at stress levels less than those required to produce failure)
   1) Elastic
   2) Viscous (at stress of 50 percent of uniaxial compressive strength, the strain rate is greater than 2 microinches/inch per hour)

c) Failure Characteristics (influences safety factor)
   1) Brittle
   2) Plastic (more than 25 percent of the total strain before permanent set occurs)

In Situ

d) Gross Homogenity
   1) Massive
   2) Layered (parallel lines of weakness)

e) Formation Continuity
   1) Solid -- joint spacing > 6 ft (1.8 m)
   2) Blocky -- 3 in. (7.6 cm) $<$ joint spacing < 6 ft (1.8 m)
   3) Broken -- fragments pass through 3-in. (7.6-cm) sieve

The pre-failure deformation parameter indicates the time-dependent deformation (creep) characteristics. In most cases, according to Coates, the mechanical properties of the rock will be of minor significance compared to the structural aspects of the in-situ rock mass which cause the rock to creep. Specifications for a conventional uniaxial test shown below apply if test results are used to divide rocks into elastic and viscous types:

a) apply stress approximately equal to 0.5 Q_u;
b) loading cycle; record strain readings continuously or at every 1/5 of the load increment;
c) establish load; keep constant until the strain rate is less than 0.2×10^{-8} percent per minute or cumulatively less than 2×10^{-8} percent per hour;
d) unload sample as quickly as possible; maintain the sample at zero stress until the strain rate is less than 0.2×10^{-8} percent per minute or 2×10^{-8} percent per hour;
e) reapply the load to 0.5 Q_u; record strain; unload; record strain.

The strain rate of 2×10^{-8} percent per hour was based on the amount of creep that would be required during one month in a typical drift to produce visible distress in tightly placed sets (Coates and Parsons, 1966).

The failure mode was considered important but difficult to properly characterize. Additionally, the type of failure is sensitive to the stress environment. Since the violence of rupture (failure) varies with the energy stored in the rock before failure, Coates suggested that this quantity could be measured by determining the amount of plastic (irrecoverable) strain as a proportion of the total strain in a uniaxial compression test. The figure of 25 percent of the total strain before permanent deformation was considered prudent as the dividing line for differentiating rock materials into two groups -- elastic and viscous.

STRENGTH AND DEFORMATION CLASSIFICATION (Miller and Deere, 1966)

Miller and Deere proposed an engineering classification and a series of index properties for intact rock specimens. The basis for this system was the strength and deformation characteristics of the rock material:

Strength

Class		Uniaxial Compressive Strength (psi)	Uniaxial Compressive Strength (MPa)
A	Very High Strength	> 32,000	> 220
B	High Strength	16,000 - 32,000	110 - 220
C	Medium Strength	8,000 - 16,000	55 - 110
D	Low Strength	4,000 - 8,000	26 - 55
E	Very Low Strength	< 4,000	< 26

Modulus Ratio

Class		Modulus Ratio
H	High Modulus Ratio	> 500
M	Average Modulus Ratio	200 - 500
L	Low Modulus Ratio	< 200

Modulus Ratio = $E_t/\sigma_{a(ult)}$
where E_t = tangent modulus at 50 percent of ultimate strength and
$\sigma_{a(ult)}$ = ultimate uniaxial compressive strength

There were three basic reasons that a division between the high strength and the very high strength rocks was selected at 32,000 psi (220 MPa):

1. Empirical correlations between the uniaxial compressive strength and the Shore hardness as well as the Schmidt hardness demonstrate changes in slope in the proximity of 32,000 psi (220 MPa). "This relationship may be interpreted as indicating that rocks with compressive strengths in excess of about 32,000 psi have an inter-granular coherence which cannot be measured by hardness tests, but which accounts for very high compressive strengths" (Miller and Deere, 1966).
2. Empirical evidence indicated that most rocks with compressive strengths in excess of 32,000 psi (220 MPa) had essentially linear stress-strain curves.
3. There existed a natural boundary or separation at 32,000 psi (220 MPa) between certain geologic formations:
   a) most foliated, metamorphic rocks (slates, schists, gneisses, and some phyllites) and the common sedimentary rocks demonstrably had strengths less than 32,000 psi (220 MPa).
   b) Very high strength igneous rocks (diabase and dense basalt) are separated from other fine-grained igneous rocks (dacite, rhyolite, and andesite) and most of the granite rocks by the 32,000-psi (220-MPa) level of strength.

As Miller and Deere first conceived their classification system, the modulus of elasticity was to be used directly and subdivided into groupings such as:

Category 1: < 1 x 10^6 psi (6.9 x 10^3 MPa)

Category 2: 1 to 2 x 10^6 psi (6.9 to 13.8 x 10^3 MPa), etc.
This proved to be an unfruitful approach since each strength category would have three or more modulus possibilities and some 20 classifications would result. To prevent this, the authors chose to use the modulus ratio, which they defined as the tangent modulus at 50 percent ultimate strength divided by the uniaxial compressive strength.

STRENGTH AND FAILURE MODE CLASSIFICATION (Coates and Parsons, 1966)

Extensive testing and suggestions from the field caused Coates and Parsons to modify the rock classification system first proposed by Coates in 1964. Realizing that a classification system should indicate strength, compressibility, and continuity of the rock mass, and that these properties are extremely difficult to determine for the in-situ rock mass, the authors recommended that strength and compressibility of the rock material be found by routine tests and that some geological information about the rock mass be used to augment test results:

- Geological Name of Rock
- Uniaxial Compressive Strength
  - Weak: $<$ 69 MPa or 10,000 psi
  - Strong: $>$ 69 MPa or 10,000 psi
- Deformation and Failure Characteristics
  - Elastic
  - Yielding ("if either the relative permanent strain at any stress level exceeded something like 25 percent or if the creep rate under sustained loading exceeded something like 2μ/hr" (Coates and Parsons, 1966)).

In testing, only the rock material is investigated. Properties of the intact sample provide only limited information for field problems since the properties of the rock mass may vary through a great range of values form those of the intact sample to those of the filling material in joints of a loose formation.

The strength of intact samples is "seldom a critical quantity in problems of ground control. Consequently, the original concept was that this property might be usefully divided simply into two groups: weak and strong or possibly with a third group of very strong". It was thought that individuals with some experience in the field could most probably classify the rock with respect to strength by either visual examination or at least by means of a simple empirical relationship with a hardness or rebound test. Although the strength of the rock mass is a significant property "which only in a few cases will be governed by the strength of the rock substance, ... it cannot be expected to be given in a classification system. Although we are still not certain that the property of 'failure characteristics' can be adequately characterized by the relative permanent strain and whether this category can be included in a simple classification system, we are not inclined to eliminate it at the present time without additional work".

STRUCTURAL CLASSIFICATION (Obert and Duvall, 1967)

In general, the first appraisal of the structural-mechanical properties of subsurface rock is the result of either visual examination of exploratory drill cores or of evaluation of laboratory tests on representative rock samples cut from these cores. Although the principal variables of importance with respect to rock structure are the magnitude and direction of the preexisting stresses and the actual rock strengths, it is usually assumed that the preexisting state of stress is principally due to the weight of the overburden rock. Tests of strength of rock substances and other mechanical properties are made on relatively small, uniform intact samples. But as Obert and Duvall indicated, "if the body of rock from which the specimen was taken is correspondingly uniform, classification by compressive strength, or any combination of mechanical properties, has a real value. But in most instances at the scale of an underground opening, rock contains mechanical defects such as joints, fractures, and faults, and as is well know, the in-situ mechanical properties of a body of rock at this scale will depend to some indefinite degree on these defects." It would seem, therefore, that laboratory tests on intact samples do not provide a satisfactory basis for a rock classification system with respect to structural considerations.

Instead of classifying rock simply by means of laboratory tests on intact samples, Obert and Duvall have classified rock for structural purposes with respect to the geological and mechanical properties which would "permit the construction of a specified type of underground structure." This structural classification system for rock was formulated for use in rock structure designs and incorporated designated mechanical and geological rock characteristics. However, the size and depth of underground openings affect the utility of this classification since a low strength rock may be competent at shallow depth but incompetent at a greater depth (i.e., chalk has a relatively low compressive strength of approximately 7 MPa and will sustain a 10-meter diameter unsupported opening at a depth of about 30 meters but is incompetent at depths of 300 meters). Also, a rock may be elastic at one depth but inelastic at another (i.e., salt is relatively elastic at a depth of 30 meters but inelastic at 300 meters).

Structural Classification of Rock*

Descriptive Term	Typical Rock Types	Rock Quality Designation (RQD)
I. Competent		Virtually 100%
A. Massive		
1. Elastic	Thick-bedded sandstone and limestones; massive marbles, quartzites, granites, and gabbros; bonded and jointed igneous and metamorphic rocks	
2. Inelastic	Evaporite minerals: halite (salt), trona, potash, and borate ore	
B. Laminated (Bedded)		Almost 100% between partings
1. Elastic	Mostly sedimentary rocks not included in Category I.A.1. -- bedded rocks in which the laminae are not cemented; some metamorphic rocks: foliated quartzites, schists, and gneisses	
2. Inelastic	Most coals; some halites and potashes; oil shales	
C. Jointed		Almost 100% between joint planes
II. Incompetent		Very low

*Extension of L. Obert, W. I. Duvall, and R. H. Merrill, *Design of Underground Openings in Competent Rock,* **Bulletin** 587, US Bureau of Mines (1960).

Competent Rock -- any rock which, because of its mechanical and geological characteristics, is capable of sustaining underground openings without the aid of any structural support except that provided by unmined rock in the form of pillars and sidewalls (stulls, light props, and rock bolts are not considered structural supports).

Massive -- implies that the spacing between joints, partings, faults, etc., is relatively equal to or greater than the critical dimensions of the underground opening or that the bond strength across partings or joints is comparable to the rock strength.

Competent, Massive, Elastic Rock -- competent, massive rock which requires no remedial treatment or which shows only negligible time-dependent effects in an underground structure.

Competent, Massive, Inelastic Rock -- competent, massive rock which demonstrates a tendency to flow or creep (evidenced by roof sag, floor heave, pillar shortening, etc.)

Competent, Laminated, Elastic Rock -- competent rock including all thinly laminated but relatively elastic sedimentary rocks in which the laminae are separated by and(or) divided into approximately parallel planes of weakness.

Competent, Laminated, Inelastic Rock -- competent rock wherein the rock material within the laminae is inelastic (openings in this rock are subject to floor heave or roof sag).

Competent, Jointed Rock -- competent rock with more than one set of virtually parallel planes of weakness.

Incompetent Rock -- rock incapable of sustaining unsupported underground openings; e.g., containing more than one geologically distinct system of joints or closely spaced random joints.

Joint -- break of geological origin in continuity of rock with no displacement parallel to plane of breakage.

Fracture -- fresh (unweathered), unbonded break in continuity of rock with no displacement and not oriented in a regular system (often manmade).

Parting -- thin layer of deposited or altered material separating beds in sedimentary or metamorphic rocks -- generally unbonded, but indurated depositional materials may cause bonding.

Separation -- relatively fresh break along bedding plane or between beds (usually manmade).

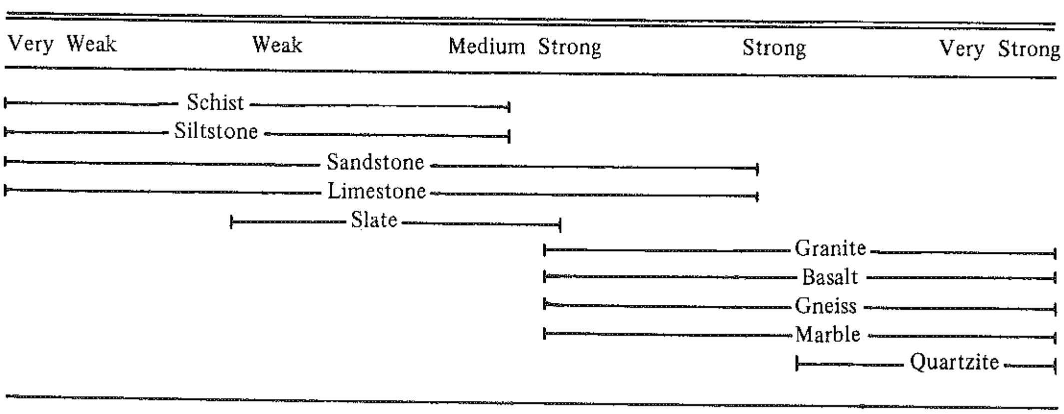
STRENGTH CLASSIFICATION (Stapledon, 1968)

In a discussion of Coates' paper entitled "Classification of Rocks for Rock Mechanics" (1964), Stapledon introduced the following rock classification system based on unconfined compressive strength:

Classification of Rock Materials

Category	Unconfined Compressive Strength (psi)	(MPa)
Very Weak	< 1,000	< 7
Weak	1,000 - 3,000	7 - 21
Medium Strong	3,000 - 10,000	21 - 69
Strong	10,000 - 20,000	69 - 138
Very Strong	> 20,000	> 138

This classification system, in conjunction with careful visual examination and simple field tests, will prove adequate generally to decide upon the strength range term for most rock materials. The range of strength of some common rock materials are:



According to Stapledon, dry samples should be used since "some rocks in the 'medium strong' and 'weak' ranges lose up to 80 percent of their strength after soaking in water for 1 or 2 weeks." Additionally, Stapledon mentioned that it might be wise to include Coates and Parsons' criteria for "elastic" and "yielding" materials as set forth in their 1966 paper.

ROCK INDURATION CLASSIFICATION (Duncan and Jennings, 1968)

It has been stated by many investigators that rock strength characteristics are of more importance in terms of rock engineering than the rock texture or geologic origin. To this end, Sowers and Sowers (1970) describe a rock classification system in terms of unconfined compression strengths and simple field tests, first developed by Duncan and Jennings:

Hardness	Unconfined Compressive Strength (psi)	Unconfined Compressive Strength (MPa)	Field Test
Very Hard	> 20,000	> 138	Difficult to break a 4-in. (10-cm) piece with a pick
Hard	8,000 - 20,000	55 - 138	4-in. (10-cm) piece broken with one blow of geology hammer
Soft	2,500 - 8,000	17 - 55	Pick point may scratch or dent specimen
Very Soft	1,000 - 2,500	7 - 17	Crumbled with a pick, easily scratched with a knife

VISUAL AND LABORATORY TESTING CLASSIFICATION SYSTEM (Duncan, 1969)

Duncan devised a rock classification system based upon visual observations as well as laboratory tests on intact samples. The visual observations may be done in the field or in the laboratory and the investigator need not be a geologist to use the system.

Visual Rock Classification[a]

Group Number	Texture	Structure	Composition	Color[b]	Grain Size[c]
I	Crystalline	Homogeneous (h)	Non-calcareous (N)		Coarse (a)
II	Crystalline-indurated	Lineated (*l*)	Part Calcareous (P)		Medium (b)
III	Indurated	Intact-foliated (i)	Calcareous (C)		Fine (c)
IV	Compact				
V	Cemented				
VI		Fracture-foliated (f)			

[a]One term is selected from each column.

[b]After Group Number and Composition letter, place a
(1) for light colored or a
(2) for dark colored.

[c]Regularity:
(x) for equigranular
(y) for inequigranular

Crystalline: No spalling when scratched with knife blade; rock consists entirely of interlocking crystals or interlocking crystal grains visible to the naked eye.

Crystalline-indurated: No spalling when scratched with knife blade; isolated crystals or crystal grains visible to naked eye and embedded in an indurated matrix.

Indurated: No spalling when scraped with knife blade; no interlocking crystals nor interlocking crystal grains visible to naked eye; may be coarse-, medium-, or fine-grained.

Compact: Spalling occurs when sample scraped with knife blade; individual crystals and grains invisible to naked eye.

Cemented: Spalling occurs when sample scraped with knife blade; crystals and grains visible to naked eye.

Homogenous (h):	Random crystal and grain arrangement; no visible linear or planar structure.
Lineated *(l)*:	Linear rather than planar particle orientation.
Intact-foliated (i):	Visible planar structure; no closed or incipient fracture.
Fracture-foliated(f):	Visible planar structure; closed or incipient fracture (cleavage planes or bedding planes).
Non-calcareous(N):	No calcium carbonate; sample not reactant with dilute HCl.
Part-calcareous(P):	Non-calcareous material (quartz or clay matter) present in substantial amounts; some rocks react with dilute HCl.
Calcareous(C):	Calcium carbonate is main constituent; reacts with dilute HCl (effervesces).
Light (1):	No description given by Duncan (1969).
Dark (2):	For non-calcareous rocks; black or gray materials.
Dark (2):	For calcareous rocks; materials of a "muddy" composition.
Coarse-grained(a):	Particles > 2 mm in diameter; particles easily visible to naked eye.
Medium-grained(b):	0.1 mm < particles < 2 mm; particles visible to naked eye.
Fine-grained(c):	Particles < 0.1 mm; particles invisible to the naked eye.
Equigranular(x):	Grains approximately the same size throughout.
Inequigranular (y):	Range of grain sizes throughout.

As an example, a rock classified as Group III i (N)1 would be indurated, intact-foliated, non-calcareous, and light colored; a rock classified as VI would be fracture-foliated.

The second aspect of Duncan's rock classification system involves laboratory tests on intact samples. Index properties are chosen to define

a) the nature of the solid mineral grains within the rock material;
b) the nature and extent of the voids within the mineral aggregate which comprises the rock material; and
c) the nature of the bond, if any, existing between the solid mineral grains.

To accomplish these laboratory determinations, specific index tests are needed. These tests establish

a) the saturation moisture content (i_s),
b) the dry apparent specific gravity (G_b), and
c) the saturation swelling coefficient (ϵ_s).

Following the selection of properties indicative of the rock aggregate, voids, and bonding, and the specific index tests necessary to describe these properties, Duncan then devised a classification system based upon the laboratory tests.

Laboratory Tests for Rock Classification

Direct determination of

a) natural moisture content (w),
b) saturation moisture content (i_s),
c) dry apparent specific gravity (G_b) and dry density,
d) saturated apparent specific gravity and saturation density,
e) bulk density at natural moisture content,
f) solid mineral grain specific gravity,
g) swelling coefficient (oven-dried condition to fully saturated), and
h) swelling coefficient (natural moisture content to fully saturated).

Indirect determination of

a) porosity and
b) void ratio.

As Duncan indicated, determination of the dry specific gravity, saturation moisture content, and the saturation swelling coefficient of rock materials permit field identification and rock classification to be verified or amended. The relationship between rock classification test values (G_b, i_s, and ϵ_s) and field descriptions is depicted below:

Group	Texture	G_b	i_s (percent)	Swelling Characteristics
I	Crystalline	> 2.5	< 2.0	Non-swelling
II	Crystalline-indurated	> 2.5	< 2.0	Non-swelling
III	Indurated	> 2.5	< 2.0	Non-swelling
IV	Compact	2.0 - 2.5	2.0 - 15.0	Swelling
V	Cemented	2.0 - 2.5	> 2.0	Non-swelling
VI	Any of the above			

Rationale for Laboratory Tests

Properties are selected which are indicative of

a) the nature of the solid constituents within the rock material,

b) the nature and extent of the voids within the mineral aggregate which comprises the rock material, and

c) the nature of the bond, if any, between the solid mineral grains.

These properties include:

a) Solid mineral grain specific gravity, G_s (dimensionless)

$$G_s = W_s/V_s\gamma_w = \gamma_s/\gamma_w$$

where W_s = weight of solid rock material,
V_s = volume of solid rock material,
γ_s = unit weight of rock material, and
γ_w = unit weight of water.

b) Porosity, n (as a percentage)

$$n = 100\ V_v/V$$

where V_v = volume of voids and
V = total volume of rock material.

c) Void ratio, e (dimensionless)

$$e = V_v/V_s = n/(1 - n)$$

where $V_s = V - V_v$.

d) Moisture content, w (as a percentage)

$$w = 100\ W_w/W_s$$

where W_w = weight of water and
W_s = weight of solids (oven dry; 105 C for 12 hours).

e) Saturation moisture content, i_s (as a percentage)

$$i_s = 100\ W_w/W_s$$

where W_w = weight of water at 100 percent saturation. i_s is used to indicate index of alteration or void index.

f) Dry apparent specific gravity, G_b (dimensionless)

$$G_b = W_s/V\gamma_w = \gamma_d/\gamma_w$$

where γ_d = dry unit weight of rock material (dry density).

g) Saturated apparent specific gravity, G_γ (dimensionless)

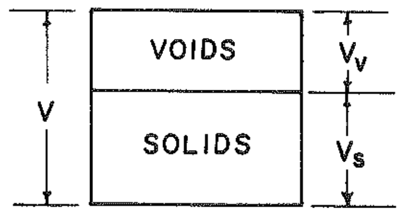
$$G_\gamma = (W_s + W_w)/V\gamma_w = \gamma_{sat}/\gamma_w$$

where γ_{sat} = unit weight of saturated rock material (saturated density).

Through a series of considerations, index properties can be chosen. Assume that no volume change occurs during the process of absorbing water. Consider a volume, $V = 1\ cm^3$. At saturation,

$$i_s = W_w/W_s = W_w/G_\gamma V\gamma_w \quad (1 \text{ gram water} = 1\ cm^3)$$

or



$$i_s = W_w/G_\gamma \text{ since } V = \gamma_w = 1 \text{ in cgs.}$$

For a saturated condition, $W_w = V_w = V_v = 1$ and

$$i_s = W_w/G_\gamma = V_v/G_\gamma \text{ or } V_v = i_s\, G_\gamma.$$

Since $V = 1$ and $n = V_v/V = i_s G_\gamma/V$,

$$n = i_s G_\gamma \text{ or } n = V_v.$$

Knowing that $V_s = G_\gamma/G_s$ and $e = V_v/V_s$, then $e = i_s G_s = n/(1 - n)$. Therefore,

$$e = i_s G_s = i_s G_\gamma/(1 - i_s G_\gamma)$$

and

$$G_s = G_\gamma/(1 - i_s\, G_\gamma) = G_\gamma/(1 - n).$$

On the basis of these considerations, the most useful index properties seem to be

i_s = saturation moisture content and
G_γ = dry apparent specific gravity.

These index properties allow the calculation of G_s, e, and n.

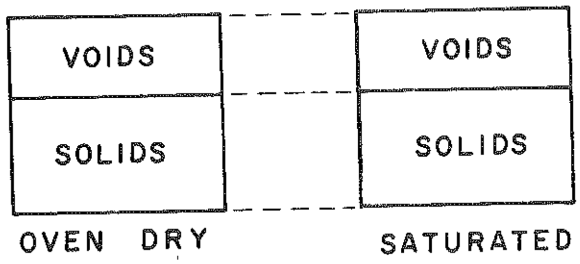
Thus, with these two index properties, the rock mineral grains are described. An index test is also needed for evaluation of bonding. A simple, free-swelling test is a measure of bond. To this end, a swelling coefficient is defined as

$$\epsilon_s = dl/l;$$

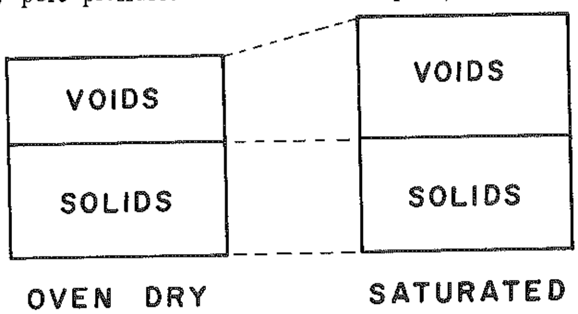
i.e. the change in length of an oven-dried sample when allowed to absorb water to saturation divided by the oven-dry length.

Empirical evidence indicates that, in general, rock materials fall into three categories:

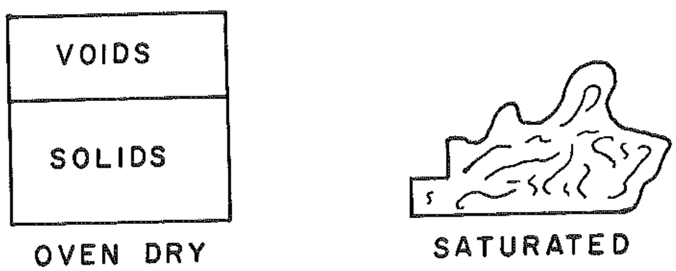
a) Rigid (no change in length)
$\epsilon_s = O/l = 0$



b) Flexible (capillary pore pressures cause voids to expand)
$\epsilon_s > 0$



c) Disintegrated



In terms of a visual classification, "rigid" rocks may be considered to be crystalline, crystalline-indurated, indurated, or cemented. 'Flexible" rocks are compact (very weakly cemented rocks which swell on contact with water).

STRENGTH AND DEFORMATION CLASSIFICATION (Coates, 1970)

Coates (1970) proposed a general classification system containing as most important the following properties:

a) strength of rock material,
b) deformation characteristics of the rock material prior to and after failure, and
c) gross homogeneity and isotropy of the rock mass.

This system, then, takes both intact sample and in-situ mass properties into account. Specifically, the system included:

Tests of Rock Material

1.	Strong	> 10,000 psi (69 MPa) compressive strength
2.	Weak	< 10,000 psi (69 MPa) compressive strength
1.	Elastic	No time-dependent characteristics and a brittle failure mode
2.	Yielding	Swell or creep characteristics and more than 25 percent of the strain at any stress level is not recoverable

Tests or Measures of a Rock Mass

Description	Joint Spacing
1. Massive	> 6 ft (1.8 m) -- spacing
2. Layered	
3. Blocky	1-6 ft (0.3 - 1.8 m) -- layers
4. Broken	< 1 ft (0.3 m) -- blocks
5. Very Broken	< 3 in. (7.6 cm) -- fragments

This system can be used to describe a rock in general terms, but Coates suggested that if a geological rock type name can be easily established, then that name should be used. It has been suggested (Miller and Deere, 1966) that the strength divisions are rather arbitrary and at least should be expanded to include:

Description	Compressive Strength (psi)	Compressive Strength (MPa)
1. Very Strong	> 25,000	> 172
2. Strong	10,000 - 25,000	69 - 172
3. Weak	5,000 - 10,000	34 - 69
4. Very Weak	< 5,000	< 34

Presently, it is extremely difficult to test the strength and deformation characteristics directly under in-situ conditions. Therefore, use of the intact samples, generally taken from drill cores, at least gives an indication of the maximum strength parameters for a given formation as suggested below:

Intact		In Situ
Weak	then	Certainly Weak
Strong	either or	Strong Weak (due to structural anomalies)

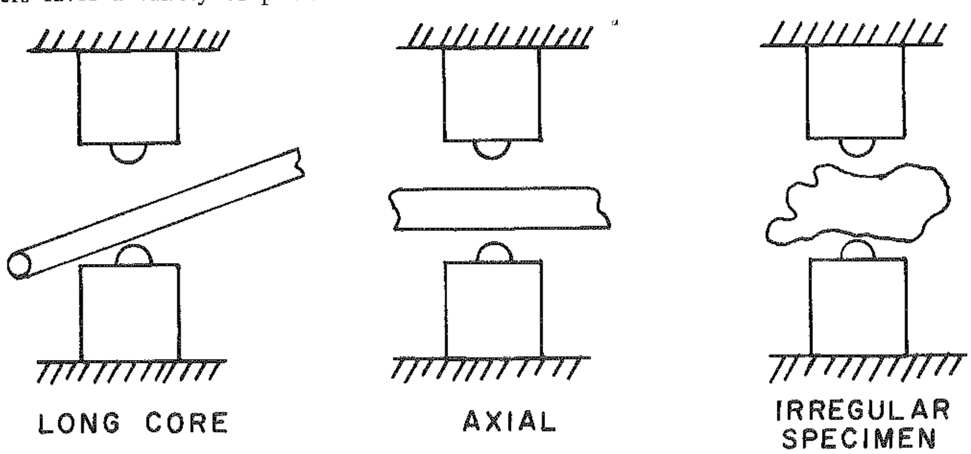
HIGHWAY CLASSIFICATION SYSTEM (Franklin, 1970)

The classification system proposed by Franklin appears to be applicable to highway engineering use. It was developed for use in a road construction project and has been used successfully at least five times subsequent to its development. The main outline is as follows:

Property	Index	Test
1. Brokenness	Fracture Index	Observation and Measurement
2. Strength	Strength Index	Point Load Test
3. Durability	Slake Durability Index	Slake Test
4. Mineralogy		Observation
5. Texture		Observation

where Fracture Index is the average linear size of blocks that comprise the rock mass, Point Load Strength Index the ratio of applied force at failure to squared distance between loaded points, in a point load test, and Slake Durability Index is the percentage ratio of the final weight of rock lumps subjected to drying, then slaking (soaking) in a dispersing agent (two percent solution of sodium hexametaphosphate) for one hour compared to the initial dry weight of the sample.

The Fracture Index accounts to a degree for the fissured state of the in-situ rock mass; but as Franklin suggested, "a more complete description would include such fissure properties as orientation, roughness, openness, continuity, filling and alteration". It appears that a Fracture Index as defined by Franklin would be difficult to determine in practice. To avoid time-consuming sample preparation, Franklin and others favor a variety of point load tests:



The Point Load Index is a function of direct tensile strength. The Durability Index should correspond to resistance to weathering effects. Resistance of the rock material to weathering in terms of strength reduction is obviously important when considering exposed rock surfaces. Franklin also discussed the mineralogical assemblage of the rock in terms of an engineering classification:

Name	Rocks	Engineering Property	Indices
1. Quartzofeldspathic	Acid igneous rocks, quartz, arkose sandstones, gneisses, and granulites	Usually strong and brittle	Porosity, quartz-feldspar ratio, feldspar freshness
2. Lithic/Basic	Basic igneous rocks, lithic and graywacke sandstones, and amphibolites	Usually strong and brittle	Porosity, texture, quartz content, freshness of dark minerals
3. Pelitic (clay)	Mudstones, slates, and phyllites	Often viscous, plastic, and weak	Porosity, density, durability, and quartz and clay contents
4. Pelitic (mica)	Schists	Often fissile, and weak	Porosity, fissility, and mica and quartz contents
5. Saline/Carbonate	Limestones, marbles, dolomites, and salt rocks	Sometimes viscous, often plastic and weak	Porosity, texture, and mineral types

Franklin also suggested that, in addition to the five mineralogical categories just listed, an indication of the relative amounts of mineral types present may be useful. In terms of engineering mechanical quality, minerals likely to be of importance are:

Franklin's Mineral	Hardness	Moh's Hardness Scale Mineral
1. Quartz	7	Quartz
2. Dark Grains		
3. Fresh Feldspars	6	Feldspar
4. Salts and Carbonates	3 - 5	Apatite, Fluorite, Calcite
5. Altered Minerals		
6. Micas and Platy Minerals	2	Gypsum
7. Clay Minerals	1	Talc

"The only textural parameter commonly used in engineering descriptions of intact rock is grain size, but the value of this property as an index to mechanical behavior may be questioned" (Hagen, 1972). Of greater mechanical importance would seem to be such items as sorting, crystallinity, and porosity which Franklin takes into account by noting the proportions of four textural constituents (Schwalb, 1969; Mesri and Gibala, 1972):

1. coarse, fragmented material (detrital or clastic),
2. coarse, crystalline material,
3. muddy, microcrystalline material, and
4. pore space.

These constituents could be represented by a volumetric percentage and size value. Although there is no convenient isotropic classification system in use (1974), some type of textural description is necessary. To this end, Franklin proposed the following rock categories:

1. isotropic,
2. oriented (preferred grain orientation), and
3. segmented (differing grain size or mineral content).

It was also noted that coherence, fissility, or friability of the rock material may be noted and used in classifying the rock.

BRINELL HARDNESS CLASSIFICATION(van der Vlis, 1970)

Empirical relationships between Brinell Hardness Numbers (BHN) and elastic moduli of rock induced van der Vlis to develop a rock classification system which "allows the estimation of elastic moduli from a combination of visual examination and the hardness qualification". Coates (1964) suggested that rock materials might be classified by means of visual examination and(or) some simple empirically-oriented hardness or rebound test which would allow strength correlations. To measure the hardness of intact smaples, a ROHA-tester was used. This is a test in which a spherical steel indenter in contact with the material is subjected to a specific load, and the depth of penetration of the sphere is measured. The BHN is defined as the ratio of load to spherical area of indentation:

$$BHN = L/Dh$$

where
L = load (kg),
D = diameter of sphere (mm), and
h = depth of penetration (mm).

The impression is assumed to be spherical.

Van der Vlis proposed a system based upon visual examination and a simple compression indication using fingers and forceps. He further concluded that the hardness of rock could be measured adequately by means of a steel ball indenter. Additionally, an empirical relationship appeared to exist between the Brinell Hardness Number and the elastic moduli of rock, thereby enabling one to estimate Young's modulus from hardness tests. With this information, van der Vlis designed a simple classification system based entirely on visual examination and hardness testing:

Appearance	Descriptive Term	BHN (kg/mm^2)	E (MPa)	G (MPa)
No cementing material present	Unconsolidated	< 2	< 0.7	< 0.25
Pieces can be easily crushed with the fingers	Loosely consolidated	2 - 5	0.7 - 0.9	0.25 - 0.35
Pieces can be crushed when rubbed between the fingers	Friable	5 - 10	0.9 - 1.2	0.35 - 0.50
Pieces cannot be crushed with fingers, but forceps will crush them	Consolidated	10 - 30	1.2 - 2.6	0.50 - 1.00
Pieces cannot be broken with forceps	Medium Hard	30 - 50	2.6 - 4.0	1.0 - 1.5
	Hard	50 - 125	4.0 - 9.2	1.5 - 3.5
	Very hard	> 125	> 9.2	> 3.5

The empirical relationships between BHN and E and G enable the rock materials to be placed into finer subdivisions than Coates' original subdivisions (1964) but without losing the advantage of simplicity through visual examination and simple testing. In this respect, quantification with the help of the very simple ROHA-tester meets the need for finer subdivisions as expressed by Stapledon (1968) in a discussion of Coates' experimental criteria. As van der Vlis indicated, the real advantage of this rock classification system is that it enables the engineer to estimate the elastic region of the rock material in question (Cottiss, Dowell, and Franklin, 1971).

ROCK QUALITY SCORE (Cottiss, Dowell, and Franklin, 1971)

Cottiss, Dowell, and Franklin collaborated on a system having four descriptive criteria for use in classifying rock masses:

Criterion	Test
1. Fracture spacing in rock cores (direct observation)	RQD and velocity ratio
2. Fracture orientation	Observation
3. Roughness	Observation
4. Infilling	Velocity ratio

Since they are of greatest importance to the mechanical character of rock, the following three properties should be measured in some form:

1. brokenness -- obtained from measurements of fracture spacing that were obtained as an integral part of core loggings,
2. hardness -- mechanical competence of intact material, and
3. durability -- susceptibility of shales and mudstones to weakening and disintegration in water.

They also noted that "the strength of blocks, as well as their size, must be included in even the simplest of rock classifications".

The authors used nine index tests to achieve a rock quality (hardness) score: dry fracture, wet fracture, slake loss (percent), dry specific gravity, porosity (percent), sound velocity, Schmidt rebound hardness, Brazilian strength, and the uniaxial compressive strength. The rock quality score is calculated from

$$\text{Rock Quality Score} = 0.5 + 20\ N\ /\ 9\ T$$

where N = sum of all coded values for the rock sample and
T = number of test results included.

In particular, ranges of all test values obtained for each index test were divided into nine groupings such that an equal number of test values fell into each grouping. Each sample was allotted a coded score between 1 and 9 for each of the nine test results. Except for porosity and the slake loss index, the indices contained values sorted into ascending order of magnitude. In this manner, all rock samples which slaked would score less than 20. In addition, if a specimen yielded a positive slake loss percentage, then its dry specific gravity and porosity values would be zero since sample preparation included water immersion which would be detrimental to the sample's integrity. Being zero, both the dry specific gravity and porosity values would not contribute to the rock quality score. If instead, the slake loss percentage were equal to zero, then both the slake loss and wet fracture would not be used in the calculation and the rock quality score computation would be altered slightly:

$$\text{Score} = 10.5 + 90\ N\ /\ 9\ T.$$

The index tests were defined as:

a) dry fracture -- undefined.
b) wet fracture -- sample immersed in two percent aqueous solution of sodium hexametaphosphate for one hour to simulate weathering of clay material, then washed over a No. 7BS sieve; visually compare material retained on sieve with a set of standards on a scale from 1 (completely dispersed) to 10 (intact specimen).
c) slake loss -- weight of rock material washed through sieve after soaking in deflocculating agent (expressed as a percentage of the total dry weight).
d) dry specific gravity -- sample dry weight divided by the weight of an equal volume of water.
e) porosity -- sample pore volume divided by total volume (expressed as a percentage) where the pore volume is equal to the saturated-surface-dry weight minus the oven-dry weight divided by the product of water density and gravitational acceleration.

f) sound velocity -- the length of the specimen divided by the travel time of the P wave through the specimen (this test only carried out for specimens suitable for the uniaxial compression test).

g) Schmidt hardness -- after impact upon the rock specimen, a rebound number is measured which is the rebound height of a spring-driven plunger expressed as a percentage of the orignal spring compression.

h) Brazilian strength -- specimen discs sawed to a thickness/diameter ratio less than one:

$$\text{Strength} = 2\ F/(\pi\ D\ T)$$

where F = load at failure,
D = disc diameter, and
T = disc thickness.

i) uniaxial compression test -- specimens sawed to a aspect ratio of 2:1:

$$\text{Strength} = 4F/(\pi D^2).$$

APPENDIX D

IN-SITU ROCK CLASSIFICATION SYSTEMS

Terzaghi, 1946
Talobre, 1957
U. S. Bureau of Mines, 1962
John, 1962
Onodera, 1962
Deere, 1963
Lane, 1964
Deere, 1964
Coates, 1964
Knill and Jones, 1965
Deere, Hendron, Patton, and Cording, 1966
Ege, 1967
Obert and Duvall, 1967
Scott and Carroll, 1967
Merritt, 1968

IN-SITU ROCK CLASSIFICATION SYSTEMS

TUNNEL ROCK CLASSIFICATION SYSTEM (cf. Terzaghi, 1946)

This rock classification system was proposed for use in determining the appropriate kind and amount of tunnel support necessary within a construction site. Utilizing mining terminology to categorize the in-situ rock, Terzaghi developed descriptive categories based on qualitative joint spacing and weathering characteristics. Adaptations of this system have been used by many authors (Coates, 1964; Miller and Deere, 1966; Merritt, 1968):

Rock	Descriptive Characteristics
Intact	No joints
Stratified	Individual strata; minimal strength between beds
Moderately Jointed	Jointed mass; cemented or strongly interlocked; vertical walls require support
Blocky and Seamy	Jointed mass; no joint cementing action; vertical walls require support
Crushed	Rock reduced to sand-like particles; chemically unweathered
Squeezing	Rock contains minerals with low swelling capacity; rock advances into a tunnel without perceptible volume increase
Swelling	Rock contains minerals with high swelling capacity; exhibits volume increase

Rock tunnel excavation is an extremely specialized construction activity, and as would be expected, a rock classification for tunnel operations is not wholely applicable to a generalized system. However, this system is currently in use with slight modifications for other types of underground construction projects (Morris, 1972). The modifications increase the usefulness of the original system by ascribing ranges of rock strength, deformation, and failure characteristics to the in-situ rock.

WEATHERABILITY CLASSIFICATION (Talobre, 1957)

In a system similar to Terzaghi's tunnel rock classification, Talobre in essence omitted the moderately jointed rock category and retained Terzaghi's six other rock categories, describing them in terms of qualitative weathering susceptibility (Miller and Deere, 1966; Iliev, 1966). In this manner, intact rock may be regarded as unweathered and only minimally susceptible to weathering effects over the life of the project, while crushed rock would be chemically unweathered but mechanically weathered or broken into small particles.

Neither Talobre's nor Terzaghi's systems are concerned particularly with mechanical properties of rock. However, the qualitative description of the rock mass is informative.

CLASSIFICATION OF COMPETENT ROCK (U. S. Bureau of Mines, 1962)

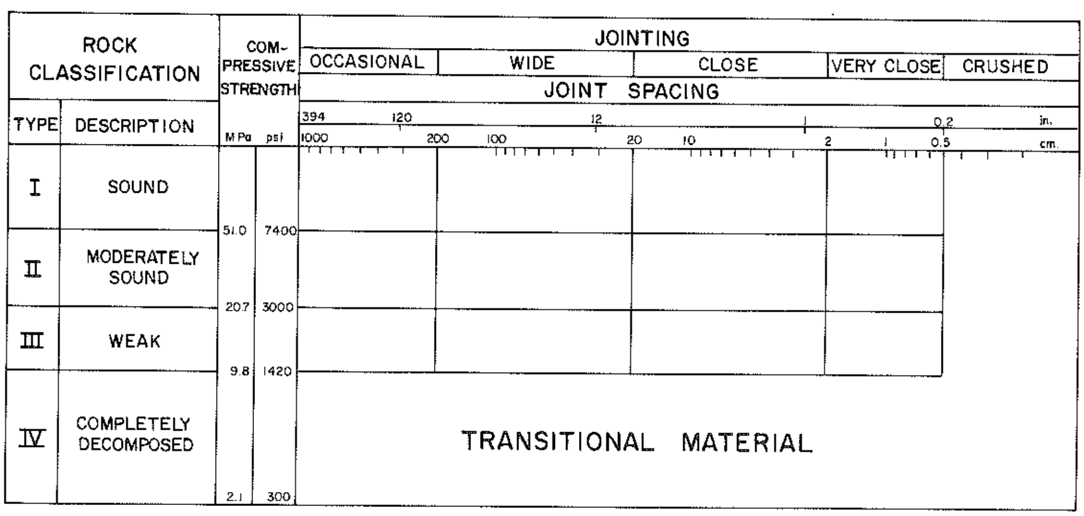
To determine the appropriate size-shape relations for underground openings and amount of artificial support required, the U. S. Bureau of Mines divided in-situ rock into two major groupings; competent and incompetent (U. S. Bureau of Mines, 1962):

Rock		Rock Characteristics
I.	Competent	Sustains underground openings without artificial support
	a. Massive-elastic	Homogeneous and isotropic
	b. Bedded-elastic	Homogeneous, isotrophic beds with bed thickness less than span of opening
	c. Massive-plastic	Flows under low stress
II.	Incompetent	Requires artificial supports to sustain an opening

The allowable use of underground rock is determined by the type of rock present, its physical and geological characteristcs, and the stress field before and during the mining project. Unfortunately, like preceding systems, this system provides no quantitive means for establishing the degree of competency or incompetency of a rock mass.

STRENGTH AND STABILITY CLASSIFICATION (John, 1962)

One of the first general in-situ rock classification systems was proposed to include a description of rock with respect to intact compressive strength, weathering, and joint spacing. John (1962) designed a system for strength and stability analysis of a particular rock mass:



The moderately sound group of rock was further described as somewhat weathered and the weak rock group as decomposed and weathered. This system, developed in the Austrian School of Rock Mechanics; has not gained widespread acceptance in the United States because intact sample compressive strength is not considered an appropriate parameter upon which to base an in-situ classification system.

DYNAMIC INVESTIGATION (Onodera, 1962)

In an attempt to express technically significant rock properties numerically, Onodera proposed a description of rock masses utilizing the in-situ dynamic modulus and the intact sample modulus of elasticity measured in the laboratory. The ratio of these moduli was defined as the soundness of the rock (this was later modified to become the velocity ratio):

$$\text{Soundness} = \epsilon_d / E_d$$

where ϵ_d = in-situ dynamic modulus and
E_d = laboratory determined modulus of elasticity.

In unjointed material, ϵ_d is approximately equal to E_d.

The "crack coefficient", first introduced by Kudo (cf. 1959) in discussing the evaluation of foundation rocks in situ for dams, was used to categorize in-situ rock according to the degree of decrease in elasticity due to the presence of faults, joints, cracks, and other interstices (Onodera, 1962):

$$\text{Crack Coefficient} = (E_d - \epsilon_d)/E_d.$$

Combining the soundness and crack coefficients, Onodera devised a soundness classification of in-situ rock:

Class	Grade	Soundness	Crack Coefficient
A	Excellent	> 0.75	< 0.25
B	Good	0.50 - 0.75	0.25 - 0.50
C	Available	0.35 - 0.50	0.50 - 0.65
D	Deficient	0.20 - 0.35	0.65 - 0.80
E	Bad	< 0.20	> 0.80

In conjuction with this classification system, Onodera described the various classes of rock in terms of "geological diagnostics":

Class	Geologic Description
A	Fresh; no alteration; almost no joints, etc.
B	Jointed or cracked but only slight partings; weathering present only on parting surfaces
C	Parted by joints or cracks with or without minimal interstitial clayey matter; fresh but joints weathered
D	Partings are wide and open and ususally accompanied by fissure water; rock more weathered
E	Advanced weathering; conspicuously jointed, cracked, or crushed

Unfortunately, these geological diagnostics provide no indication of the frequency of joints, an important factor in the structural evaluation of dam foundations.

JOINT SPACING AND BEDDING THICKNESS CLASSIFICATION (Deere, 1963)

Discontinuities (defects) in the rock mass include solution channels and cavities, shear zones, slickensides, dikes, and porous zones. Major engineering effects of these discontinuities are existence of zones of weakness and creation of zones of concentrated (high) stress conditions.

Deere (1963) recommended a series of quantitative descriptive terms to categorize joint spacing and bedding thickness:

Joints	Joint Spacing or Bed Thickness (in.)	(cm)	Bedding
Very Close	< 2	< 5	Very Thin
Close	2 - 12	5 - 30	Thin
Moderately Close	12 - 36	30 - 90	Medium
Wide	36 - 120	90 - 300	Thick
Very Wide	> 120	> 300	Very Thick

Deere also recommended that individual rock core lengths be measured and related to joint spacing and bedding thickness. Core breakage due to faulty drilling techniques can be easily discounted in most cases. Complete core descriptions should be a part of the engineering record, including such information as the nature of joint infiltration material, surface irregularities (plane, curved, or irregular), and degree of smoothness (slick, smooth, or rough) (Deere, 1963).

ROCK QUALITY CORRELATIONS WITH JACKING AND SEISMIC TESTS (Lane, 1964)

During the course of the site investigation of the Latiyan Dam, Iran, Lane (1964) correlated results of in-situ jacking and seismic tests with various grades of rock quality. This particular area of Iran contained three primary rock types -- quartzite, sandstone, and shale. Consequently, the rock grade classification system was an engineering adaptation to facilitate construction of the dam in this location:

Rock	Grade	E_1 (10^{-5} psi)	E_1 (kPa)	E_2 (10^{-5} psi)	E_2 (kPa)	V_1 (ft/sec)	V_1 (m/s)	V_1/V_2	Mean Core Recovery (percent)
Quartzite	I	8.5	58.6	21.0	144.8	12400	3780	0.72	--
and	II	6.4	44.1	14.0	96.5	11100	3383	0.69	75
Sandstone	IV	2.1	14.5	5.8	40.0	8500	2591	0.59	55
	V	2.6	17.9	2.9	20.0	6600	2012	0.50	24
Shale	VI	1.1	7.6	2.9	20.0	8200	2499	0.74	19

where E_1 = in-situ secant modulus of deformation,
E_2 = in-situ modulus of elasticity (third loading cycle),
V_1 = seismic velocity,
V_2 = saturated, laboratory sonic velocity, and
V_1/V_2 = fracture index.

Since this system was based upon a subjective evaluation of the geologic conditions of the Latiyan area only, the application of this system is extremely limited.

ROCK QUALITY CORRELATION WITH HARDNESS AND FRACTURE FREQUENCY (Deere, 1964)

Rock hardness and fracture frequency as determined from rock core examination has been correlated with Deere's modified rock quality designation (Deere, 1964). The objective of the resulting rock classification was to generalize the average in-situ rock quality.

Rocks were rated utilizing the modified RQD as "Excellent", "Good", "Fair", and "Poor" in descending order of rock quality:

Rock	Description
Excellent	Hard or unweathered (fresh);
Good	High RQD
Fair	Soft or unweathered; Closely
Poor	jointed; Low RQD

Although the modified RQD is a quantitive measure of the rock mass, values were not specifically assigned to the various rock grades, nor was "hardness" specified numerically. The system was designed to make maximum use of drilling information (Merritt, 1968).

STRENGTH-CONTINUITY CLASSIFICATION (Coates, 1964)

Combining field and laboratory tests, Coates (1964) proposed a method of in-situ classification based on intact strength, deformation characteristics, and degree of mass continuity by assuming an intact sample of a highly deformable rock would behave mechanically in a similar manner as in the field, and a strong rock would be either strong or weak, depending upon geologic discontinuities. Coates' classification has been previously described (see APPENDIX C, INTACT AND IN-SITU ROCK CLASSIFICATIONS, Coates, 1964).

This in-situ classification of rock should be supplemented with additional information such as joint orientation, permeability, porosity, and the presence and degree of altered (weathered) zones (Merritt, 1968). The major contribution of Coates' classification system was that, for the first time, the continuity of a rock mass was delineated by measurement of joint spacings.

GEOLOGICAL AND WEATHERABILITY CLASSIFICATION (Knill and Jones, 1965)

Extending the earlier work of Lane (1964), Knill and Jones (1965) developed classification systems for use at the Rosieres, Sudan, and Latiyan, Iran, dam sites. These classifications were designed to categorize the complex geology of each region. Combining core logging, borehole analysis, seismic surveys, in-situ deformation tests, permeability tests, geologic mapping, and general rock behavior during construction, rock grades were established for the engineering assessment of rock quality in situ.

As reported by Knill and Jones, "the most important single event in the geologic history of the Rosieres area, from an engineering viewpoint, has been the period of weathering which affected the near-surface bedrock." The primary factor in the rock weathering process has been a chemical decay of mineral constitutents (biotite and feldspars). Particular attention was given to gneisses as they were the most severely weathered rock at the dam site:

Gneissic Rocks at Rosieres

Grade	Description	Recovery (percent)	Engineering Description
I	Fresh	> 90	Least amount of blasting powder needed
II	Slightly Weathered	> 70	Permeability 1 x 10^{-5} cm/sec
III b	Moderately Weathered	45 - 70	Small proportion of friable material
III a	Highly Weathered	15 - 45	Only part of rock disintegrates in water
IV	Completely Weathered	< 15	Very permeable; mechanically excavated; slopes disintegrate in wet conditions

Experience on the site demonstrated that boundaries between various grades of weathered rock were occasionally sharply defined, but otherwise weathered margins were developed to depths of over 2 or 3 meters. Differences in the effect of weathering on the gneisses, early granites, and late granites appeared to have resulted from textural differences between these rock types -- gneisses had smooth grain boundaries and no interlocking, early granites had a fabric intermediate between late granite and gneiss, and the late granite had sutured boundaries with considerable interlocking.

The complexity of rock types caused initiation of a relatively simple means of describing rock in the area: I -- fresh, II -- slightly weathered, III -- highly or moderately weathered, and IV -- completely weathered. These grades were all based upon similar engineering characteristics displayed by various rocks within each grade.

An essentially similar system was developed for the sandstone, quartzites, and shales at the Latiyan Dam site:

Devonian Rocks at Latiyan

Grade	Description	Geologic Characteristics	Engineering Characteristics
I	Sound	Massive; widely spaced joints	Requires blasting
II	Bedded	Some shale layers	Requires blasting; stable slopes of 70° up to 20 m
III	Thinly Bedded or Flaggy	Some shale layers	Requires blasting
IV	Blocky or Seamy	Frequent intercalations of shale and clay-silt; some open joints	Requires blasting; stable slopes of 45 - 50° up to 15 - 20 m high
V	Broken	Faulted or weathered; some shale, generally found in a loose condition	Same as IV
VI	Thinly Bedded	Thin clay-shale seams	Requires blasting; slopes fail at angles of 40° over 20 m high
VII	Friable	Clay-shales	Mechanical excavation; slopes similar to VI

This classification was based on an assessment of various geological characteristics which control engineering behavior of the rock.

DEFORMATION AND SHEAR STRENGTH CLASSIFICATION (Deere, Hendron, Patton, and Cording, 1966)

One of the major considerations in the design of a structure in rock is the determination of the engineering properites of rock material at a particular site. A quantitative index of rock mass quality must be determined during preliminary stages of site selection and(or) initial stages of a particular design project. This time limitation implies a quantitative system should be based upon seismic surveys or borehole analysis and core logging, or some combination of these techniques in conjunction with tests on intact specimens.

With respect to tests, Deere et al. (1966) have proposed that deformation modulus and shearing resistance are the most significant parameters upon which to base correlations of rock types in preliminary design situations. The authors of this system proposed that:

a) field jacking tests or pressure chamber tests be conducted to determine the deformation modulus of the rock mass,

b) core borings be made to obtain samples for laboratory determinations of static modulus and sonic pulse velocity,

c) field seismic velocities be determined to obtain the velocity ratio (field seismic velocity divided by laboratory seismic velocity),

d) quantitive assessment of rock quality be made below the area to be loaded in the field test, and

e) a series of shearing resistance tests be performed on laboratory samples and in the field.

The use of core borings to relate modified RQD and fracture frequency with a description of rock quality is, in part, a quantitative approach to Deere's (1964) earlier description of rock quality:

Rock Description	RQD (percent)	Fracture Frequency (fractures/ft)
Excellent	> 90	< 1
Good	75 - 90	1
Fair	50 - 75	1 - 2
Poor	25 - 50	2 - 4
Very Poor	< 25	> 4

The in-situ deformation modulus can be estimated using this modified RQD system assuming that, if the joint spacing is wide enough, the deformation modulus of a rock mass will approach the value obtained from a laboratory specimen. Either the modified RQD or velocity ratio may be used as indices to determine rock quality; there exists a direct correlation between these two indices. Relating either of these indices to results of field jacking tests or seismic tests allows prediction of the in-situ deformation modulus.

CORE INDEXING CLASSIFICATION (Ege, 1967)

To correlate core borehole data and geophysical test data with the engineering behavior of rock, Ege (1967) devised a relative core indexing system. A core index number was obtained based on 10-ft (30-m) intervals of core. The quantitative core index was obtained by adding the joint frequency and the product of 0.1 and the percent values for core loss and broken core (pieces less than 3 in. (7.6 cm)). The multiplication factor of 0.1 was used to limit the core index number to values between 1 and 10.

There is approximately a one-to-one inverse relationship between the modified RQD and the core index (correlation coefficient by least squares method equal -0.989). Since both indices measure essentially the same properties of the rock core, a high correlation was expected. The modified RQD is easier to obtain in the field and would be preferred for field operations.

STRUCTURAL CLASSIFICATION (Obert and Duvall, 1967)

Obert and Duvall (1967) proposed a structural classification of in-situ rock combining both geologic and mechanical properties (see APPENDIX C, STRUCTURAL CLASSIFICATION, Obert and Duvall, 1967). The classification system was directed towards engineering properties which influence the design and construction of underground openings. Rock was categorized as either competent or incompetent, based upon the mass joint frequency and degree of weathering.

OBSERVED GEOLOGIC CHARACTERISTICS (Scott and Carroll, 1967)

Scott and Carroll (1967) proposed a system or technique whereby predictions of economic and engineering parameters could be made to guide tunnel construction contractor bidding. The authors classified the Precambriam granite with inclusions of Precambriam metasedimentary rocks at the Straight Creek Tunnel site in Colorado using seismic and electrical resistivity surveys in a pilot tunnel and in boreholes. The bedrocks were found to be extensively faulted and sheared and locally altered (weathered).

An arbitrary numerical rating scale of 1 through 5 was established by Scott and Carrol (1967) in which 1 represented the "best" rock and 5 the "poorest" rock. Quantitative as well as qualitative criteria were used to further specify rock quality:

Rock Quality	Quantitative Criteria		Qualitative Criteria		
	Fracture Spacing (fractures/ft)	Mineral Alteration (percent of rock)	Faulting	Foliation and Schistosity	Rock Type
1	> 3	< 5	None	None	Granite or diorite dikes; sparse migmatite
2	1 - 3	5 - 10	Minor; some slicks; minor gouge	Poorly defined	Granite, sparse gneiss and migmatite
3	0.3 - 1	10 - 15	Moderate; slicks common; minor gouge	Poorly to well defined; may be absent in granite	Granite and metamorphics; occurrences about equal
4	0.1 - 0.3	15 - 20	Moderate to severe; slicks and gouge on most surfaces	Well defined in metamorphics	Schist, gneiss, or migmatite; sparse granite
5	< 0.1	> 20	Intense	Very well defined	Schist; sparse granite

Comparisions of geophysical data and rock quality at various locations suggested that, as the rock quality improved, seismic velocity and electrical resistivity both tended to increase. Velocity and resistivity values were correlated statistically with the following economic and engineering parameters: height of tension arch, vertical load, type of steel support required, set spacing, percentage lagging and blocking, rock quality, the time rate of construction, and the cost of construction per foot. These relationships, established in exploratory or preliminary stages of construction, can be used as the project progresses to predict potentially hazardous rock conditions. Additionally, predictions of economic and engineering parameters could be established to guide construction in new tunnels while the methodology could be used for describing new sites.

MODIFIED RQD AND VELOCITY CLASSIFICATION (Merritt, 1968)

Utilizing information from various sites with various rock types present, Merritt (1968) determined that Deere's original modified RQD (Deere, 1963) best correlated with results of other types of borehole tests. In this determination, Merritt used core base lengths of 0.10, 0.20, 0.35, 0.50, and 1.00 ft (0.03, 0.06, 0.11, 0.15, and 0.30 m) in which these intervals of broken rock were disregarded in the modified RQD calculations.

After extensive testing and correlating procedures, Merritt presented a system in which in-situ rock quality could be determined either by seismic measurements or core logging. An estimate of the relative percentages of each category can be made for a general evaluation of the rock:

Rock Description	Modified RQD (percent)	Velocity Index
Excellent	> 90	> 0.8
Good	75 - 90	0.6 - 0.8
Fair	50 - 75	0.4 - 0.6
Poor	25 - 50	0.2 - 0.4
Very Poor	< 25	< 0.2

APPENDIX E

CORRELATION PARAMETERS

CORRELATION PARAMETERS

ANISOTROPY

The very nature of rock material as a matrix of composite elements and minerals promulgates a directional variation of mechanical properites; this is termed "anisotrophy". The rock anisotropy is a function of original sedimentation and subsequent compaction and cementation processes. Dominant directional dependency causes are (Somerton et al., 1970):

Scale of Effects	Causes	Functions
Mineralogical	Crystall Arrangement	Mineral Particle Shape
Petrological	Texture Character	Mineral Orientation
Macrostructural	Quasi-anisotropy	Lamination of Isotropic Media

McWilliams (1966) noted that preferred orientation of defect structures (grain boundaries, cleavage planes, twinning planes, inclusion trains, and pre-existing microcracks) contribute to rock anisotropic effects. The term "quasi-anisotropy" was coined by Silaeva and Bayuk (1967) to describe the apparent anisotropy caused by lamination of isotropic media found in bedded or stratified sedimentary rocks.

Imposed stress may also produce anisotropy. Somerton et al. (1970) state that "anisotropy in deeply buried sedimentary rocks may be the result of plastic flow" and the directional character of subsurface stresses "can result in the development of fracture patterns in preferred orientations".

Porous, permeable sedimentary rocks are composed of pore channels as well as a matrix structure. These pore channels functionally affect sedimentary anisotropy (Somerton et al., 1970):

Structure	Affected Physical Properties
Pore Channels	Fluid Permeability
	Electrical Conductivity (except when conductive solids are present)
Solids Matrix	Shear Wave Velocity
	Thermal Conductivity
	Dry Dilatational Wave Velocity
	Elastic Moduli

Since most sedimentary and metamorphic rocks have preferred orientations, the effect of anisotropy on rock strength is of extreme importance (Jaeger, 1972). Quantitative measurements of directional dependency may be defined by the ratio of the specific material property values measured in each of two mutually perpendicular directions to the value measured in a reference direction. For example, in bedded sedimentary rocks, the anisotropy can be expressed as the value of a property measured perpendicular to the bedding plane divided by the value measured parallel to the bedding plane (taken as the reference plane). For unbedded rocks, values are generally measured in two mutually perpendicular but arbitrary directions. Silaeva and Bayuk (1967) introduced the anisotropy coefficient, A, defined as the percentage difference in the material property being measured relative to a specified direction:

$$A_{21} = 100\,(V_2 - V_1)/V_1 \qquad V_2 \text{ with respect to } V_1$$

$$A_{23} = 100\,(V_2 - V_3)/V_3 \qquad V_2 \text{ with respect to } V_3$$

$$A_{13} = 100\,(V_1 - V_3)/V_3 \qquad V_1 \text{ with respect to } V_3$$

where V = measured material property,
(3) = perpendicular to bedding plane, and

(1),(2) = mutually perpendicular within the bedding plane.

Anisotropy can be subdivided into primary and secondary categories. Rock material develops anisotropy as anisotropic environments dominate in nature. This anisotropy begins with each mineral member in the aggregate matrix; i.e., orientation with respect to adjacent grains, cementing substance, and the resulting general vertical and(or) horizontal forces applied within this environmental condition (Jovanovic, 1970):

Anisotropy	Scale
Primary	Mineral aggregate (orientation)
Secondary	Anthropogenetic origin, fissure system, and stratification

Primary and secondary anisotropy have been adopted in practice (Watznauer, 1966). Discontinuity, effecting rock deformability, is then within the category of secondary anisotropy and may be considered anisotropy of stress states. Secondary anisotropy is an indicator of design investigations required since it is related to ultimate stress and the modulus of elasticity (Peres Rodrigues, 1970; cf. Peres Rodrigues, 1966).

DENSITY

Density is defined as the "weight of solid mineral matter per unit volume" (Duncan, 1969a). As Duncan points out, it is essential to differentiate between certain densities when dealing with rock materials:

a) solid mineral grain density, γ_s -- weight of solid mineral aggregate per unit volume of solids,
b) dry density, γ_d, -- weight of dry mineral aggregate per unit of total volume (volume of solids and volume of voids),
c) saturated density, γ_{sat} -- weight of mineral aggregate and water (voids saturated) per unit of total volume (volume of solids and volume of voids), and
d) bulk density, γ -- weight of mineral aggregate and water (voids partially filled with water) per unit of total volume (volume of solids and volume of voids).

Variations in density may well be expected among individual rock specimens from a particular site (location). Since a rock specimen of high grain density may possess almost any variation of pores (voids), there is no direct relationship between porosity and solid mineral grain density; e.g.

dry density = f (porosity and solid mineral grain density)

and

saturated density = f (porosity, solid mineral grain density, and saturation altering the void volume).

The density parameter does not correlate directly with a specimen's strength since it does not imply anything about the nature of the bonding between mineral grains (Duncan, 1969a).

DILATATIONAL WAVE VELOCITY

Rinehart, Fortin, and Burgin (1961) reported that the propagation veolicty of dilatation stress waves in rock material is a function of

a) initial state of stress,
b) stress level of the propagation wave,
c) moisture (water) content,
d) porosity,
e) texture, and
f) propagation direction with respect to rock stratification.

Initial State of Stress

In general, the propagation velocity, V_p, increases with a corresponding increase in pressure (Tocher, 1957; Wyllie, Gregory and Gardner, 1956). Rinehart et al. (1961) observed that rocks which exhibited well-defined textural properties necessarily possessed a relatively narrow range of propagation velocities. They also noted that the same type of rock from different origins may have propagation velocities over a six-fold range. As would be expected, rocks which are relatively dense and compact possess higher velocities than less dense rock.

Data presented by Miller and Deere (1966) showed that most rocks exhibited an increase in propagation velocities with an increase in applied stress. The range of wave velocity increase was enormous, being 1/4 percent for Solenhofen Limestone to 132 percent for Luther Falls Schist. In general, it would probably be correct to note that applying stress to confine a less competent rock has a tendency to increase its competency; confining a competent rock would have only relatively minimal effects towards attaining greater competency. "The increase in V_p with increasing axial stress is concluded as being the result of the closing of micro-cracks" (Miller and Deere, 1966). These authors also report "the average coefficients of variation for the dilatational wave velocity measurements are smaller than for any other property except unit weight. At a stress level of 100 - 150 psi, V is 2.7 percent, and at 5,000 psi, V is 1.9 percent; the relative order being as expected". Miller and Deere's empirical results demonstrate that "in general, the velocities measured during unloading are higher than those measured during loading. Just as for the modulus of deformation. E, the reason for this hysteresis effect is probably due to friction between crack surfaces, which prevents sliding in the opposite sense (hence, crack openings) immediately after the load is reduced".

Assuming that the static and dynamic properites of rock are interchangeable, one may arrive at the following equation (Miller and Deere, 1966):

$$V_p = \sqrt{E\ (1 - \nu)/\rho(1 + \nu)\ (1 - 2\nu)}$$

where
V_p = dilatational wave velocity in an unbounded medium,
E = Young's modulus,
ν = Poisson's ratio, and
ρ = $\gamma/g \approx$ mass density.

It should be remembered that the interchangeability of static and dynamic properties is pertinant in two instances:

a) compact rocks at low stress levels (100 psi or 0.7 MPa), and
b) less compact rocks at increased stress levels (5,000 psi or 34 MPa).

Moisture (Water) Content

As the moisture content increases to a saturation condition in layered sedimentary or metamorphic rock, the dilatational wave velocity may increase some 10 to 15 percent (Somerton et al., 1970). The presence of moisture in pore spaces reduces both the apparent heterogeneity and anisotropy. Non-porous rocks naturally would not be affected appreciably by moisture conditions.

Porosity(Absorption)

The relative heterogeneity (discontinuous nature) increases as porosity increases; this has the impact of decreasing the wave velocities. This increasing heterogeneity interrupts the flow of dilatational wave patterns.

Temperature

An increase in temperature is generally accompanied by a decrease in wave propagation velocity of from one to five percent per 100 C at standard pressure (Rinehart et al., 1961). This decrease is attributed to varying internal strains (expansions) of the rock constitutents. These strains may cause permanent internal microfissures which destroy some of the continuity of the rock matrix structure.

Stratification

The propagation wave velocity measured parallel to rock layering (stratification) is usually greater than the velocity measured perpendicular to stratification.

DRILLABILITY

"Drillability" and "hardness" are often used interchangeably, and in fact, these terms are often applied indiscriminately to describe the resistance of rock to penetration by any type of drilling tool (cf. Mather, 1951). At present, there has been a tendency in the drilling industry to assign to "hardness" a variety of meanings which relate to the type of drilling method employed:

Hardness	Drilling Method
Resistance to abrasion	Diamond drilling
Resistance to impact (indentation)	Percussion drilling
Analogous to compressive test	Rotary drilling

"Hard rock" generally describes a rock mass (geologic formation) which is difficult to drill. But, as Shepherd (1950) describes, many "hard rocks" have been drilled more easily than softer rocks since hard rocks are more brittle; increased rock chipping occurs which effectively increases the drillability -- a hard igneous or metamorphic rock may be drilled with greater efficiency than a compact limestone.

It has long been felt by investigators that hardness is closely related to rock drillability. Protodyakonov (1963) reported that the mechanical properties of rocks with respect to drilling may be characterized by hardness, deformability, and abrasivity. Additionally, he noted that drilling efficiency not only depends upon the mechanical rock properties but also upon the drilling machinery and the efficiency of the drillers themselves. Research by Shepherd (1950, 1951), in correlating the resistance to penetration with the physical properties of rock material, demonstrated that, with respect to hardness, information obtained from Shore scleroscope readings must be carefully analyzed to obtain reliable drillability indications.

Since Scott's (1946) experiments utilizing a micro-bit drill to correlate rock drillability with the rock crushing strength (compression strength), many investigators have reported micro-bit drillability correlations and even rock classifications based on micro-bit tests (Head, 1951):

a) Scott (1946) -- first micro-bit correlation of drillability and crushing strength,
b) Head (1951) -- micro-bit drillability classification for 15 geologic formations, and
c) Rollow (1963) -- discussion concerning drilling efficiency and drillability by correlating drilling rate and teeth wear to ease of drilling.

It should be realized that micro-bit measures relate to the three major types of hardness -- abrasion, indentation, and rebound.

Finally, with respect to hardness and drillability, Head (1951), utilizing a Knoop testing device, reported after testing seven different rock types there was no consistent empirical relationship between hardness and drillability. Drillability appears to depend on the rock crystal constituent bonding.

DRY APPARENT SPECIFIC GRAVITY

The dry apparent specific gravity, G_b, of a rock material is defined as (Duncan, 1969a; Cottiss et al., 1971)

$$G_b = W_s/V\gamma_w$$

where W_s = oven-dryed weight of a given volume of rock sample,
V = volume of rock sample, and
γ_w = density of water.

G_b defines the weight of solid mineral grains per total volume of solids and voids, with the voids empty. For testing purposes, the rock specimen should be waxed after drying, since a good measure of V can be obtained by determining the water displaced by the sample. A rock of low porosity would not require waxing.

Ranges of G_b reflect general rock texture characteristics (Duncan, 1969a):

Dry Apparent Specific Gravity	Texture
$G_b < 2.0$	Hard soils
$2.0 \leqslant G_b \leqslant 2.5$	Cemented and compacted rock
$G_b > 2.5$	Crystalline, crystalline-indurated, and indurated rock

This apparent relationship between the field description of a rock sample and the dry apparent specific gravity as an index property is included within Duncan's rock classification system (see APPENDIX C, Duncan, 1969).

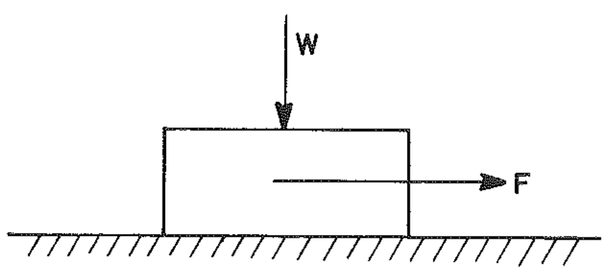
FRICTION

The frictional properties of rock minerals and(or) rock constituents are extremely important in engineering assessments of rock behavior (Duncan, 1969a). Frictional effects are apparent on all scales (Jaeger and Cook, 1969):

a) microscopic scale, with respect to friction on boundaries of Griffith cracks;
b) megascopic scale, at which friction occurs between individual rock grains and(or) pieces of aggregate; and
c) macroscopic scale, on friction surfaces of joints or faults.

Friction is a retarding function relating normal force on a contact plane to the shear force necessary to initiate sliding (motion) on that plane:

Amonton's Law: $F = \mu W$



where F = shearing force parallel to contact surfaces,
W = force normal to plane of contact, and
μ = coefficient of friction.

The coefficient of friction, μ, is a function of the nature of the contact materials and the finish and state of the contact surfaces. Jaeger and Cook (1969) state that, based on empirical evidence, μ is not a function of the normal force nor the area of contact.

Amonton's Law may be interpreted in terms of stress by dividing the above equation by the contact area:

$$F/A = \mu W/A$$

or

$$\tau = \mu\sigma$$

where τ = shear stress across surfaces necessary to initiate sliding and

σ = normal stress across the surfaces in contact.

With regard to rock behavior, μ, the coefficient of friction, is not invariant as with metals but varies with σ; μ is usually greater for small values of σ (cf. Bowden and Tabor, 1950; cf. Maurer, 1965). At low stress levels, Jaeger (1959) defined a linear law for frictional rock behavior:

$$\tau = s_o + \mu\sigma$$

where s_o = inherent shear strength of the contact surface (similar to cohesion, c, of soil mechanics) (Jaeger and Cook, 1969).

Typical Values of μ and s_o

		s_o	
Rock	μ	(psi)	(kPa)
Sandstone	0.51	40	276
Granite	0.64	45	310
Gabbro	0.66	55	379
Trachyte	0.68	60	414
Marble	0.75	160	1103

HARDNESS

Hardness is a general term used to describe several different characteristics of materials with respect to their resistance to abrasion, indentation, cutting, wear, or rebound. It is a physical-mechanical property primarily associated with material surface character. The term "hardness" is vague; this ambiguity is compounded by the use of five basic hardness measurements:

a) abrasion(scratch) hardness,
b) indentation hardness,
c) magnetic hardness,
d) portable hardness, and
e) rebound(dynamic) hardness.

Hardness tests are very simple mechanical tests. A hardness number expresses the resistance or toughness of a rock specimen by describing the elastic deformation of the specimen under the influence of an imposed force. Richards (1961) defined technological hardness as the magnitude of resistance of a material against a permanent set (deformation) of its surface. In general, hardness is a function of rock strength, toughness, resilience, elasticity, bonding, and cementation. But "a numerical value of hardness is as much a function of the kind of test used as it is a material property" (Miller and Deere, 1966).

From early studies in metallurgy, an apparent correlation between hardness and other mechanical bulk properites was recognized. Hardness in one form or another is almost universally used as one aspect of rock classification systems. This use has not been totally successful. "The repeatability of the hardness measurement depends mainly on the homogeneity of the sample to be tested ... Depositional and(or) various diagenetic changes within a rock substance can also lead to variations in hardness" (van der Vlis, 1970).

Measurements of hardness generally fall into one of three categories -- abrasion(scratch) hardness, indentation hardness, and rebound(dynamic) hardness:

1. Abrasion Hardness

a. Mohs Mineral Hardness Scale

Hardness is an important aid in mineral identification, reflecting to some extent the physical and mechanical mineral properties. Hardness of a mineral may be measured by its resistance to abrasion. A series of ten common homogenous minerals have been chosen as the basis for a scale of comparative hardness; i.e., susceptibility to scratching. Mohs scale (1824) (after Zwikker, 1954) follows:

Mohs Hardness Number	Mineral	Scratch Susceptibility	Formula
1	Talc		$3MgO \cdot 4SiO_2 \cdot H_2O$
2	Gypsum		$CaSO_4 \cdot 2H_2O$
		Fingernail	
3	Calcite (Mica)		$CaCO_3$
		Penny	
4	Fluorite		CaF_2
5	Apatite		$CaF_2 \cdot 3Ca_3P_2O_8$
		Knife Blade, Glass	
6	Feldspar		$K_2O \cdot Al_2O_3 \cdot 6SiO_2$
7	Quartz	Steel File	SiO_2
8	Topaz		$(AlF)_2 \cdot SiO_4$
9	Corundum		Al_2O_3
10	Diamond		C

Mohs scale of hardness has been accepted universally as a method of measuring mineral hardness and has lately become an important aid in correlating rock hardness with rock strength (de Beer, 1968). In this relative scale, each mineral can be scratched by those that follow it, and each in turn will scratch the preceding ones in the scale. It must be understood that mineral hardness is a function of cohesion, brittleness, compressive strength, tensile strength, yield point, and the elastic limit of the composite minerals. The Mohs scale is most successful with fine-grained monomineralic rocks but has also been used with polymineralic rocks.

In 1954, Tabor demonstrated that the relative Mohs scale gives scratch hardness values which correspond well with indentation hardness values -- each mineral increment except diamond on the Mohs scale corresponds to a 60 percent increase of indentation hardness (Miller and Deere, 1966).

b. Mineral Composition

Since rock may consist of several mineral aggregates and is thus heterogenous, abrasion tests based on the Mohs scale are inadequate. Realizing this, Shepherd (1950) published a method for measuring the composite mineral hardness number based on the relative percentages of different minerals within a rock matrix. This hardness is defined by

$$H = \Sigma(SM/100)$$

where H = rock hardness,
S = decimal equivalent of percentage of mineral present, and

M = Mohs mineral hardness number.

Inherent in this method is the problem of obtaining a good estimation of rock mineral content, and this concept ignores completely the strength of mineral bonds (layer, ionic, covalent) which determine aggregate hardness (Miller and Deere, 1966).

c. **Dorry Abrasion Test**

Jackson (1916) described in the U. S. Department of Agricultural Bulletin 347, **Methods for the Determination of Physical Properties of Road-Building Rock**, a test for determining the relative hardness of rocks. In this test, a cylindrical rock specimen of 25-mm diameter is supported against a revolving, cast-steel disk under a load of 2.45 kPa. The specimen is abraded in inverse ratio to its hardness; therefore, the loss of sample weight is an index of hardness. For rock comparison purposes, the rock hardness coefficient is empirically obtained by arbitrarily subtracting one-third of the specimen's weight loss from the number 20 (Miller and Deere, 1966).

d. **Deval Abrasion Test**

This standard method of testing for rock abrasion hardness was approved by ASTM (D2-33, reapproved 1968) but was withdrawn in November 1972. Some 50 rock specimens of approximately uniform size having a total weight of 5 kg ± 10 g after washing and drying are placed in a Deval Machine, a revolving hollow cylinder of 200-mm diameter and 340-mm depth mounted on a shaft at an angle of 30 degrees to the axis of rotation of the shaft. After the rock specimens have been tumbled for 10,000 revolutions at a speed of 30-33 revolutions per minute and sieved on a No. 12 (1.70-mm) sieve, the relative weight of material passing the sieve is an index of hardness. The material passing the sieve expressed as a percentage of the orginal weight of the test sample, or the French coefficient of wear, may be used as the index:

$$\text{French coefficient of wear} = 40/W$$

where W = weight of material passing the No. 12 sieve expressed as a percentage of total weight.

e. **Burbank Abrasion**

In 1955, Burbank published an article entitled *Measuring the Relative Abrasiveness of Rocks, Minerals and Ores* in **Pit and Quarry** magazine. He proposed a system whereby a steel paddle impactor revolving on a shaft strikes a column of rock particles falling away from an outer, more slowly revolving drum. The loss in weight of the paddle is an index of relative rock abrasion.

2. *Indentation Hardness*

Indentation hardness is the most widely used test for relative hardness measurement. The test procedures, involving different testing machines, basically utilize one principle; a penetrator is subjected to a specific load and itself impresses a rock specimen causing a set or permanent deformation in the sample. The dimensions of the impression caused by the penetrator are an index of the relative hardness of the rock material. Essentially, the only difference among indentation tests is the specific shape of the penetrator (i.e. spherical, conical, or pyramidal):

Penetrator Shape	Hardness Test	ASTM Designation
Spherical	Brinell	E 10-66
	Rockwell	E 18-67
Conical	Rockwell	E 18-67
Pyramidal	Vickers	E 92-72
	Knoop	C 730-72T

For relative hardness in rock mechanics, hardness tests from metallurgical applications have generally been used. These include:

a. **Brinell Hardness, ASTM E 10-66 (reapproved 1972)**

Calibrated machine forces a steel ball, under specified conditions, into the surface of the tested material. After removal of the load, the diameter of the resulting impression is measured. Brinell Hardness is defined by

$$HBN = 2L/(\pi D(D - \sqrt{D^2 - d^2}))$$

where HBN = Brinell Hardness Number,
L = applied load, kg,
D = ball diameter, mm, and
d = mean diameter of impression, mm.

It has been found empirically that the Brinell Hardness Number is a function of the size (diameter) of the steel indenting ball, magnitude of indenting load, and the elastic characteristics of the indenting ball (ASTM E 10-66; van der Vlis, 1970). Many techniques involving different loads and load durations are used:

Device Designation	Ball Diameter	Load	Load Duration
HB	10 mm	29.4 kN	10-15 seconds
53 HB 10/500/30	10 mm	4.9 kN	30 seconds

Since hardness is not a unique rock material characteristic but a function of degree of heterogeneity and indenter ball size, some investigators, among them, Huitt and McGlothlin (1958), have proposed that Meyer's relation (1908) be used for the evaluation of embedment properties of rock (van der Vlis, 1970):

$$d/D = B^{1/2} (L/D^2)^{m/2}$$

where d = diameter of indentation circle,
D = ball diameter,
B = material hardness constant,
L = load, and
m = material correlation constant.

Variation of m was found to be less that 10 percent for tested rock specimens (Solenhofen Limestone, Oberkirchen Sandstone, Udelfang Sandstone, and Marl) and therefore may be neglected "and the data for one rock type can be reasonably well described by a single straight line" (van der Vlis, 1970). Although the specific rock hardness may be characterized by B, both B and m are exceedingly tedious to evaluate under laboratory conditions.

Additional complications in the Brinell Hardness test may be due to a phenomenon known as "piling up", wherein the indentation ball causes an upward extrusion of rock material forming a raised crater. In very soft materials, the indentation ball may sink to a depth greater than the radius of the ball and give ambiguous results.

It has become common practice to assume that the volume of penetration is hemispherical and calculate the Brinell Hardness as

$$BHN = L/Dh$$

where L = load (kg),
D = ball diameter (mm), and
h = depth of penetration (mm).

Empirical results have shown "that for a certain ball size, the ratio L/h . . . is a constant, and the hardness of rock thus measured can be expressed as a single number." This is the basis of van der Vlis' rock classification system (see APPENDIX C, INTACT ROCK CLASSIFICATION SYSTEMS, van der Vlis, 1970). Van der Vlis' preliminary study demonstrated that a useful classification for rock could be established utilizing the Brinell Hardness Number.

Van der Vlis further investigated the influence of the voids content on the rock hardness. The penetration of a variety of ball sizes into rock specimens saturated with and immersed in brine (10 percent NaCl solution) was measured. Results showed that the liquid filling of pores in rock specimens reduced the relative hardness.

b. **Rockwell Hardness, ASTM E 18-67**

The Rockwell Hardness is based upon the depth of penetration caused by an increment of load rather than upon the diameter of the indentation as is the Brinell Hardness. Initially, a "minor" load is applied to the sphero-conical or spherical penetrator and the test gauge is zeroed at this loaded indentation. Then, the "major" load is applied and the depth of penetration is based on the increment of penetration resulting from the increment of load. The Rockwell Hardness Number, HR, is derived from this net increase in the depth of penetration as the load on the penetrator is increased from a minor load to a major load and then returned to the minor load.

c. **Vickers Hardness, ASTM E 92-72**

The Vickers test was introduced by Smith and Sandland in 1925 and is presently defined as using "calibrated machines to force a square-based pyramidal diamond indenter having specified face angles, under a predetermined load, into the surface of the material under test and to measure the diagonals of the resulting impression after removal of the load" (ASTM E 92-72). The Vickers Hardness Number, HV, is equal to the applied load divided by surface area of the permanent deformation made by the indenter having included face angles of 136 degrees:

$$HV = 2L \sin(\alpha/2)/d^2 = 0.8544\ L/d^2$$

where L = applied load (kg),
d = mean diagonal of impression (mm),
α = 136 degrees = face angle of indenter.

As in the case of Brinell "piling up" and "sinking-in", similar malfunctions may occur in the Vickers Pyramid Test. Piling-up corresponds to "convexity" and sinking-in corresponds to "concavity". In convexity, the measured diagonal values of the area decrease and thereby cause erroneously high hardness values; similarly, concavity causes erroneously low hardness numbers. Correction of this error may be accomplished by correction of the measured surface area or impression.

Many investigators have utilized these metallurgical tests in rock and mineral measurements. Knoop, Peters, and Emerson in 1939 reported the Knoop Hardness Numbers for the standard minerals used by Mohs (1824) in the Mohs scale. Brace reported results of using the Vickers Test with varying loads on four monomineralic isotropic samples (1960). Kraatz (1964) performed Rockwell Hardness Tests on 24 rock samples described in Miller and Deere's report (1966) on an engineering classification of rock. Duncan (1969a) reported on Young and Millman's extensive investigation utilizing both Vickers and Knoop indenters on Mohs indicator minerals. Their results, in which log M versus log HV was plotted, demonstrated an almost linear relationship.

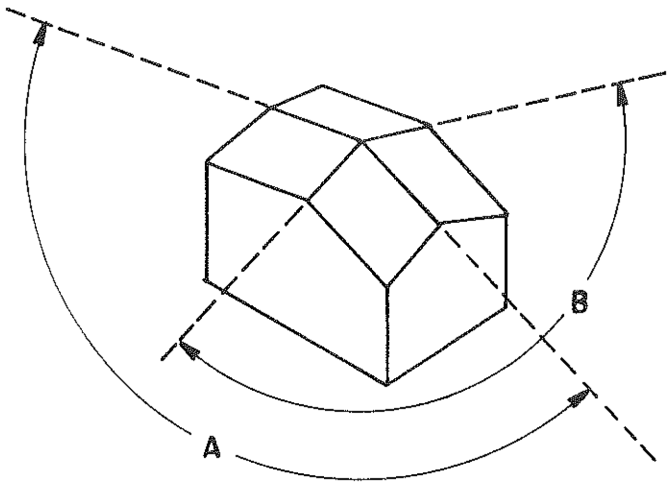
d. **Knoop Hardness, ASTM C 730-72T**

This test is "an indentation hardness test using a calibrated machine to force a pointed, rhombic-base, pyramidal diamond indenter having specified face angles, under a predetermined load, into the surface of the material under test and to measure the long diagonal of the resulting impression after removal of the load" (ASTM C 730-72T). As with other indentation hardness numbers, the Knoop Hardness Number is obtained by dividing the applied load upon the test surface by the projected area of indentation:

$$KHN = (L/A_p) = (L/d^2C_p)$$

where KHN = Knoop Hardness Number,
L = applied load (kg-force),
A_p = projected area of indentation (mm^2),
d = length of the long diagonal of the indentation (mm),
C_p = (cot A/2 x tan B/2)/2g,
A = included longitudinal edge angle, and
B = included transverse edge angle.

For a perfect indenter, A = 172° 30' 00", B = 130° 00' 00", and C_p = 0.07028.



The required number of indentations is a function of the particular specimen. In general, it is considered adequate to perform at least ten indentations reporting m (number of indentations), KHN, and the standard deviation:

$$S = \sqrt{1/(m - 1) \Sigma (\overline{KHN} - KHN_m)^2}$$

where S = standard deviation of a single observation,
$\underline{m}$ = number of indentations,
$\overline{KHN}$ = mean KHN, and
KHN_m = KHN obtained from the m^{th} indentation.

3. *Rebound(Dynamic) Hardness*

"Dynamic hardness of a material may be defined, by analogy with static hardness, as the resistance to local indentation when the indentation is produced by a rapidly moving indenter" (Miller and Deere, 1966). In general, an indenter is allowed to rebound off a tested material; the rebound height is an indication of material relative hardness. Most often, the indenter falls under a gravity force and indents the surface of the testing material.

There exist many types of dynamic hardness testers. Among these are the Wust and Bardenbeuer apparatus which have a ball indenter, with a magnetic release which allows a ball to drop a specified vertical distance, and the Izod Impact Machine which utilizes a special anvil and pendulum arrangement. The most widely used dynamic testing machines in terms of rock mechanics have been the Shore Scleroscope and the Schmidt Hammer.

a. **Shore Scleroscope**

The Shore Scleroscope is a relative hardness tester which is portable, easy to operate, and nondestructive. This rebound device allows for rapid and inexpensive tests. A small diamond-pointed hammer is allowed to drop a fixed vertical distance onto a testing surface. The rebound of the hammer is a relative measure of the elastic property of the tested material (Snowden, 1948; Miller and Deere, 1966). Kapadia (1951) demonstrated empirical relationships between scleroscope relative hardness and elastic and strength characteristics of a few rock types. Gilbert (1954), concerning himself with mineral hardness, determined the scleroscope hardness of Mohs minerals and found an approximate linear relationship between the two categories (Miller and Deere, 1966).

Many investigators have reported relatively successful attempts to determine and correlate scleroscope hardness and rock properties:

1) Grenves (1909) -- reported that the scleroscope does measure the relative hardness of a material,
2) Griffith (1937) -- obtained relative scleroscope hardness for typical rocks,
3) Obert, Windes, and Duvall (1946) -- obtained scleroscope hardness for mine rock,
4) Wolansky (1949) -- obtained Shore scleroscope hardness values and correlated them with rock

drillability and workability,

5) Shepherd (1950) -- used Shore scleroscope to study rock hardness and rock drillability,

6) Wuerker (1953) -- empirically described the relationship between rock compressive strength and scleroscope hardness from data encompassing more than 100 rock specimens, and

7) Miller and Deere (1966) -- demonstrated that the uniaxial compressive strength and modulus of elasticity correlate well with the product of Shore (or Schmidt) hardness and the dry unit weight of the rock.

b. Schmidt Hammer

The Schmidt hammer was designed by Schmidt to be used to estimate concrete strength in-place. In this sense, the hammer can be nondestructive and is extremely portable. Like the Shore scleroscope, the Schmidt hammer is a rebound device; a definite amount of stored energy is imparted, upon impact, to the testing surface. There are two basic sizes of hammer devices:

1) Type N hammer imparts 1.65 foot-pounds (2.24 J) of impact energy and

2) Type L hammer imparts 0.54 foot-pounds (0.73 J) of impact energy.

As pointed out by Miller and Deere (1966), the N-type hammer has a tendency to destroy all but the strongest rock specimens while the L-type hammer destroys the weakest specimens.

The hammer test is very simple and is a rapid method for determining relative rebound-hardness values for rock. Hucka (1965) and Miller and Deere (1966) report good reproducibility of test results, but, Cottiss et al. (1971) report that the Schmidt rebound hardness is "not noted for giving reproducible results ..., with care, the average value can give a useful indication of rock strength, particularily if used in conjunction with other types of tests."

There seems to be no perceptible difference in Schmidt relative hardness values from block specimens or NX-size cores -- assuming that the block specimens and(or) core remain intact (Miller and Deere, 1966). Knill and Jones (1965) utilized the Schmidt hammer to determine the rebound hardness of granite cores and Hucka (1965) suggested using the Schmidt hammer to determine the strength of in-situ rock.

Empirical results obtained by de Beer (1968) using a Schmidt hammer demonstrated the instrument could clearly distinguish between different categories of rock hardness. Rebound numbers of a particular instrument should be correlated with the uniaxial compressive strengths of the particular rocks; this correlation can form the basis of a classification of rock strength. De Beer used a hardness classification of rock obtained through a personal communication with Jennings and Klingman (1966) and classed a series of rocks relative to rebound numbers:

Hardness Classification	Rebound Number Range
Very Stiff Soil	16 - 20
Very Soft Rock	20 - 24
Soft Rock	24 - 30
Hard Rock	30 - 45
Very Hard Rock	45 - 60
Very, Very Hard Rock	> 60

Very stiff soil was undefined except by the rebound range,
Very soft rock crumbles under firm blows of the sharp end of a geology pick or is scratched by fingernail,
Hard rock cannot be scraped with a knife but can be broken with the hammer end of a geology pick with one firm blow,
Very hard rock can be broken with the hammer end of a pick under more than one blow, and
Very, very hard rock requires many blows with a geological pick to break through intact material.

This hardness classification was designed by Jennings and Klingman for the field assessment of a rock formation to obtain a preliminary concept of the hardness of a rock.

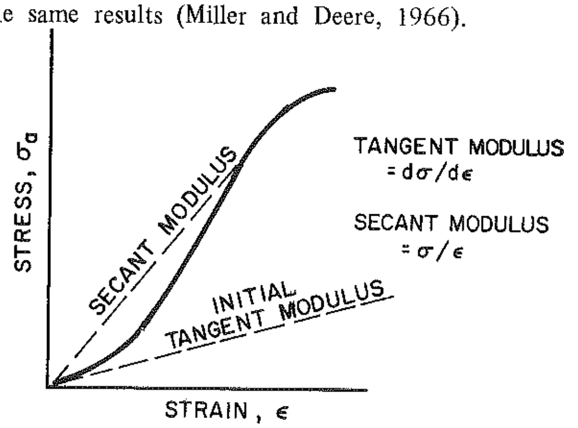
HYSTERESIS

According to Miller and Deere (1966), most rocks exhibit elastic hysteresis, defined as the plastic "loop" or difference exhibited in stress-strain curves when a material is loaded and unloaded with the material not sustaining a permanent deformation. This nonlinear elastic behavior of rocks under uniaxial compression is attributed to the presence of pores and(or) minute surface cracks, which are open at low stresses (Jaeger, 1962; Duncan, 1969a; cf. Ide, 1936; cf. Walsh, 1965). These cracks close as the stress level is raised, and subsequently, the rock becomes elastically stiffer. Above these relatively low stress levels which close specimen cracks, the rock material is characterized by linear stress-strain curves since there is no further change in the rock stiffness. Walsh (1965) explained that the modulus of elasticity, the stress-strain curve characteristic, varies more in rock than in an equivalent uncracked solid because the rock deforms from sliding of cracked surfaces even after crack closure during the loading cycle.

Hysteresis seems to be restricted to uniaxial compression -- it does not appear to occur during hydrostatic compression, according to Walsh (1965). The slope of the stress-strain curve during removal of uniaxial stresses is initially greater than the slope measured during loading for all values of stress -- probably because of friction. Microcracks which have been closed and undergone sliding during uniaxial compression do not immediately slide in response to load reduction; but instead, a residual strain is often observed (Walsh, 1965). Further loading-unloading cycles produce similar hysteresis loops, but they have a tendency to move slightly to the right on a stress-strain diagram because of transient creep with no permanent deformation or "anelasticity" (Jaeger, 1962).

MODULUS OF ELASTICITY

The "modulus of elasticity", "Young's modulus", "elastic modulus", "compression modulus", and "modulus of deformation" are terms used almost interchangeably to describe the linear segment of the stress-strain plot of a rock material. There are two basic definitions for the modulus of elasticity; the tangent modulus and the secant modulus. For unilinear stress-strain curves, both methods give exactly the same results (Miller and Deere, 1966).



TYPICAL CURVE FOR SANDSTONE
For low stress levels, the curve is normally highly nonlinear and concave upward. (After Krynine and Judd, 1957; Jaeger, 1962; Brace, 1963; Miller and Deere, 1966)

The modulus of elasticity (E or M) is the ratio of stress to strain for a material under given loading conditions and is numerically equal to the slope of the tangent or the secant to a stress-strain curve. It is recommended that the term "modulus of elasticity" be used for materials which deform according to Hooke's Law; i.e., exhibit a linear relationship between stress and strain (ASTM D 653-67). It is recommended that "deformation moduli" be used for materials that deform nonlinearly (ASTM D653-67).

Miller and Deere consider two moduli of deformation important for rock classification studies -- the initial tangent modulus, E_i, for the initial loading cycle and the tangent modulus at a stress level of 50 percent of the ultimate compressive strength at failure, E_{50} or E_{t50}. The initial tangent modulus is usually difficult to determine accurately because of the closure of microfissures present in rock, specimen seating problems, and other anomalous behavior arising from the specimen's stress history. Using the tangent modulus at 50 percent of the ultimate strength reduces the stress history effects and the initial effects of large strain at lower stress levels.

For the range of stresses developed in mine operations, many types of rock tested in uniaxial compression are relatively elastic; that is, the strain is recovered upon removal of the stress. However, a significant number of rock types are not linearly elastic; moreover, there are great variations in the elastic properites of the same rock type from different geographic locations because of differences in degree of microfracturing and stress histories (Obert and Duvall, 1967).

In general, most rock materials exhibit, to some degree, both elastic and plastic behavior. Consequently, many stress-strain diagrams are nonlinear and vary with specific ranges of applied stresses and rates of stress application. Additionally, creep occurs in many rock types. Duncan (1969a) suggests that the term "non-elastic modulus of deformation, N_m" be used to characterize a value for the nonlinearity of stress-strain relationships of most rocks. Essentially, the variability of the stress-strain curve is such that one should always specify the method used in obtaining the value for E and "commonly, the gradient of the tangent to the second or third loading or unloading cycles may be adopted" (Duncan, 1969a).

MOISTURE CONTENT

Duncan (1969a) points out that in a practical field investigation there are certain measurements of rock condition of special interest to engineers. Among these are

a) natural (in-situ) moisture content,
b) saturation moisture content, and
c) porosity.

For compacted shales, Duncan recommends that the investigator also determine "the extent to which changes in void volume may occur with changes in stress conditions in the ground and with variations in moisture availability." Some of the less competent rock types (shale, siltstone, and mudstone) are sometimes permanently affected by changes in moisture in terms of expansion or contraction of the rock and also by "general deterioration of surface or near-surface exposure" (Obert and Duvall, 1967). Another parameter, the shrinkage limit (defined as the moisture content at which further loss of moisture does not reduce the specimen volume) is important in terms of compacted specimens.

Colback and Wiid (1965) reported that moisture content has a major influence on compressive strength and elastic properties of at least some rocks. Specimens of quartzitic shale and quartzitic sandstone were found to lose up to 50 percent of their compressive strengths under saturated, submerged conditions as compared to dry specimen compressive strength.

Obert et al. (1946) found that oven-dried samples did not exhibit the same elastic characteristics as air-dried specimens. Air drying followed by oven drying produced results which are often irreversible:

a) dynamic moduli decreased (< 15 percent),
b) uniaxial compressive strengths increased (≈ 6 percent), and
c) Shore hardnesses increased (≈ 20 percent) or showed no change.

In addition, significant changes were recorded when specimens were tested under conditions from air-dried states through various moisture conditions including the saturated state (Miller and Deere, 1966; Obert and Duvall, 1967):

a) dynamic moduli increased (19 - 35 percent) for some rocks and decreased for others,
b) compressive strengths decreased (≈ 12 percent), and
c) Shore hardnesses decreased (≈ 10 percent).

Of course, 10 or 15 percent difference in the empirical values from air-dried and oven-dried specimens or air-dried and saturated specimens may not be particularly significant considering the heterogeneous nature of most rocks. Moisture content is not particulary important for competent rocks; but weak, highly reactive rocks are extremely susceptible to moisture. Miller and Deere recommended that rock specimens be subjected to 2 weeks of air drying prior to testing, as a result of Obert's observations.

The International Society for Rock Mechanics Commission on Standardization of Laboratory and Field Tests has suggested a method for the determination of the water content of a rock sample (1972):

Apparatus

a) oven capable of maintaining a temperature of 105 C ± 1.5 C for at least 24 hours,
b) non-corrodible sample container with an airtight lid,
c) dessicator, and
d) weighing balance with an accuracy of 0.01 percent of the sample weight.

Procedure

a) container and lid are cleaned, dried and weighed, to obtain weight "A";
b) a representative sample (at least 10 specimens of rock of at least 50 gm each) is selected;

c) each sample is placed in container with lid and the combination is weighed to obtain weight "B";
d) remove lid and dry sample to constant weight in oven at 105 C; and
e) replace lid and allow sample to cool in dessicator for 30 minutes and then weigh to obtain weight "C".

Calculation

Water content to nearest 0.1 percent = w = 100 (B - C)/(C - A).

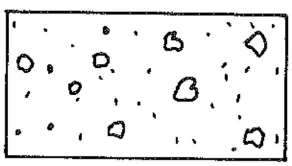
PERCOLATION

Percolation is the flow of liquid through a pervious media. Quantitatively, percolation occurs according to a regime described by Newtons' equilibrium equations and the continuity equation; it is measured as a function of potential gradients and time.

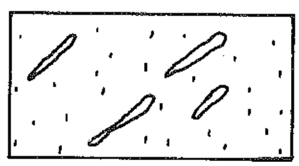
Terzaghi (1962b) characterized percolation in two ways: "primary percolation", a function of microfissures and some microfractures and macrofractures, and "secondary percolation", a function primarily of microfractures and macrofractures. Macrofractures are gross mass discontinuities and consequently can only be measured in situ.

Rock may be classed into two main categories according to the value of percolation coefficient in a direction parallel to the axis of the rock specimen (Jaeger, 1972). Habib and Vouille (1966) tested rocks, with the following results:

Category	Rock Type	Permeability (10 cm/s)	Shape of Discontinuities	Voids (percent)
1	Limestone	7.5	Spherical and ellipsoidal	15 - 25
	Hard Sandstone	240	Spherical and ellipsoidal	15 - 21
2	Quartz	0.15 - 1.3	Fissured	< 5
	Hard Schist (stratified)	1.9 - 12	Fissured	< 5



SPHERICAL VOIDS



FISSURED VOIDS

Test results demonstrated that when a rock is microfissured, with fissures much longer in one direction than in another direction, the permeability decreased with decreasing pressure gradient. When the permeability is independent of the pressure gradient, it may be assumed that the sample voids are more or less spherical or ellipsoidal in shape.

PERMEABILITY

ASTM D 653-67, Standard Definitions, defines permeability as "the rate of discharge of water under laminar flow conditions through a unit cross-sectional area of porous medium under a unit hydraulic gradient and standard temperature conditions, usually 20 C." The rate of water discharge through a rock material is not in itself an indication of strength or weatherability. Granites attacked by water deteriorate through solution of silica, but this silica is redeposited onto rock surfaces causing the pores to plug up, thereby decreasing the permeability. Limestones demonstrate just the opposite effect in that deterioration produces increasing permeability (Jaeger, 1972).

Before actually testing a rock specimen, it is necessary to eliminate any air enclosed in the rock pores. This is usually accomplished by saturation under pressure with water. To be absolutely free of air bubbles, it is conceivable that some dense rock samples may require water percolation for up to one week (Jaeger, 1972).

The permeability factor, K, is defined by

$$K = QL/pA$$

where
K = permeability factor = coefficient of permeability,
Q = discharge of water percolated through specimen,
L = specimen length,
p = pressure differential between the two faces of the specimen, and
A = specimen cross-sectional area.

Generally, rocks with low porosity values are considered to have low values of permeability; but permeability is also a function of capillary action. The capillary action may affect ground water conditions to a greater extent than would otherwise be expected by a low porosity rock (Duncan, 1969a).

PETROFABRICS

Jaeger and Cook (1969) indicated that "the study of petrofabrics comprises the study of all fabric elements, both microscopic and macroscopic, on all scales." In general, "microscopic" refers to discontinuous features of the rock fabric between the joint systems. These features produce anisotropy in elastic properties and rock strength characteristics. Petrofabric measurements, which are more rapid than mechanical measurements, provide information relating to preferred particle (crystalline) directions or orientations. This rapidity makes petrofabric measurements more susceptible to statistical analysis. Specimen stress history may be inferred by such parameters as "twin lamellae in calcite and dolomite, quartz deformation lamellae, kink bands, and translation or twin gliding in some crystals..." Implications are obvious that "macroscopic" refers to joint systems or fault systems.

POISSON'S RATIO

The dimensional shortening of a specimen under the action of an axial compressive stress is usually accompanied by an increase in the specimen's cross-sectional area. Poisson's ratio is defined as the unit lateral deformation divided by the unit longitudinal deformation occurring within the elastic, or linear, limit of stresses.

The range of values for Poisson's ratio has been given as -1 to 0.5. Conditions for extreme values can be obtained from two basic equations:

$$G = E/2(1 + \nu)$$

and

$$K = 2G(1 + \nu)/(1 - 2\nu)$$

where
G = shear modulus or modulus of rigidity,
E = Young's modulus or modulus of elasticity,
K = bulk modulus of elasticity, and
ν = Poisson's ratio.

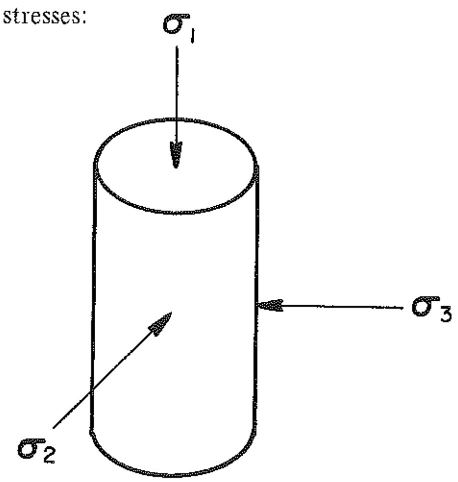
As Seeley and Smith (1959) pointed out, G approaches infinity as ν approaches -1. If ν becomes less that -1, G becomes negative; therefore, the minimum value of Poisson's ratio is $\nu = -1$. Similarly, K becomes infinite when $\nu = 1/2$, and as ν becomes greater than 1/2, K assumes negative values. Therefore, the maximum value of Poisson's ratio is $\nu = 1/2$. Many investigators have reported negative values of Poisson's ratio (Miller and Deere, 1966; cf. Windes, 1950; cf. Wuerker, 1953; cf. Blair, 1955). These negative values are most probably attributable to closure of microfissures and macrofissures at low stress levels and errors in measurement.

PORE PRESSURE

Most rocks at atmospheric pressure contain voids. Some of these voids are continuous and form passages (networks) and others are isolated, formed by grain-boundary cracks. There exists a general agreement among investigators (Murrell, 1965; cf. Hubbert et al., 1959; cf. Robinson, 1959; cf. Heard, 1960; cf. Handin et al., 1963) that, provided rock specimens have connected systems of pores, subsequent rock fracture is primarily controlled by the effective stresses:

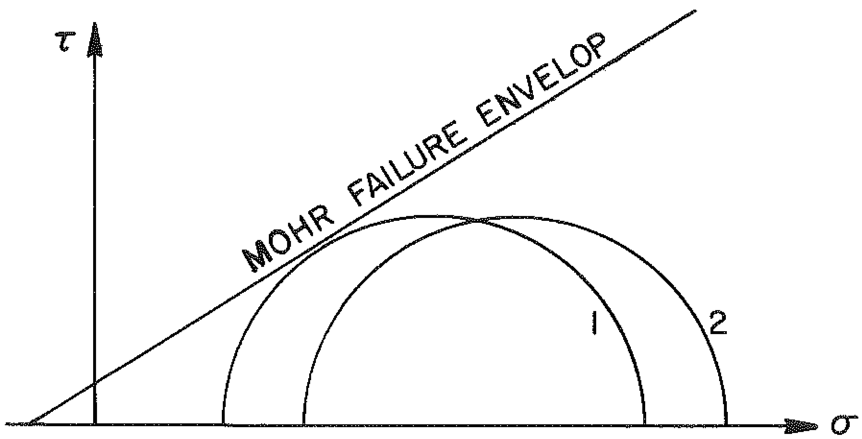
$$\sigma_1' = \sigma_1 - \mu$$
$$\sigma_2' = \sigma_2 - \mu$$
$$\sigma_3' = \sigma_3 - \mu$$

where σ' = effective stress,
σ_1 = principal vertical stress,
σ_3, σ_2 = principal horizontal stresses, and
μ = pore pressure.



INTACT SPECIMEN

Pore pressure effects on rock failure characteristics may be seen from experimental results obtained by Handin et al. (1963). The sandstone stress-strain diagram shows that Curve 2, Mohr circle with zero pore pressure, lies inside the Mohr envelop. As the pore pressure is increased, Curve 2 moves to the left until it becomes tangent to the Mohr envelop (Jaeger and Cook, 1969). Therefore, for a given state of stress, the addition of pore pressure reduces the effective rock strength (Obert and Duvall, 1967).



1- MOHR CIRCLE FOR EFFECTIVE STRESS
2- MOHR CIRCLE FOR TOTAL STRESS

CURVE 1

μ = 500 bars
σ_1 = 5,400 bars
σ_3 = 2,000 bars
σ_1' = 4,900 bars
σ_3' = 1,500 bars

CURVE 2

μ = 0
σ_1 = 5,400 bars
σ_3 = 2,000 bars

Serdengecti, Boozer, and Hiller (1962) have demonstrated that pore fluids may affect the cementing matrix (material) in sedimentary rocks. This implies that additional effects which are not attributable to pressure may occur when water is present in the voids.

POROSITY

The occurrence of pores in the fabric of a rock material results in a reduction in strength and an increase in deformability. It has been widely accepted that a "small volume fraction of pores can produce an appreciable mechanical effect" (Cottiss, Dowell, and Franklin, 1971) in rock material.

ASTM C20-70 defines the apparent porosity, n, as the ratio of the volume of the open pores of the specimen to its exterior volume:

$$n = 100\ (W_{sat} - G_w)/B_v$$

where W_{sat} = saturated weight, in grams to nearest 0.1 gm,
G_w = dry weight, in grams to nearest 0.1 gm,
B_v = total or bulk volume = $W_{sat} - W_{sub}$, and
W_s = suspended weight, in grams to nearest 0.1 gm.

The International Society of Rock Mechanics suggested in 1972 a method for porosity (density) determinations using saturation and caliper techniques:

Apparatus

a) oven capable of maintaining a temperature of 105 C ± 1.5 C for at least 24 hours,
b) dessicator for cooling specimens,
c) instrument to measure dimensions to an accuracy of 0.1 mm,
d) vacuum saturation equipment capable of sustaining a specimen submerged under vacuum of less than 800 N/m^2 (0.08 bar) for at least 1 hour, and
e) balance capable of measurement with an accuracy of 0.01 percent of the specimen weight.

Procedure

a) at least three specimens from a representative sample are machined to a right-cylinder or prism configuration to be tested separately and averaged,
b) specimen bulk volume, B_v, is calculated from an average of several caliper (vernier) readings for each dimension,
c) specimen is dried to constant weight at 105 C and cooled in dessicator for 30 minutes and weighed to determine its grain weight, G_w,
d) specimen is saturated by water immersion in a vacuum of less that 800 N/m^2 for at least 1 hour with periodic agitation to remove entrapped air, and
e) specimen is surface dried with moist cloth and its saturated-surface-dry weight, W_{sat}, is determined.

Calculations

a) pore volume $P_v = (W_{sat} - G_w)/\rho_w g$ and
porosity $n = 100\ P_v/B_v$ to nearest 0.1 percent
where ρ_w = density of water = mass of water per unit volume and
g = gravitational acceleration.
G_w = grain weight = equilibrium weight of a specimen after oven drying at 105 C obtained from successive weighings at 4-hour intervals which do not differ by more than 0.1 percent of the specimen weight, and

b) B_v = bulk volume by
1) Caliper Method -- from regularly shaped specimens of cylinders or prisms measured with a caliper (vernier) or
2) Buoyancy Method -- difference between saturated-surface-dry and saturated-submerged specimen weights (not suited to friable, swelling, or slaking rocks).

$$B_v = (W_{sat} - W_{sub})/\rho_w g$$

where W_{sub} = saturated-submerged weight after being submerged in vacuum of less than 800 N/m^2 for at least 1 hour and weighed under water.

In addition to these techniques, the I.S.R.M. Commission on Standardization of Laboratory and Field Tests mentions:

a) determination of bulk volume, B_v, by

1) mercury displacement method or
2) water displacement method,

b) porosity (density) determination by
1) saturation and buoyancy techniques,
2) mercury displacement and grain specific gravity techniques, or
3) mercury displacement and Boyle's law techniques, and

c) determination of grain volume, G_V, by
1) Boyle's law method or
2) pulverization method.

"Porosity calculated from bulk volume and grain volume using the pulverization method is termed **total porosity**, since the pore volume obtained includes that of "closed" pores. Other techniques give **effective porosity** values, since they measure the volume of interconnected pores only." Thus, porosity is basically a measure of the water retaining capacity of a rock, which is a function of

a) specimen cavities,
b) specimen microfractures,
c) mass joints (discontinuities),
d) particle shape,
e) particle grading, and
f) particle orientation.

RELATIVE ABSORPTION

The on-site availability of water for rock absorption tests is an important engineering consideration. There are conditions in which certain rock types may exhibit increased void volumes due to either rock relaxation and(or) water ingress. This driectly results in strength decreases (Duncan, 1969a).

Hamrol (1961) described the use of absorption as an index property to indicate the degree of alteration (weathering) of rock. This absorption parameter was known as "Quality Index." Empirically, he correlated the modulus of elasticity with the rock absorption (Quality Index) for some extremely weathered granites. This use of absorption as an index property led Rocha (1964) to describe the Quality Index as a method to control the depth for foundation excavation in rock. Rocha also described the use of absorption for exploratory mapping to reduce the number of in-situ tests required at a particular site (Miller and Deere, 1966).

The U. S. Bureau of Reclamation (1953) suggested a vacuum-saturation process to determine the absorption of water in a rock specimen. The suggested procedure eliminates air from rock pores and, additionally, de-airs the water; atmospheric pressure forces water into the rock pores, assuring rock saturation. After 5 days, the rock specimen is removed from the water, surface dried, and weighed to the nearest 0.1 gm. Absorption is given by

$$\text{Absorption (\%)} = 100(W_2 - W_1)/W_1,$$

in which W_1 = weight of oven dry specimen and
W_2 = weight of surface-dried, saturated specimen.

This procedure is approximately the same as the ASTM Standard Test for absorption (ASTM C97-47): oven dry specimen for 24 hours at 105 C ± 2 C, cool for 30 minutes, and weigh to nearest 0.02 gm; immerse specimen in distilled water at 20 C ± 5 C for 48 hours, surface dry, and again weigh to nearest 0.02 gm. The calculation is the same as that noted by the Bureau of Reclamation. (Also see POROSITY and VOID INDEX.)

RESILIENCY

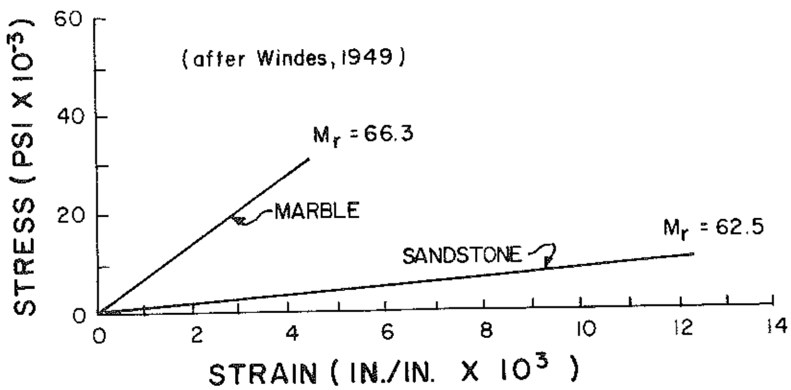
Eshback (1952) indicated the resilience of a material is equal to the external work expended during the process of deformation as long as the state of stress lies within the proportional limit of the material. The total resilience of a specific material is equal to the product of its volume and the modulus of resilence:

$$M_r = \sigma_a(y)^2/2\ E$$

where M_r = modulus of resilience (in.-lb/in.^{3}).
$\sigma_a(y)$ = yield strength, and
E = elastic modulus.

The modulus of resilience is equal to the area encompassed under the elastic, linear, portion of the stress-strain curve -- i.e., strain energy absorbed per unit volume when the material is stressed to its proportional limit. Jastrzebski (1959) defined resiliency as "the capacity of a material to absorb energy in its elastic range".

As would be expected from the formula above, the failure of rock in terms of resiliency is a function of the rock strength, $\sigma_a(y)$, and elasticity, E. However, similar values for the modulus of resilience in different intact rock specimens do not directly signify correspondence in specimen elastic strength or deformation characteristics (Miller and Deere, 1966):



As can be seen from the stress-strain diagram for samples of marble and sandstone, the marble sustains a larger load and smaller deformation than the sandstone specimen, even though both absorb the same order of energy during deformation. Richards (1961) pointed out that a low value of resilience is desirable for good drainage conditions while a high value of resilience is indicative of low internal heat generation.

SIZE

The size effects of specimens have been for the most part ignored by most investigators since an overwhelming desire for replicability of results by different practitioners has caused adherence to standardized specimen sizes. Additionally, in normal laboratory testing, there is not a great difference in sizes of specimens from one laboratory to another.

The tensile strength of rock specimens appears to depend more on specimen size than does any other mechanical property of rock materials (Jaeger and Cook, 1969). However, compression strength, as well as tensile strength, is affected by variations in specimen size. Weibull's theory (Jaeger and Cook, 1969) accounts for this situation by suggesting that rock specimens may be visualized as being comprised of smaller samples (constitutent parts) and that, like the proverbial chain, the rock is only as strong as its weakest subsample (smaller sample). Tests conducted by the U. S. Bureau of Reclamation on concrete seem to substantiate this conclusion. Accepting this theory implies that strength is a function of microfissuration of rock specimens. Therefore, strength would be a function of size since larger specimens would be more likely to contain fissures to a greater extent than would smaller specimens. Bernaix (1966) tested cylinders of 10-, 36-, and 60-mm diameters having aspect ratios of 2:1 in uniaxial compression. He suggested that two values may be useful in rock classification systems (Jaeger, 1972):

R_{10}/R_{60}

where R_{10} = mean crushing strength of 10-mm cylinders and
R_{60} = mean crushing strength of 60-mm cylinders

and

S_d/M

where S_d = standard deviation for compressive test series and
M = mean for series of tests (same diameters).

Bernaix's results are shown, in part, below:

Crushing Strengths of Rock Types

Rock Type	Fissures	S_d/M	R_{10}/R_{60}
Very Poor Gneiss	Microfissures Abundant Microfractures	0.37	2.90
Poor Gneiss	Microfissures Microfractures Abundant Macrofractures	0.30	1.90
Jurassic Limestone	Few Microfissures Abundant Macrofractures	0.25	1.40
Biotite Gneiss	Average Microfissures	0.22	1.25
Compact Limestone	No Microfissures	0.005	1.00

SONIC PULSE VELOCITY

Standardized pulse techniques are ususally employed to determine the sonic velocities for intact rock specimens (Miller and Deere, 1966). This technique utilizes a low-amplitude, short-duration stress pulse generated at one end of a seated (100 - 150 psi (689 - 1034 kPa) seating load) specimen. Miller and Deere used a high voltage (1000 volt maximum) pulse applied to a transmitting crystal from a high-gain, high-pass signal amplifier. The generated voltage pulse imparted a dilation motion to the crystal which in turn repeatedly stressed one end of the specimen. "The arrival of the pulses at the opposite end of the specimen, causes mechanical contraction of the receiver crystal, generating a voltage across the crystal faces" (Miller and Deere, 1966). The velocity, V_p, of the dilatational waves, or the bulk compressional velocity, is calculated from

$$V_p = L/t$$

where V_p = sonic pulse velocity (in an unbounded medium),
L = specimen length, and
t = travel time of wave through specimen.

The constrained modulus, M_c, is computed empirically from the sonic pulse velocity (dilatation wave velocity) by the following equation (Miller and Deere, 1966):

$$M_c = \rho V_p^2$$

where ρ = mass density.

Additionally, an approximate value for Young's modulus (modulus of deformation) of rock material may be calculated from the sonic velocity (Duncan, 1969a):

$$E_{dyn} \approx \rho V^2_{lab}/g = G_b V_{lab}/74 \text{ lbs/in.}^2$$

where	E_{dyn}	=	dynamic modulus of deformation,
	V_{lab}	=	dilatational wave velocity obtained in the laboratory,
	g	=	gravitational constant, and
	G_b	=	bulk modulus.

SWELLING

Rock materials with high clay contents are prone to swelling, weathering, and disintegration when exposed to wet-dry weathering cycles. The swell potential of a rock is a function of its water content (Duncan, 1969a):

Swell Potential	Moisture Content
100%	$w \leqslant SL$
0%	$w = w_{sat}$

where	w	=	moisture content,
	SL	=	shrinkage limit, and
	w_{sat}	=	saturation moisture content.

Index tests to predict the mechanical performance of rock with regard to swelling are best used in rock classifications to compare one rock with another (Franklin, 1972). The necessity of determining swelling characteristics may be seen by noting that the mechanical characteristics of clay-rich rocks may vary between wide limits; tests are necessary to determine their performance in contact with water. Three swelling index tests advocated by the International Society for Rock Mechanics Commission on Standardization of Laboratory and Field Tests are the swelling pressure index, the swelling strain index, and the unconfined swelling strain index.

Swelling Pressure Index is a measure of the pressure necessary to constrain an undisturbed rock specimen at constant volume when it is immersed in water. Duplicate specimens are prepared for all swell index testing; one for water content determination and the second for the swell test.

Test Procedure

1) The apparatus designed for soil consolidation testing is assembled and a small axial force is applied to the specimen within the metal specimen ring.
2) The consolidation cell is flooded with water to a level covering the top porous plate.
3) Applied force is regularly adjusted to maintain zero specimen swell; specimen thickness should be maintained to within 0.01 millimeters.
4) Swelling force is recorded as a function of elapsed time. Swelling force is recorded until it reaches a constant level or passes a peak.

Calculation

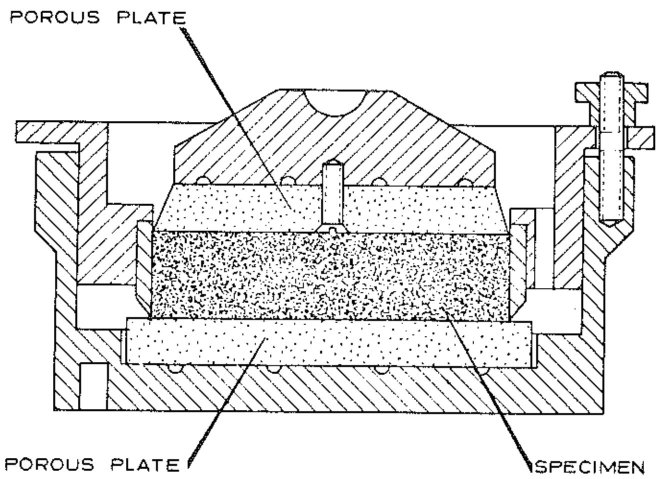
$$\text{Swelling Pressure Index} = F/A$$

where	F	=	maximum axial swelling force recorded during the test and
	A	=	cross-sectional area of specimen.

Swelling Strain Index is intended to measure the axial swelling strain developed against a constant axial pressure or surcharge when a radially confined, undisturbed rock specimen is immersed in water. Laboratory-determined saturation swelling pressure is significant in assessing the extent to which failure may occur by rock fracturing upon the ingress of water to a partially unconfined rock mass (Duncan, 1969a). Naturally, this laboratory swelling strain does not apply directly to in-situ rock because of the presence of joints or cracks in the rock mass (Duncan, 1969a; Franklin 1972).

Test Procedure

1) The soil consolidation apparatus is assembled; a specimen is loaded axially to a surcharge pressure of 29 kPa.



2) The consolidation cell is flooded with water so as to cover the top porous plate.
3) Swelling displacement is recorded as a function of elapsed time. Swelling displacement should continue to be recorded until it reaches a constant level or passes a peak.

Calculation

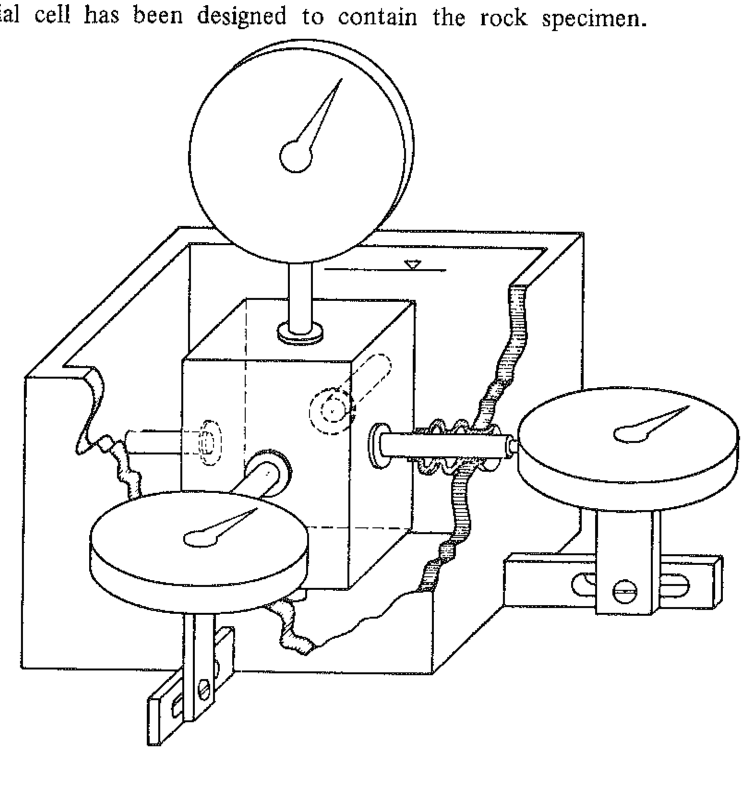
Swelling Strain Index = 100 d/L (in percent)

where d = maximum swelling displacement and
L = initial specimen thickness.

Swelling Strain Unconfined Index is intended to measure the swelling strain developed when an unconfined, undisturbed rock specimen is immersed in water. Franklin recommends that this index be applied to rock specimens which do not change their geometry appreciably on slaking.

Test Procedure

1) A special cell has been designed to contain the rock specimen.



2) Gauge shafts are arranged to coincide with the axis or axes of the rock specimen.
3) Bearing plates (glass or other hard material) are positioned at each gauge point and cemented to the specimen with a water-stable adhesive.
4) The cell is flooded to cover the specimen.
5) Swelling displacement or displacements are recorded as a function of elapsed time. Swelling displacement is continuously recorded until it reaches a constant level or passes a peak.

Calculation

Unconfined Swelling Strain Index in x direction = 100 d/L (in percent)

where d = maximum swelling displacement in x direction and
L = initial distance between gauge points in x direction.

Undisturbed rock specimens should be tested since rock fabric condition has an effect on permeability and swelling characteristics. If the need exists, remolded specimens may be prepared following standard procedures for soil compaction. Such testing is necessary to assess joint-filling materials. Although these indices are commonly required for classification or mechanical characterization of relatively soft rock materials, harder rocks may be classified if they are in an advanced state of weathering or similar deterioration. Rock materials which disintegrate during these tests should be characterized by using soil classification tests such as liquid and plastic limit determinations, particle-size distribution, and(or) clay mineral content (Franklin, 1972).

TENSILE STRENGTH

Although tensile strength of rock is an important parameter in rock operations and theories of failure mode, direct measurements of the tensile strength commonly are not made. This is primarily due to the difficulties involved, especially in terms of the specimen and conditions (Reichmuth, 1968; Stagg and Zienkiewicz, 1969; Jaeger and Cook, 1969). Minor scratches on the surfaces of specimens to be tested in tension have a pronounced effect on strength -- this is true especially for glass and metals. End conditions are significant but to a lesser extent in rocks because of the heterogeneous nature of rock and the large number of natural mechanical defects inherent in rock material. Usually, large numbers of tests are necessary to determine the tensile strength of rock because the tensile strength is also extremely susceptible to specimen size.

It should be noted, however, that, from a structural overview, the tensile stresses are not of a sufficient magnitude to be of serious consideration except in cases of unsupported spans of bedded (layered) rock material. In-situ measurements indicate that most underground stresses are not tensile but compressive (Obert and Duvall, 1967).

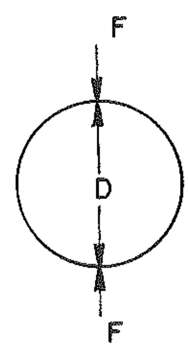
The most practical method of direct tensile strength determination for engineering purposes consists of placing a specimen under a tension load. This is accomplished by attaching metal end caps to a cylindrical rock sample by means of an epoxy resin and simply pulling the metal caps apart. Indirect methods of tensile strength determination include the Brazilian test and the point load tensile test.

Brazilian Test

Essentially, this is a test in which a specimen disc is loaded diametrically, producing an approximately uniform tensile stress over the major portion of the vertical diameter. Upon specimen failure, a measure of tensile strength is obtained (Coates, 1970; Cottis, Dowell, and Franklin, 1971):

Brazilian strength, $T_s = 2F/\pi DT$

where F = external load at failure,
D = disc diameter, and
T = disc thickness (length).



Generally, $T/D < 1$. Before failure occurs, some crushing of the sample may take place at the points of load application. Wedging action from the ensuing crushed zones and the probable occurrence of non-elastic stress distribution are apt to complicate tensile measurements. Fairhurst (1964) suggested the elimination of these complications by applying the load on an arc of $\tan^{-1}$ 0.125 and using a non-yielding loading pad. Paone and Bruce (1963) have reported an empirical formula to account for this crushing action:

$$T_s = 0.79\ F/(D - 1.7F/Q_u)^2$$

where Q_u = compressive strength (psi),
F = external loads (lbs), and
D = disc diameter (in.).

Point Load Test

This test uses externally applied vertical compressive forces to induce internal tensile stresses as in the Brazilian Test (Brock and Franklin, 1972). Probably the greatest advantage of this method is that there is no specimen preparation problem (Reichmuth, 1968; Cottiss et al., 1971) -- the tensile stresses maximize in the interior of the specimen. Therefore, surface irregularities are only of minor importance. Additionally, since the load is applied at only two points, parallel surfaces as required in uniaxial compression tests are not necessary. Excellent reproducibility and test expediency further ensure this test as an index test. The only limitation of this test is that a sample size should be chosen small enough so that significant localized failure at the points of load application does not occur. Of course, one must note that some magnitude of indentation under load will occur for most rocks, but this is not considered a localized failure (Miller and Deere, 1966; Reichmuth, 1968).

Reichmuth demonstrated an empirical relationship between tensile strength and failure load (Reichmuth, 1963; Richmuth, 1968):

$$T = \sigma_t = 0.96\ F/D^2$$

where T = tensile strength,
F = load at failure (lbs), and
D = core diameter (in.).

In this test, each specimen was loaded midway between the ends. The load was applied in smooth, even strokes by means of an hydraulic hand pump. Miller (1965) tested 28 different rock types using the Reichmuth point-load test and found a relationship between the ultimate uniaxial compressive strength $\sigma_{a(ult)}$ and the tensile strength:

$$\sigma_{a(ult)} = 21\ \sigma_t + 4000\ \text{lbs/in.}^2$$

Many investigators in the field simply assume the tensile strength is approximately 10 -15 percent of the compressive strength. The U. S. Bureau of Mines is a little more conservative in that it uses

$$S_c \approx -16\ S_t$$

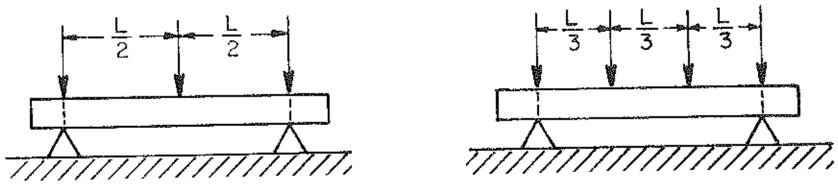
where S_c = compressive strength and
S_t = tensile strength.

The tensile strength is approximated as about six percent of the compressive strength. Advantages of the point load test are numerous (Reichmuth, 1968):

a) the test may be applied to irregularly shaped specimens,
b) it may be adapted to high or low temperature use, and
c) if the rock specimen has anisotropic strength properties, the failure will align itself parallel to the planes of weakness -- therefore, one may determine the magnitude and direction of the minimum tensile strength about the axis.

Modulus of Rupture

A test for the modulus of rupture is a measure of the tensile stresses generated by an unsupported span under load (Reichmuth, 1968):



Specimens are generally costly to prepare and surface irregularities affect test results. This type of test is not well suited to the requirements of a classification system nor determination of index properties.

TEXTURE

The texture of a rock material consists of the arrangement of constituent material grains, the bonding mechanism, and often the grain size. Deere (1963) has proposed that most rock materials may be grouped into one of three textural categories: interlocking, cemented, or laminated-foliated. For these three categories, the grain size has been neglected. Much study has indicated that grain size, the only textural parameter commonly used in engineering descriptions of intact rock, is not a good index to the mechanical behavior of rocks.

Franklin (1970) reported that sorting, crystallinity, and porosity are of greater mechanical significance than grain size. These properties may be quantified by using percentages of four textural constituents: coarse fragmented (detrital or clastic) material, coarse crystalline material, muddy or microcrystalline matrix, and pore space. Each constituent should be represented by a volumetric percentage and a size value. Franklin suggested that textural descriptions can be supplemented by noting the homogeneity and isotropy of the rock: isotropic, oriented (grains in preferred orientation), or segregated (layers of differing grain size or mineral content), Also, the coherence, fissility, or friability of the rock material may be observed.

Rock petrological properties, fabric, mineral constituents, and grain size should be considered as a logical inclusion in a simplified engineering rock classification system (Cottiss et al., 1971). These properties may be obtained utilizing a low-power stereobinocular microscope. Miller and Deere (1966) reported the use of petrologic analysis of thin sections of rock oriented at right angles to the axis of a particular core specimen. A Zeiss petrographic microscope was used to examine these sections and a mechanical point counter was used to determine the significant mineral percentages contained in rock specimens, except for virtually monomineral sedimentary and metamorphic rocks.

TOUGHNESS

Toughness is a physical property related to the ability of a material to absorb energy during plastic, nonlinear, deformation under load. The energy representing the work required to fracture or fail the test specimen is measured by the area under the stress-strain curve of the material (Miller and Deere, 1966).

The modulus of toughness is the maximum amount of energy a unit volume of the specimen can absorb without fracture. For materials exhibiting a parabolic stress-strain curve (concrete, etc.), the modulus of toughness can be estimated by the following equation (cf. Jastrzebski, 1959):

$$M_t = 2\sigma_{a(ult)}\, \epsilon_f/3$$

where M_t = modulus of toughness,
$\sigma_{a(ult)}$ = ultimate stress, and
ϵ_f = strain at failure.

The equation above indicates that high toughness values are associated with high strength and ductility values; brittle materials usually have low toughness values since they characteristically exhibit small plastic deformations before fracture occurs.

It might be profitable to utilize the Toughness Index for compact shale specimens:

$$\text{Toughness Index} = I_t = T_\mu = I_p/I_f$$

where I_p = plasticity index = liquid limit - plastic limit and
I_f = flow index = slope of the flow curved obtained from a liquid limit test, expressed as the difference in water contents at 10 blows and at 100 blows (ASTM D653-67).

Miller and Deere (1966) pointed out that toughness is primarily governed by the strength of the rock grain or crystal matrix structure. In effect, rock toughness is a function of the individual grain strength or mineral strengths. "The toughest rocks comprise those having strong minerals embedded in a strong matrix or cement" (cf. Shepherd, 1951).

Toughness and hardness substantially depend on the same factors. Both index properties are functions of the binding force between grains and atoms in grains (Miller and Deere, 1966). In addition, both are also closely related to the yield strength of the rock material (cf. Jastrzebski, 1959).

UNIT WEIGHT

According to Miller and Deere (1966), the unit weight is one of four index properties demonstrating the "greatest promise for serving as indices of the engineering behavior for intact rock." The Standard Definitions of Terms and Symbols Relating to Soil and Rock Mechanics (ASTM D 653-67) defines unit weight as "the weight per unit volume." Eight variations to this term are also defined:

a) Dry Unit Weight = γ_d = weight of soil solids per unit of total volume of soil mass,
b) Effective Unit weight = γ_e = unit weight of soil multiplied by the height of overburden soil,
c) Maximum Unit Weight = γ_{max} = dry unit weight defined by the peak of a compaction curve,
d) Saturated Unit Weight = γ_{sat} = wet unit weight of soil mass when saturated,
e) Submerged Unit Weight = γ_{sub} = weight of solids in air minus weight of water displaced by solids per unit of volume of soil mass ($\gamma_{sat} - \gamma_j = \gamma_{sub}$),
f) Unit Weight or Water = γ_w = weight per unit volume of water (1 gm/cm^3),
g) Wet Unit Weight = γ_{wet} = weight per unit of total volume of soil mass, irrespective of the degree of saturation, and
h) Zero Air Voids Unit Weight = γ_z = weight of solids per unit volume of a saturated soil mass.

In general, "the more compact and denser rocks . . .have a higher strength than those of less density" (Miller and Deere, 1966). But the unit weight is not a sensitive index of compressive strength. Empirically, the unit weight has a higher degree of correlation ($r = 0.784$) in linear regression analysis with the modulus of deformation (at a stress level of 50 percent ultimate strength) than with the compressive strength ($r = 0.604$) (Miller and Deere, 1966).

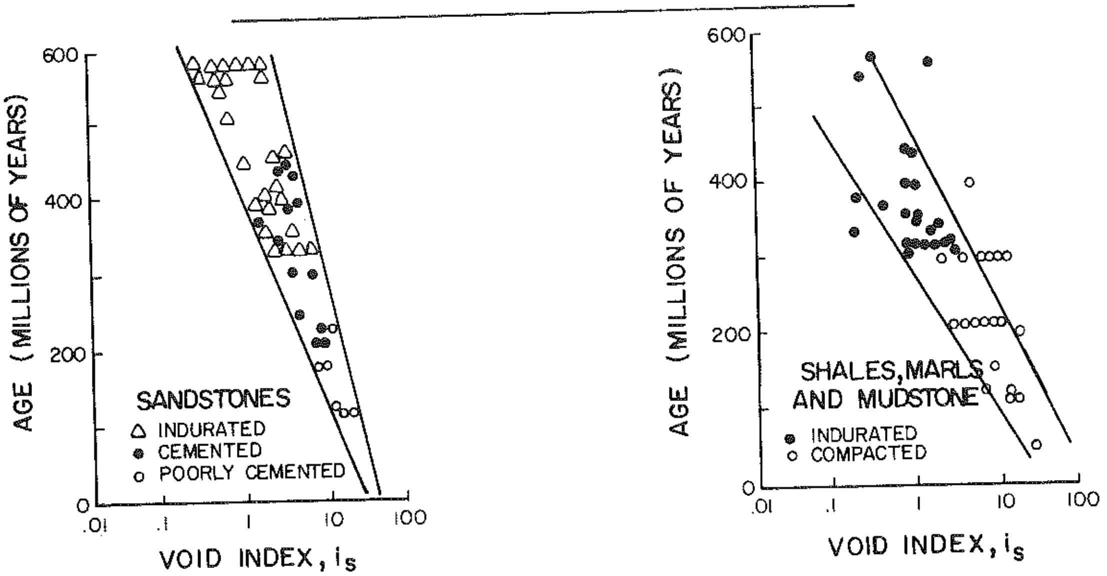
The laboratory procedure for determining unit weights as recommended by Miller and Deere, is as follows:

a) oven dry sample and weigh to nearest 0.1 gm,
b) weigh sample after 2 weeks in ambient air environment of laboratory, and
c) during the 2 weeks, the lengths and diameter of the samples are measured to nearest 0.0001 in., with the average of five measurements taken as the correct dimensions to use in calculating volumes.

VOID INDEX(INDEX OF ALTERATION)

Alteration, the degree of weathering, is a function of stress history, stress relief, type of rock material, changes in the voids and pores, and the age of the rock material. However, the void index, i_s (defined below), is a function of the specific type or rock material (compactness) and rock age (sustained compression) (Franklin, 1972):

Relative i_s	Rock Type
Low	Indurated Sandstones Indurated Shales
Moderate	Normally Cemented Specimens
High	Poorly Cemented Specimens Poorly Compacted Specimens



(after Duncan et al., 1968)

Alteration of rock causes the void index to increase, and for this reason, the void index is also called the alteration index (Jaeger, 1972).

Engineers (Serafim, 1968) and geologists use basically the same technique to establish the void index. Rock material is oven dried at 105 C for 12 or 24 hours, and the weight of the water absorbed by the oven-dried specimens is calculated after immersion in water for 12 or 24 hours:

$$i_s = \text{weight of water absorbed/weight of oven-dried rock.}$$

The International Society for Rock Mechanics' Commission on Standardization of Laboratory and Field Tests (1972) has suggested a standard method for determining the void index using the rapid absorption technique. Similar to rock absorption, the void index is defined "as the weight of water contained in a rock sample after one-hour period of immersion, as a percentage of its initial dessicator-dry-weight". This "test should only be used for rocks that do not appreciably disintegrate when immersed in water" (Franklin, 1972). The procedure for such a determinations is:

1) select sample of a minimum of ten rock specimens, each weighing at least 50 gm;
2) sample is air dried for 24 hours in an environment of dehydrated silica gel crystals;
3) sample is removed from container with surrounding gel and brushed clean of loose rock and gel crystals; weight of sample to nearest 0.5 gm = A;
4) sample is replaced in containers and water is added to fully immerse sample; container is briefly agitated and left to stand for 24 hours;
5) sample is surface dried with a moist cloth; weight of sample to nearest 0.5 gm = B; and
6) void index = i_v 100/A(B - A)

Perami and Thenoz (1968) have developed permeability tests with air to indicate any chemical changes or alterations in the rock substance. Air may infiltrate a rock specimen through minute fissures which do not normally allow penetration by water. Since air does not react chemically with the mineral rock substance, no alteration (weathering) of rock occurs (Jaeger, 1972):

$$\text{Void Coefficient} = i_1 = V/(V + V_v)$$

where V = dry volume of pulverized rock specimen and
V_v = volume of pores.

The values V and $V + V_v$ are obtained by measuring the apparent specific weight of the rock sample and the true specific weight when reduced to powder.

WEATHERING

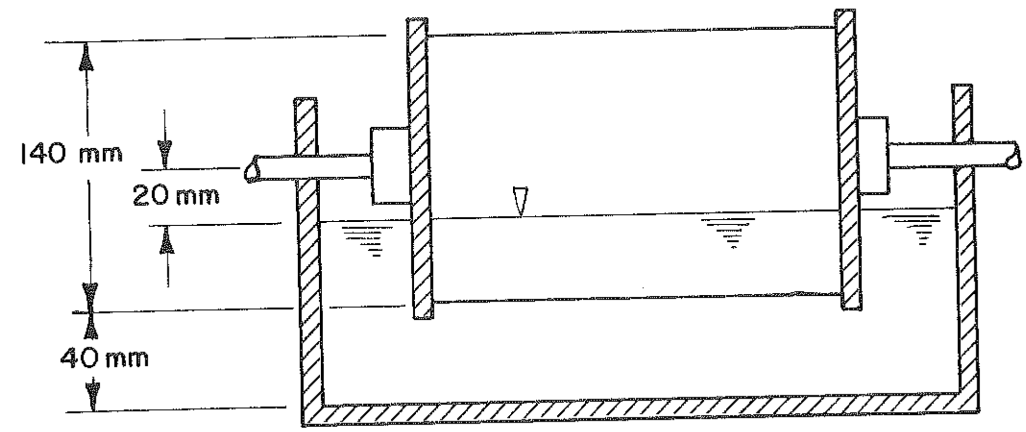
The degree of weathering may be measured by hardness tests. Large variations in hardness from one rock type to another are of more concern than slight differences in the degree of hardness demonstrated by different rock specimens of the same type. A relative term for the variations in hardness may be the best approach to describe these variations (Hamrol, 1961; Rocha, 1964; Cottiss et al., 1971). Hamrol described an index of alteration which can be correlated with laboratory and in-situ test results for rock mass strength and compressibility. Essentially, this index of alteration was described in terms of short-term water absorption by rock samples and so is applicable in differentiating zones of weathered and altered rock.

Cottiss et al. (1971) described a slake test originally conceived in 1968. Selected rock samples are immersed in a two-percent aqueous solution of sodium hexametaphosphate for one hour. The samples then are washed on a No. 7 BS 2 sieve. Slake loss index, an estimate of the degree of breakdown, is obtained as the weight of rock lost through the sieve as a percentage of the total dry weight. The fracture or wet fracture index was a second index obtained by visually comparing the material retained on the sieve with a set of standard retained weights and forms such that "1" indicated a specimen which was completely broken up and "10" was the standard for an almost intact specimen.

As part of the work of the ISRM Commission on Standardization of Laboratory and Field Tests, Franklin (1972) defined the slake durability index. The Slake Durability Test "is intended to assess the resistance offered by a rock sample to weakening and disintegration when subjected to two standard cycles or drying and wetting." The test is performed in the following manner (Franklin and Chandra, 1971):

Apparatus

1) A test drum, encasing a 2.00-mm mesh cylinder 100-mm long by 140-mm diameter, with a solid fixed base. Drum has a solid removable lid and must be sufficiently strong to retain its shape.
2) A trough to retain the drum (supported with axis horizontal, allowing free rotation), capable of being filled with a slaking fluid to a level 20 mm below the drum axis.



3) A motor drive to rotate the drum at 20 rpm ± 2.5% for a period of 10 minutes.
4) An oven capable of maintaining a temperature of 105 C ± 1.5 C for at least 12 hours.
5) A balance with an accuracy of 0.5 gm.

Procedure

1) A sample is selected to include 10 rock lumps, each weighing 40 - 60 gm, to give a total sample weight of 450 - 550 gm. Lumps should be approximately spherical in shape and corners should be rounded during preparation.
2) The sample is placed in the drum and dried to constant weight at a temperature of 105 C (usually 2 to 6 hours in oven). The weight of the sample plus drum is recorded (A).
3) The drum and sample are replaced in the trough which is filled with slaking fluid (usually tap water at 20 C) to a level of 20 mm below the drum axis. The drum is rotated at 20 rpm for 10 minutes.
4) The drum plus the retained portion of the sample are dried to constant weight at 105 C and are weighed; weight is recorded (B).
5) Repeat procedures 3) and 4); the weight (C) of the drum plus the retained dried portion of the sample is recorded.
6) The drum is brushed clean and its weight (D) is recorded.

Calculation

$$\text{Slake Durability Index (in percent)} = I_{d2} = 100\,(C - D)/(A - D),$$

for the second drying and wetting cycle.

If the second cycle slake durability index is less than 10 percent, the rock samples should be further characterized by their first cycle slake durability index (Gamble, 1971; Franklin, 1972):

$$\text{Slake Durability Index (in percent)} = I_{d1} = 100\,(B - D)/(A - D).$$

Gamble (1971) also noted that three or more cycles of slaking and drying may be useful when evaluating rocks of higher durability.

In searching for quantitative numbers of measures for weathering estimation, several empirical observations have been made (cf. Kolomenskii, 1952; cf. Talobre, 1957; Iliev, 1966):

a) specific gravity is the most indicative and reliable of the physical properties with respect to weathering;
b) modulus of elasticity, modulus of deformation, and residual deformations tend to decrease with increasing weathering; and
c) ultimate bearing resistance decreases with increasing weathering.

As Iliev (1966) demonstrated, physico-mechanical indicators may be grouped into strongly changeable, moderately changeable, and weakly changeable categories according to the degree of the influence of weathering:

Strongly Changeable	Moderately Changeable	Weakly Changeable
Pore Volume		
Modulus of Elasticity	Bulk Density	
Modulus of Deformation		Specific Gravity
Ultimate Bearing Resistance	Poisson's Ratio	
Sonic Velocity		

Iliev also introduced the weathering coefficient, K, to estimate quantitatively the degree of rock weathering:

$$K = (V_o - V_w)/V_o$$

where K = coefficient of weathering,
V_o = sonic velocity in a fresh (unweathered) rock, and
V_w = sonic velocity in a weathered rock.

Rocks virtually unaffected by weathering processes would have $K = 0$ while $K = 1$ would imply a completely weathered (deteriorated) rock.