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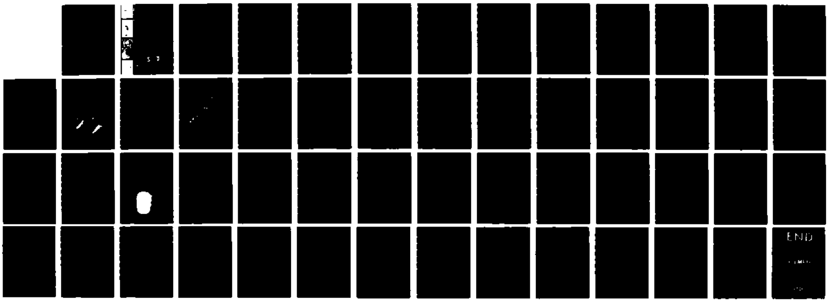

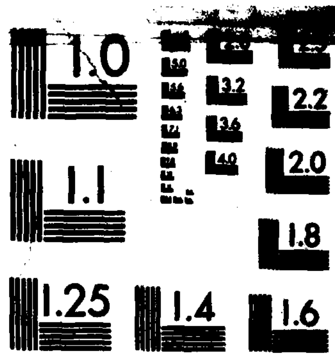

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GEOTECHNICAL DESCRIPTIONS OF ROCK AND ROCK MASSES

by

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Geotechnical descriptors for rock and rock mass are suggested for use in the field that can be readily understood by geotechnical engineers and contractors. Several rock and rock mass properties and descriptors were determined to be important in geotechnical applications, include ~~including~~ tunnel support, slope and foundation stability, and rock excavation. The descriptors are rock type, strength, discontinuity characterization, weathering, rock quality designation (RQD), ground-water conditions, and rock density. The rock name

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20. ABSTRACT (Continued).

or type should be retained in field description, but uncommon rock names should be accompanied by a brief definition to enable the user to relate to more common rock types. Rock strength should be described quantitatively by the point load index test. Descriptions of discontinuities should include measurement and classification of aperture and a determination of whether the discontinuity is open or tight; filling thickness and composition; wall asperity or roughness; and orientation of individual discontinuities, sets and systems. ~~The~~ use of stereographic projection and unambiguous azimuthal notation to describe discontinuity orientation is recommended. Bieniawski's classification of rock weathering, which classifies degree of weathering and describes the appearance in the field, is recommended. ~~The~~ use of RQD in field descriptions as developed by Deere is suggested for certain applications. Field recognition and description of seepage and groundwater conditions along discontinuities based on simple observations of the amount of water present and estimates of discharge are recommended to precede and augment the design of more elaborate pore pressure and seepage analysis investigations. Wet density of rock samples can be determined in the field on core specimens by the standard Rock Testing Handbook Methods.

PREFACE

This report represents a part of the work at the US Army Engineer Waterways Experiment Station (WES) under Civil Works Investigational Studies (CWIS) Work Unit 31754, "Rock Mass Classification Systems," sponsored by the Office, Chief of Engineers (OCE), US Army. The OCE technical monitors were Messrs. Paul R. Fisher and Ben Kelly.

The report was written by Mr. William L. Murphy, Engineering Geology Applications Group (EGAG), Engineering Geology and Rock Mechanics Division (EGRMD), Geotechnical Laboratory (GL). The principal investigator was Mr. Hardy J. Smith, Rock Mechanics Applications Group (RMAG), EGRMD. The work was under the direct supervision of Mr. Jerry S. Huie, Chief, RMAG, and under the general supervision of Dr. Don C. Banks, Chief, EGRMD, and Dr. William F. Marcuson III, Chief, GL.

Commanders and Directors of WES during the conduct of this study and the preparation of this report were COL Tilford C. Creel, CE, and COL Robert C. Lee, CE. Technical Director was Mr. Fred R. Brown.

CONTENTS

CONVERSION FACTORS, US CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

US customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	2.54	centimetres
feet	0.3048	metres
pounds (force)	4.4482	newtons
pounds (force) per square inch	6.8948	kilopascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre

GEOTECHNICAL DESCRIPTIONS OF ROCK AND ROCK MASSES

PART I: INTRODUCTION

Background and Purpose

1. Rock mass classification systems currently in use produce divergent results when used by different engineers and geologists. Geologic descriptions of rock are often misunderstood by engineers and contractors or are of insufficient engineering value. Consequently, the US Army Engineer Waterways Experiment Station (WES) is developing rock mass classification systems for engineering applications. The purpose of the study reported herein is to recommend geotechnical descriptors for rock mass properties and characteristics that can be determined in the field and understood and used by geotechnical engineers and contractors. Development of the rock and rock mass descriptors is necessary for consistent application of rock classification systems.

Approach

2. A working list of rock mass properties and conditions important to various engineering applications was formulated. A systematic study of existing descriptive terminology and classifications was then made to develop the recommended geotechnical descriptors presented in this report (Table 1). The selection of properties and conditions was strongly influenced by existing rock mass classification systems developed by others for specific applications, such as tunnel support, slope and foundation stability, and rock excavation. The rock descriptors discussed in this report are rock type, strength, discontinuity spacing, condition of discontinuity, discontinuity orientation, weathering, rock quality designator (RQD), ground-water conditions, and rock density. The discussion of each rock mass property descriptor includes the definition of the term, the importance in rock mass classification for geotechnical use, the previous and existing classifications and measurement techniques, and the recommended description and/or measurement for US Army Corps of Engineers (USACE) usage.

PART II: DESCRIPTORS

Rock Type

Definition

3. Rock type is the identification given a rock by the geologist. Examples of rock type are limestone, dolomite, sandstone, granite, banded gneiss, and mudstone. Rock types are defined individually in publications such as the Glossary of Geology (American Geological Institute (AGI) 1972). Local or colloquial rock names are sometimes used in the literature. Authors should consult publications such as the AGI glossary to achieve uniformity in rock identification. Formation name is another identifier of assemblages of one or more rock types occurring in a particular location or region. Formation names may be inconsistent from one location to another, are constantly subject to change in the literature, and are less uniform and more difficult to define than are rock types.

Importance and existing classifications

4. The rock type is the product of a classification procedure that the geologist performs to categorize the mode of formation and certain physical characteristics of the rock. In addition, however, rock name implies qualitative information on many properties to be considered as a general guide in a geotechnical project. Such information includes strength, predicted joint systems, the probability of the presence of bedding planes and possible weak zones, permeability, hardness, or resistance to abrasion, and perhaps cohesion and angle of internal friction. For example, the rock types "granite," "slate," and "dense basalt" would likely represent rocks of high compressive strength; "sandstone" would lead an investigator to suspect relatively high permeability; "clay shale" would imply bedded sediments with low shear strength. Most field data supplied for engineering evaluation of rock and rock mass behavior include the rock name even though the name may not be entered in the formal rock classification scheme, indicating that the geologist and geotechnical engineer use information implicit in the rock name in their evaluation of a problem.

5. Attewell and Farmer (1976) discuss the relevance of geologic classification of rocks to geotechnical applications. They suggest that the standard classification based on texture, mineralogy, and origin does not

adequately distinguish rock types for engineering purposes. Unweathered igneous rocks, for example, tend to be sound engineering materials relative to many sedimentary rocks, although the geologic distinctions within the igneous class are complex and many. Igneous rocks are relatively sound because they are agglomerates of strongly bonded minerals and have low primary porosity and high competency. Franklin (1970) realized that abundant geologic rock names are often applied to materials that differed insignificantly in their engineering importance. He evaluated previous attempts to simplify the geologic nomenclature for geotechnical use, but concluded that no improvement in usage could be made by a mere reduction in the number of rock classes.

6. Stagg and Zienkiewicz (1968) support the retention of geologic naming of rocks by citing several examples of mechanical properties that can be inferred from the rock name. They point out that texture, fabric, and anisotropy in the rock are usually a product of the rock's origin (mode of formation) and are related to mechanical properties of rock behavior. Most igneous rocks, for example, are generally isotropic in mechanical properties; whereas, many sedimentary rocks are laminated or bedded and are thus considerably anisotropic. The metamorphic rocks, which are often banded, foliated, and composed of platy minerals, can also be very anisotropic. Geologists commonly classify rocks on the basis of origin and mineral composition.

7. Table 2 presents a classification of common rock types based essentially on physical appearance represented by color, texture, grain (or crystal) size, and types of minerals present. The table lists the more common rock names that may occur in a geologic report as well as other terms that are sometimes applied synonymously to the rock or rock group. Igneous rocks can be described as coarse (phaneritic) or fine grained (aphanitic) depending on whether the mineral crystals can be seen by the naked eye. The many rock types within the igneous groups differ basically in the relative percentages and kinds of the feldspar minerals and the amount of quartz they contain. The general color of the rock also reflects its mineralogy. Volcanic rocks are further classified by their grain size and mode of emplacement (i.e., flow, intrusion, pyroclastic fall, etc.). The sedimentary rocks are described as clastic (particulate or mechanically deposited) and nonclastic (primarily chemical precipitates), and as coarse or fine grained. Nonclastic sedimentary rocks are subdivided into organic and inorganic types. The grain-size boundary between coarse and fine (Table 2) is ambiguous because of the recognition

of siltstone as an identifiable rock type. Some sedimentary rock types (marlstone, for example) exhibit characteristics of both divisions. The metamorphic rocks can show pronounced anisotropy and have been subdivided into isotropic and anisotropic. Table 2 is not a complete list of rock names; it is a reference guide to help the user relate less common terms and rock names that may be encountered in the literature to more commonly used or accepted terms.

Recommended descriptors

8. The geologic rock name should be included in geotechnical reports. The standard and common names that are in wide geographic use or acceptance should be used rather than the colloquial or locally popular terms. Use of the three basic classes of rocks--igneous, sedimentary, and metamorphic--should be continued because they are widely understood and are part of the rock-naming process. If an uncommon rock type must be described, the name should be accompanied in parentheses by a brief definition so that the user of the field log can relate to the general class of rock being described. For example, the rock name "syenite" might be qualified by adding "the quartz-deficient equivalent of granite," because granite is a common rock name. It should be understood, however, that the field geologist must usually make determinations of more than just engineering properties in order to accurately correlate between borings or exposures to determine continuity of the rock type and rock mass and to detect the possible presence of faults or structure not sampled. Correlation requires stratigraphic, paleontologic, and mineralogic detail that may be of little interest to the engineer, but must appear in the field logs for use by the geologist in his geotechnical evaluation of site. Formation and other stratigraphical names may be used in the geologic report, but it should be remembered that these names apply only to specific geographic locations or regions.

Strength

Definition

9. The term "strength" as applied to a rock specimen or to a rock mass has been defined in field investigations in many ways; for example, by qualitative descriptors referring to the relative density or crushing resistance of the rock under a hammer blow, by quasi-quantitative descriptors using the hardness or rebound of the specimen as determined by a simple apparatus, and by

measurements of compressive strength derived indirectly from point-load index and directly from uniaxial compression tests. Compressive strength is a measurement of the compressional load required to cause an unconfined or confined specimen to fail, as in a uniaxial (unconfined) or triaxial (confined) compression tests. Shear strength is determined in the triaxial test chamber by applying axial loads to specimens under several confining pressures or by subjecting samples to direct shear stresses. The following discussion deals only with field estimates of compressive strength. Field estimates of shear strength are not common.

Importance

10. An evaluation of the strength of rock provides an upper limit of the strength of the rock mass, represents an estimate of the true rock mass strength in massive unjointed rock masses, and is a simple and useful means of classifying the rock. Rock strength also implies the effectiveness to be expected of tunneling and other rock excavation machines.

Previous descriptors

11. The Core Logging Committee of the South Africa Section of the Association of Engineering Geologists (AEG 1978) suggests a rock strength classification as shown in Table 3* which is based on hardness as defined by simple field examination. The classification relates the strength (hardness) descriptors to simple field tests based on abrasion resistance or point-load tests, and to ranges of compressive strength derived from a relationship of Jennings and Robertson (1969). The AEG classification is designed to offer subdivisions of strengths, particularly in the lower ranges of stresses for design of foundations or slope stability and in the higher ranges of stress as in tunneling considerations. The ranges of compressive strengths used in Table 3 are similar to those of the geomechanics classification of Bieniawski (1979) which is based on the scheme of Deere and Miller (1966). Table 4 compares the schemes of Bieniawski and Deere and Miller. The slight difference in values in units of pounds per square inch between the two classifications in Table 4 is for convenience in converting to metric (SI) units in Bieniawski's classification. Bieniawski added the 150 psi** lower limit to distinguish rock

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* Tables 3, 4, and 5 are presented for discussion only, and should not be inferred as having been accepted for use.

** A table of factors for converting US customary units of measurement to metric (SI) units is given on page 3.

from soil. Bieniawski's geomechanics classification has been recommended for USACE consideration in tunnel support design (Bieniawski 1979). The qualitative description of compressive strengths expressed by AEG's six hardness classes (Table 3, field test) does not correlate with similar terminology describing strength in other classifications, such as those suggested by Coates (1970) (Table 5) and Deere and Miller (1966) (Figure 1). Medium hard rock is characterized in Table 3 by a strength range of 1450 to 3625 psi, which would correspond to a "very weak" rock by Coates (Table 5) and a "very low strength" rock by Deere and Miller (Figure 1). The apparent lack of consistency between the classifications and in the meaning of the terms "hardness" and "strength" confuses the user. Therefore, the use of a strength classification based on hardness is not recommended. Instead, rock strength preferably should be described in the field investigation by quantitative strength values determined

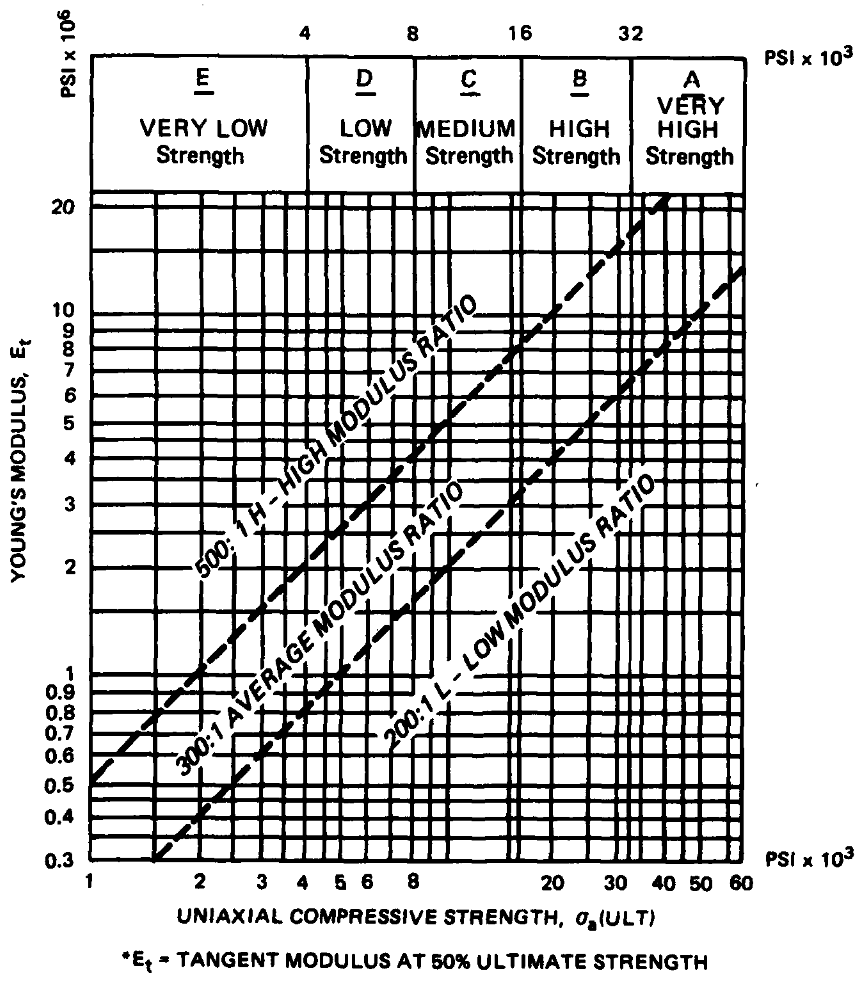

Figure 1. Rock strength classification (modified from Deere and Miller 1966)

by accepted standard strength tests such as uniaxial compressive tests or point-load indices (paragraph 12). A qualitative descriptor, using the range of terms from "very low" to "very high" strength, can be added if desired, preferably with the strength ratings of Bieniawski (Table 4).

Index tests for strength

12. Rebound tests. Good correlation between field rebound tests and compressive strength was reported by Deere and Miller (1966) and between point-load and compressive strength by Deere and Miller (1966), D'Andrea, Fischer, and Fogelson (1965), and Franklin, Broch, and Walton (1971). Deere and Miller (1966) compared Shore scleroscope and Schmidt hammer rebound indices with uniaxial compressive strengths of intact NX core specimens of 13 geologically distinct rock types. From the somewhat curvilinear relationships, Deere and Miller developed logarithmic rock strength charts with which uniaxial compressive strength can be estimated if the Shore or Schmidt (Figure 2) values and the unit weight of the rock are known. The scleroscope and Schmidt hammer are similar instruments that impart a definite amount of energy to a rock specimen by a free-falling hammer in the scleroscope and by a spring-loaded hammer in the Schmidt hammer. The rebound of the hammer from the rock surface is measured and recorded. The devices are relatively inexpensive and rapid and simple to employ on NX core at a project site. However, the rebound tests

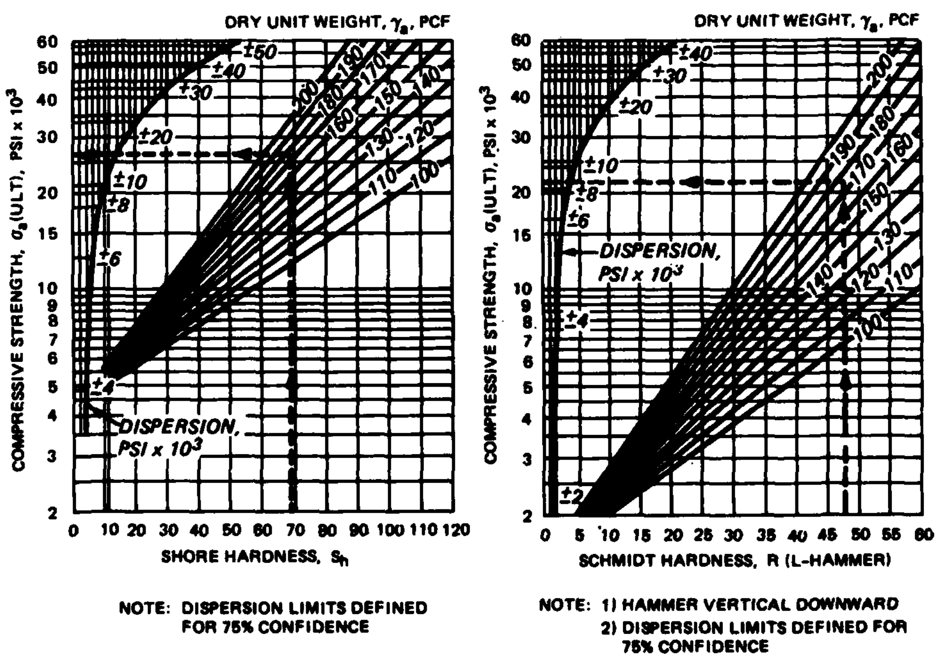

Figure 2. Rock strength charts based on Shore (left) and Schmidt (right) hardness tests (from Deere and Miller 1966)

are insensitive to strength changes, and the results are strongly influenced by variation in testing techniques (WES 1982). Attewell and Farmer (1976) question the usefulness of rebound indices, but concede that they may have value as spot checks on rock strength. The Geological Society Engineering Group Working Party (1977) of Great Britain states that there is only a 75 percent probability that the laboratory-determined uniaxial compression strength will fall within 50 percent of the strength determined by the rebound hammer test in the correlation chart (Figure 2) of Deere and Miller (1966).

13. Point-load test. The point-load test index is conducted by applying compressive point loads diametrically to a specimen and measuring the load at failure. A point-load strength index is derived from the failure load and the distance between loading platens. The specimen actually breaks in tension, but a linear relationship between point-load index and compressive strength has been shown by several authors. D'Andrea, Fischer, and Fogelson (1965) demonstrated a linear relationship for data for rock core specimens from 49 locations, Bieniawski (1975) for data for rock from four locations, and Deere and Miller for rock core samples from 27 locations. Their combined data are shown in Figure 3, a graph of point-load tensile strength index versus uniaxial compressive strength. The graph (Figure 3) relates data for various rock core diameters. Bieniawski (1975) developed a chart for converting point-load index of cores of other than NX standard size. Specimens of irregular shape can also be tested with the point-load apparatus (Attewell and Farmer 1976, and Franklin 1970). An informative dissertation on the history of development, effects of specimen shape factors, and testing procedures for the point-load strength test is available in a paper by Broch and Franklin (1972). Use of the point-load apparatus as an index for the quick field evaluation of compressive strength is recommended as a proposed standard (No. 325-82) in the Rock Testing Handbook (WES 1982).

Recommended rock strength descriptors

14. Strength is an important property required to describe adequately a rock type when index classification is used in engineering applications. Simple hammer and penknife tests have been used but seldom give objective, quantitative, or reproducible results. The uniaxial (unconfined) compression test has been widely used for rock strength classification but requires machined specimens and is therefore a slow technique, essentially confined to the laboratory. This report recommends that the point-load test index be used

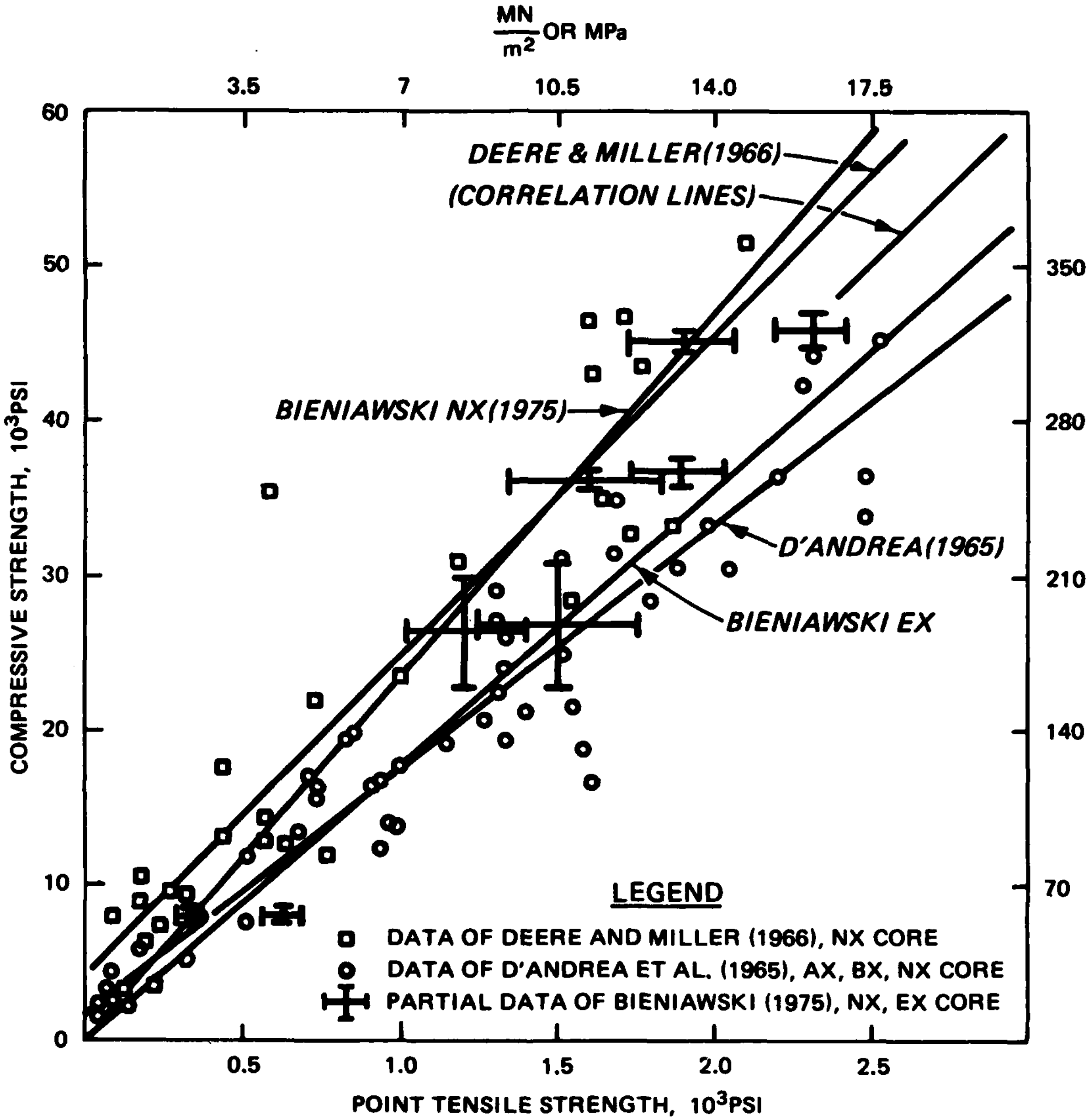

Figure 3. Relation of point-load strength index to compressive strength

to provide rock strength descriptions from the field. The point-load test has proven to be a reliable method of determining rock strength properties, and portable equipment that lends itself well to field use is commercially available. The advantages of the point-load test are:

- a. Smaller forces are needed so that a small and portable testing machine may be used.
- b. Specimens in the form of core or irregular shapes are used and require no machining.

c. More tests may be made for the same cost of uniaxial compression tests which allows adequate sampling even when rock conditions are variable.

d. Fragile or broken materials can be tested, so there is less chance of results being biased in favor of more competent strata.

e. Results show less scatter than those for uniaxial testing as reported by Broch and Franklin (1972).

f. Measurement of strength anisotropy is simplified.

If a strength classification is desired in addition to point-load values, the scheme of Bieniawski (1979) (Table 4) should be used. The report of field point-load test results should include the point load index corrected to a reference diameter of 50 mm by use of a correction chart and the uniaxial compressive strength derived from an index-to-strength conversion graph. The Rock Testing Handbook (No. 325-82) should be consulted for procedures in conducting and reporting point-load index tests.

Discontinuity

Definition

15. The term "discontinuity" encompasses all perceivable breaks or divisions in a rock mass. Discontinuities include structural features such as faults and joints and depositional features such as bedding planes, erosional surfaces, and other contacts. Some engineers define "joint" as any break in the continuity of the rock mass, including structural (stress) breaks and bedding features. Most usage distinguishes joints from bedding. However, "bedding" implies that the discontinuities are parallel or subparallel; whereas, a joint system usually consists of several sets of joints at different orientations. "Bedding" also implies that adjoining rock types may be different in character (for example, in grain size or strength); whereas, joints often separate a rock mass of unchanging rock type. Joints, faults, and bedding planes sometimes occur congruently, for example, in the case of joints occurring along bedding planes. Most often, however, a distinction between structural and bedding features can be made in the field. The distinction between bedding and jointing should be retained.

Importance

16. The importance of discontinuity analysis is expressed in the

following quotation from the International Society for Rock Mechanics (ISRM) (1978):

> The majority of rock masses, in particular those within a few hundred meters from the surface, behave as discontinua, with the discontinuities largely determining the mechanical behavior. It is therefore essential that both the structure of a rock mass and the nature of its discontinuities are carefully described in addition to the lithological description of the rock type. Those parameters that can be used in some type of stability analysis should be quantified whenever possible.
>
> For example, in the case of rock slope stability certain quantitative descriptions can be used directly in a preliminary limit equilibrium analysis. The orientation, location, persistence, joint water pressure and shear strength of critical discontinuities will be direct data for use in analysis. For purposes of preliminary investigation the last two parameters can probably be estimated with acceptable accuracy from a careful description of the nature of the discontinuities. Features such as roughness, wall strength, degree of weathering, type of infilling material, and signs of water seepage will therefore be important indirect data for this engineering problem.
>
> For the case of tunnel stability and estimation of support requirements, all the descriptions will tend to be indirect data since a direct analysis of stability has yet to be developed. However, a careful description of the structure of a rock mass and the nature of its discontinuities can be of inestimable value for extrapolating experience of support performance to new rock mass environments. Descriptions should be sufficiently detailed that they can form the basis for a functional classification of the rock mass.
>
> In time, as descriptions of rock masses and discontinuities become more complete and unified, it may be possible to design engineering structures in rock with a minimum of expensive in situ testing. In any case careful field description will enhance the value of in situ tests that are performed, since the interpretation and extrapolation of results will be made more reliable.

Previous classifications and descriptors

17. For engineering purposes descriptions of discontinuities should be quantitative when possible, pertinent to engineering usage, and should include characteristics readily measurable or determinable in the field. Characteristics of discontinuities that meet the above restrictions are spacing (or bed thickness), true orientation or attitude within the rock mass and relative orientation with respect to excavation surfaces, and condition (surface

roughness, width of opening, degree of weathering, and filling material properties). Some or all of the above characteristics have been applied in rock classification for engineering purposes by Bieniawski (1979); Deere, Merritt, and Coon (1969); Coates (1970); John (1962); Underwood (1967); Barton, Lien, and Lunde (1974); Franklin (1970); AEG (1978); and others. Other descriptions of discontinuity geometry such as surface area and area intensity have been suggested by Fookes and Denness (1969) (in Attewell and Farmer 1976). The following discussion develops descriptive terminology for discontinuity spacing, condition, and orientation.

Discontinuity spacing

18. Determination of spacing. Spacing is the distance separating planes of discontinuity in a rock mass. The term "joint spacing" is analogous to the term "bedding thickness." Ideally, the spacing applies to the three-dimensional rock mass, but realistically measurements of spacing are usually made in the field in one or two dimensions. Borehole core and photolog spacing measurements are one-dimensional (along a line) and most rock exposure measurements are two-dimensional (in a plane). Borehole measurements are biased in favor of discontinuities lying at nearly right angles to the borehole axis and against those lying parallel to the borehole axis because more of the former intersect the borehole. Similar but less severe bias occurs in two-dimensional rock exposure measurements. The geologic report should qualify the reported spacing values by stating the methods used to determine spacing. Preferably the report should make the determination of three-dimensional spacing by analyzing all complementary data from boreholes, trenches, cuts, and other exposures.

19. Previous classification/description schemes. Bedding thickness was classified by McKee and Weir (1953), and their classification was adopted by Pettijohn (1957) for geologic usage. Their terms (Table 6) were based on field examinations of sedimentary rock units. Rock strata* less than 1 cm thick were termed "laminations," and strata greater than 1 cm thick were termed "beds." Subdivisions of the two major groups (beds and laminations) were added for classification (Table 6). The splitting properties listed in Table 6 are vaguely defined and have been used by geologists in the field in

* "Strata" (stratum) is the general term for layers of rock; "bed" and "lamination" are terms of magnitude.

lieu of quantitative bed thickness values. The splitting terms are widely used and are included for completeness only. Deere (1964) developed a similar quantitative scheme for engineering purposes and applied it to joint spacing as well as strata thickness (Table 7). Figure 4 compares discontinuity spacing schemes of several authors. The scheme of Deere (1964) is similar to an earlier rock mechanics scheme devised by Klaus John (1962). The Core Logging Committee of the South Africa Section of AEG (1978) proposed a purely logarithmic division of discontinuity spacing in a range from 0.3 to 100 cm. Coates' (1970) rock mechanics classification used the terms "broken," "blocky," and "massive" to describe the spacing of rock mass discontinuities with a narrower range of values than the above schemes. No rationale for the choice of numerical values of the various divisions of spacing categories was given by the authors, but all of the schemes approximate a log-scale division because the widths of the categories on a logarithmic scale are roughly the same within a given scheme. Log-plots allow subdivisions of equal widths at both extremes of a scale. The difference between the four schemes is basically the number of categories (or classes) and the position of division points between classes. Deere's engineering usage scheme allows good resolution in the middle of the scale (close to moderately close to wide), is widely published, and agrees

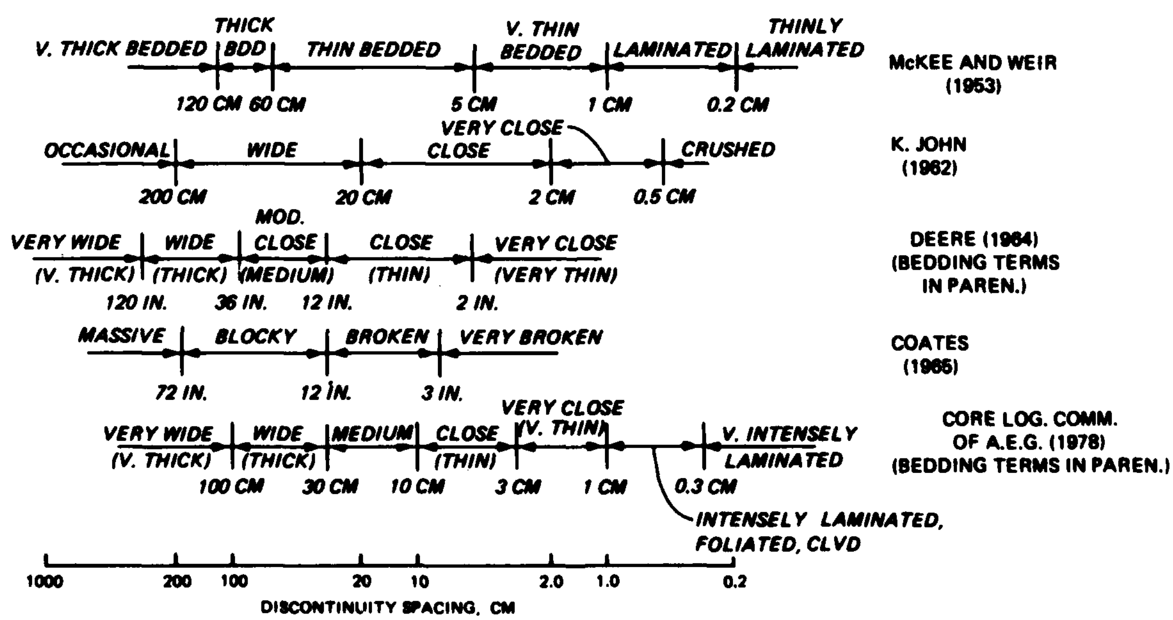

Figure 4. Discontinuity spacing divisions of several investigators

well with subdivisions of the spacing scale widely used by geologists (McKee and Weir 1953; and adopted by Pettijohn 1957). Deere's system has also been adopted by Bieniawski (1979), whose geomechanics classification has been recommended for USACE usage.

20. Recommended descriptor for spacing. This report recommends a slightly modified discontinuity spacing terminology and class division of Deere (1964). The actual spacing measured in the field should also be stated because further subdivisions may sometimes be needed for special cases. Deere's system does not subdivide bedding thickness less than "very fine" (less than 2 in.); whereas, McKee and Weir (1953) subdivided further by defining strata less than 1 cm as "laminated" (Table 6 and Figure 4). Laminations are commonly ascribed to shales, and because shales are an important rock type in engineering problems, the terms "laminated" and "laminations" should be retained. The system of Deere (1964) can then be modified by assigning "very thin bedding" to the range 1/2 in. (in place of 1 cm) to 2 in., and "laminated" to strata less than 1/2 in. thick. Table 8 shows the system adopted in this report. A term analogous to "laminated" for use with joints is not considered necessary. The system omits "massive," which is often used to describe rock units that display no visible bedding planes and often behave isotropically and appear homogeneous, such as a massive sandstone. Although the term "massive" is in widespread use, it is believed unnecessary in the system because the terms "very wide" and "very thick" can be applied. Bedding or joint geometries, such as cross-bedding, that are not adequately described by the terms in Table 8 should be described separately.

Condition of discontinuity

21. Definition and importance. Condition describes the roughness, the degree of weathering, the width of opening (or aperture), and the character and presence of filling material of rock mass discontinuities. Condition is an important consideration in classifying rock mass quality for engineering purposes. For example, joint condition accounts for as much as 25 percent of the rock quality rating of Bieniawski's (1979) geomechanics classification scheme. Joint roughness and joint alteration are essential factors in computing the rock mass quality of Barton, Lien, and Lunde's (1974) system. Franklin (1970) included openness (width), roughness, and infilling material for his fissure descriptions for a mechanical classification of rock properties. The Core Logging Committee for AEG, South Africa Section (1978), recognized as

significant to engineering behavior of rock masses the following discontinuity features:

a. Separation of fracture walls (aperture).

b. Filling.

c. Roughness (asperity).

d. Orientation.

The shear strength along discontinuities and the stability of the rock mass are affected by the height and strength of surface irregularities (roughness) and the strength and thickness of the filling material, which is often clayey and considerably weaker than the host rock. Aperture determines the secondary permeability or effective porosity of a rock mass. Orientation (discussed under a separate heading) of discontinuities and sets* of discontinuities influence the stability of excavations in rock. Weathering of discontinuities is discussed in Part III.

22. Aperture. Discontinuity wall separation has been described qualitatively and quantitatively. Franklin (1970) suggested using only the terms "tight" and "open" to describe discontinuity aperture. Similarly, Deere (1964), referring primarily to discriptions of discontinuities in rock cores, preferred the terms "tight" for discontinuities the surfaces of which could be tightly fitted together and "open" for those the surfaces of which could not be intimately mated. Deere also recommended that the ranges in aperture be recorded for open discontinuities. Other writers have suggested quantitative ranges and divisions for classifying aperture, but with considerable disagreement, as shown in Table 9. The usefulness of aperture description is in the determination of secondary permeabilities (or effective porosity) and water inflows and in evaluating the shear strength of the rock mass as controlled by discontinuities.

23. The effect of aperture on shear strength should be evaluated with respect to the filling within the discontinuity and the roughness (asperity amplitude and waviness) of the surfaces. For example, the combined effects of aperture, roughness, and filling were summarized in AEG, South Africa Section (1978) after a discussion by D. R. Piteau:

a. With tight discontinuities (no separation, no filling), the shear strength depends on properties of the wall rock.

b. With open discontinuities with measurably thick filling but with some interlocking of asperities, the shear strength depends

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* Discontinuities having the same orientation comprise a "set."

on both filling properties and on wall rock strength.

c. With open discontinuities with thick filling and no interlocking of asperities, the shear strength is controlled by the properties of the filling material.

Figure 5 illustrates the influence of aperture and filling thickness on discontinuity shear strength.

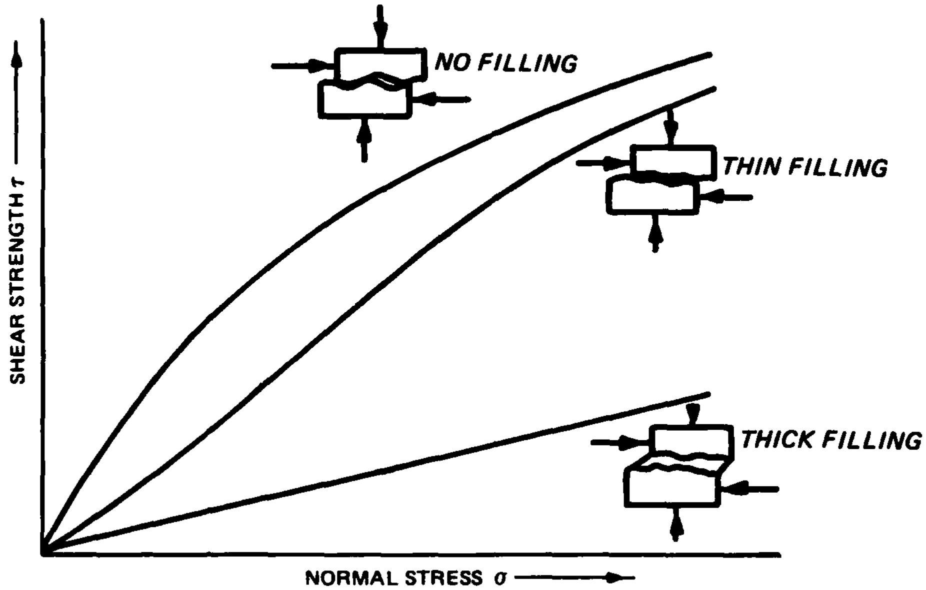

Figure 5. Relationship between shear strength and normal stress for discontinuities with different thickness of gouge infilling (Hoek and Bray 1974)

24. Effective porosity* of a rock can be estimated from analysis of the volume of open discontinuities determined from borehole photographic or television logging. Effective porosity was defined for the borehole photography analysis of jointing in the foundation of Teton Dam** as the total open discontinuity (joint) volume divided by the volume of the boring. For the Teton Dam analysis, a determination of joint condition and aperture was made for every visible joint in the boring walls. Joint condition was described as tight if no aperture was present, open if separation of the walls was consistent, and partially open if the joint walls did not remain separated

* The term "effective porosity" as used herein denotes the fracture porosity of rocks that have little or no primary (grain) porosity.

** D. C. Banks. 1977. "Borehole Photography Analysis, Teton Dam," Letter Report, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

throughout the film record. The volume of partially open joints was halved for effective porosity calculations; the volume of open joints was taken at 100 percent. The primary considerations in the analysis, however, were whether a joint was tight or open and the actual aperture of the open joint.

25. Discontinuity apertures can also be measured in excavation made in rock if the excavation surfaces are fresh, for example, in machine-bored tunnels. Borehole determinations can be made using impression-type packers, which expand against the borehole walls and take imprint of wall irregularities such as open discontinuities. The ISRM (1978) emphasizes, however, that measurements of the exposed surfaces of open discontinuities may not be representative of water-conducting potential because wall roughness may reduce flow velocities, and flow in joints may be tubelike rather than sheetlike. Also, open discontinuities may be filled or closed at some distance from the measured exposure. In situ permeability testing (pump testing, bailing, falling head, etc.) is a more reliable indicator of flow through apertures.

26. Recommended descriptors for aperture. For the above reasons the classification or division of ranges of aperture is considered unnecessary in the general description of discontinuities in the field. Instead, the simple determination of tight or open should be made and the actual aperture measured normal to the plane of the discontinuity recorded along with and qualified by other joint conditions including filling and roughness. Special applications requiring a subdivision of classes of aperture, such as the "partially healed" subdivision for porosity estimation mentioned above, can be approached for specific cases requiring more detailed investigative procedures.

27. Filling. The material within the walls of a discontinuity should be described in terms of its thickness, relative grain size and, if possible, its composition. Fillings such as calcite and gypsum that are subject to removal under construction stresses or by solution may produce greater apertures than those initially measured. If the thickness of such a filling is recorded, the effect of subsequent widening of the aperture can be predicted or expected. Fillings of cohesionless materials such as wall rock alteration products or infiltrating clastic materials may flow out when the rock mass is excavated. Fillings of clays with a high activity number* can undergo

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* Activity of a clay is defined as the plasticity index divided by the weight percent of particles smaller than 0.002 mm.

considerable volume change in the presence of varying moisture conditions. Brekke and Howard (1972) suggest that swelling clays can cause a loss of strength through swelling and may produce considerable swelling pressure when confined. Low activity or inactive clays are relatively weak materials with correspondingly low resistance to shear along discontinuities. Fillings of metamorphic minerals such as chlorite, talc, and graphite impart low coefficients of friction to discontinuity walls even when present in thin coatings (Brekke and Howard 1972). Thick fillings of materials of low seismic velocity attenuate shock waves and can influence blasting results in rock excavation. Table 10 describes materials often filling discontinuities and the potential problems associated with the fillings (Brekke and Howard 1972).

28. Recommended descriptors for filling. The thickness of the filling (width of the filled discontinuity) limits the degree to which discontinuity wall roughness increases shear strength along the discontinuity (paragraph 29). The minimum and maximum thickness of the filling should be measured and determination made of the mineralogy and approximate grain size and gradation of the filling. If the mineralogy cannot be determined by field observation, a sufficient sample of the material should be obtained for laboratory determination, especially where the presence of active clays is suspected. The ISRM (1978) suggests the thickness be measured to 10 percent and an estimate made of the average (modal) width. Description of important complex filled discontinuity zones such as shear zones should be accompanied by a scaled sketch of the zone (ISRM 1978). Water conditions of filled discontinuities should be described as suggested in paragraphs 45-48.

29. Roughness. Roughness (asperity) of discontinuity walls is described by the presence or absence of surface irregularities and their magnitudes. Site investigation should include sufficient description of surface roughness to aid in the design of laboratory and in situ shear strength testing programs. For example, Goodman (1968) evaluated the effects of roughness, filling thickness, and water content on load deformation curves of a large number of laboratory direct shear and several in-situ block shear tests on discontinuities. Figure 6 summarizes Goodman's evaluation in which he defined four types of stress-strain responses. Figure 6 also illustrates the effects of condition, especially roughness, on peak strengths, residual (ultimate) strengths, and stiffnesses. The several peaks displayed on the stress-strain curves for

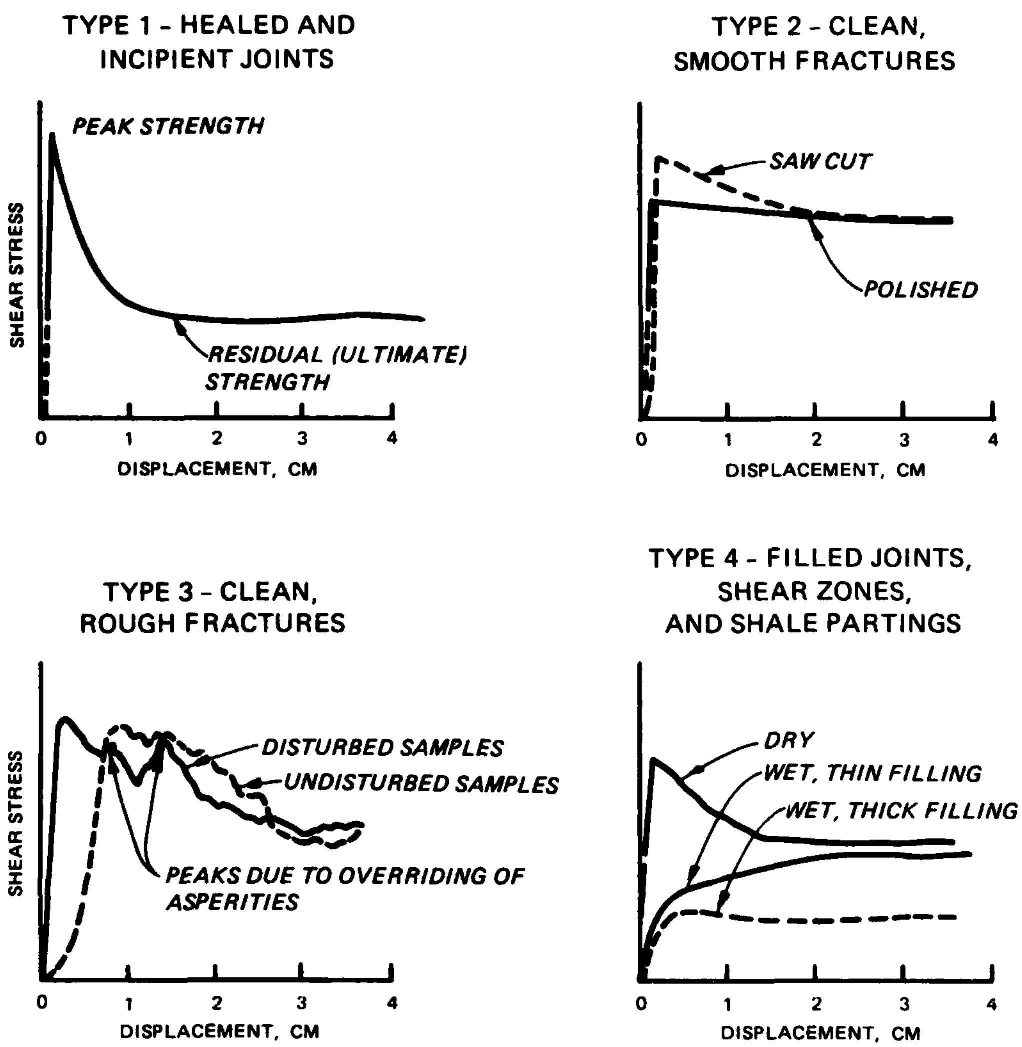

Figure 6. Typical shear stress-deformation relationships for various discontinuity surface conditions (after Goodman 1968)

type 3 discontinuities (clean, rough fractures) are reportedly caused by overriding of asperities during shear displacement. The responses of type 4 discontinuities (filled joints, shear zones, and shale partings) were sensitive to water content and filling thickness. The clean surfaces (types 1-3) reportedly were unaffected by water contents.

30. Bieniawski's (1979) Geomechanics System of rock mass classification and the AEG (1978) Core Logging Committee's terminology suggest a roughness description scheme based on visual examination in the field. Barton, Lien, and Lunde's (1974) Q System for rock mass classification also evaluates roughness (an important factor in the Q System) from simple field observations. The asperities described by the above authors are small enough (amplitudes of

millimetres or tenths of an inch) to be readily apparent on core-sized samples. Surface irregularities of larger amplitude are described collectively as waviness. Waviness refers to large-scale undulations which affect the strength of the in situ discontinuity. Characteristics of waviness are not apparent on small specimens obtained from drilled core. Unevenness refers to small-scale roughness which affects the strength normally sampled in laboratory or medium-sized in situ shear tests. Characteristics of unevenness are apparent on drilled core specimens.

31. Recommended descriptors for roughness. The International Society for Rock Mechanics (ISRM 1978) suggests means for measuring roughness by profiling and other methods. However, actual measurement of asperity amplitude and wavelength on discontinuity surfaces is tedious, nonstandardized, and only practical on large exposed surfaces. It is reasonable to suggest a simple qualitative roughness description terminology for field use. Four previously published roughness classification schemes are shown in Table 11. Bieniawski's (1979) and AEG's (1978) schemes are similar: (a) their terms are defined, (b) their terms are simple, and (c) their schemes have fewer categories than the other schemes. Bieniawski's scheme is recommended primarily because it distinguishes the condition of slickensides, the polished and striated surfaces characteristic of shear planes. Bieniawski's descriptions and definitions are given in Table 12.

Orientation

32. The orientation of discontinuities can be described in absolute terms (orientation in space) and in relative terms (orientation with respect to excavation surfaces, tunnel axes, stress fields, etc.). Orientation can apply to individual discontinuities, and to sets of discontinuities making up a system. Bieniawski (1979), following Wickham, Tiedemann, and Skinner (1972), preferred a qualitative assessment of orientation relative to the alignment of the axis of a driven tunnel, and developed descriptive terminology for various relative alignments and dips (Table 13). Bieniawski extended the use of the orientation classification to foundations and slopes in his geomechanics classification, but did not explain the extension. Hoek and Bray (1974) recognized the importance of discontinuity orientation to slope stability. Figure 7 illustrates several simple types of slope problems produced by adverse orientation of planes of weakness. The use of stereoplots to describe the interactive geometry of the slope and

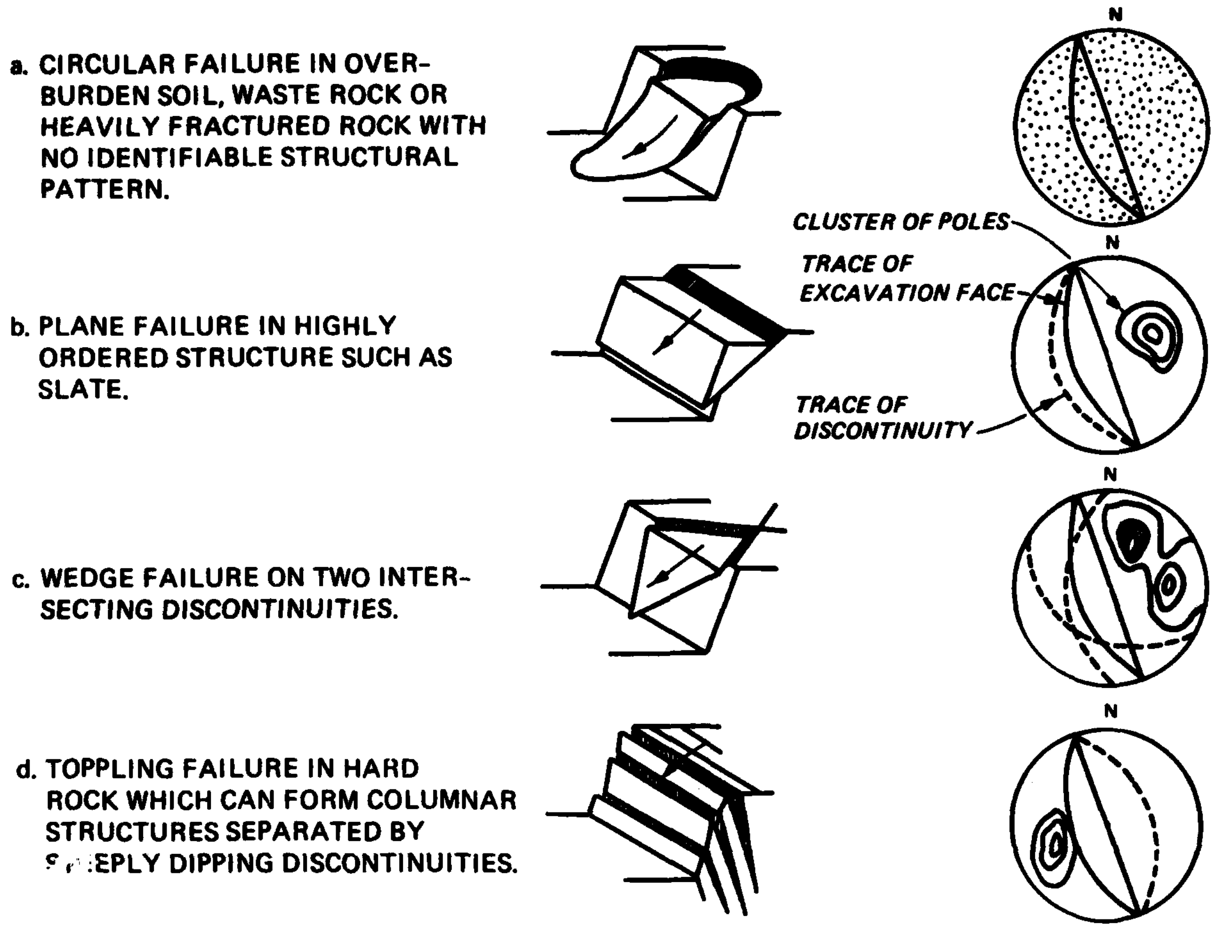

Figure 7. Main types of slope failures and stereoplots of related structural conditions (after Hoek and Bray 1974)

failure planes in Figure 8 is discussed below.

33. The importance of discontinuity orientation to blasting efficiency and to excavation stability were discussed in the Corps manual on systematic drilling and blasting for surface excavation (Department of the Army, Office, Chief of Engineers (OCE) 1972). Orientation may adversely or favorably control alignment of the cut face, produce overbreakage of the rock mass by transmittal of blast energy beyond the design grade or surface, cause ravelling of the excavated surface, or result in postexcavation failure of the finished excavation walls. Similar effects apply to underground excavation blasting.

34. Orientation notation. The orientation of planar discontinuities can be determined at a single point on the plane by recording the direction of a horizontal line on the plane (the strike) and the maximum angle of

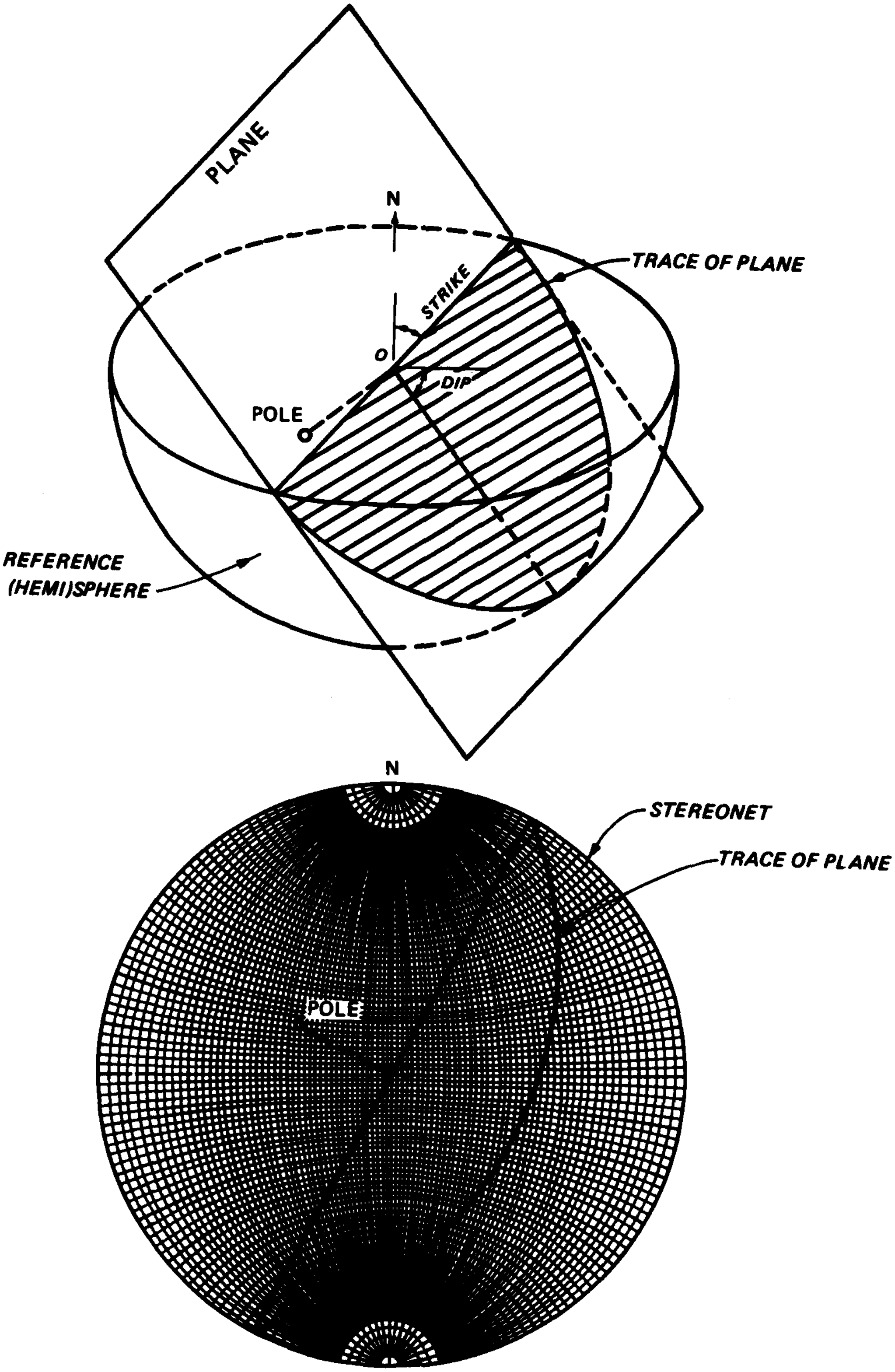

Figure 8. Representation of plane in stereographic projection from reference hemisphere (top) to stereonet (bottom)

inclination of the plane from the horizontal (the dip). Strike is commonly recorded in the field as the number of degrees between 0 and 90 west or east of north (for example, N50°W). Dip is recorded as the number of degrees between 0 and 90 below the horizontal in a vertical section perpendicular to the strike of the plane. Numerical methods for analyzing orientation and frequency data require that the usual geologic description be converted to a system in which the strike is recorded unambiguously by the use of a single number between 0 and 180 or 360 degrees, rotated clockwise or counterclockwise from north or south depending on the convention used and the dip by a single number between 0 and 90 or 180 degrees rotated according to a convention. Numerical conversion of strike and dip data from the field has been used by Hendron, Cording, and Aiyer (1980) for vector analysis of stability of slopes cut by discontinuities. Numerical conversion has also been used to permit computer analysis and processing of borehole photography data on discontinuities, especially in inclined borings for which the recorded apparent orientation data must be converted to true orientation.*

35. Data analysis. Discontinuity orientation data are commonly analyzed by plotting the orientations on the two-dimensional projection of a reference sphere by the technique of stereographic projection. Figure 8 shows a plane represented on the reference sphere projection, or stereonet, by a single point, the pole, which represents the intersection with the lower hemisphere of a line normal to the plane and passing through the center (0) of the sphere. The pole is unique to the plane of that orientation and all discontinuity planes recorded can be represented on a single stereonet. The stereonets of discontinuities associated with slope failures on the right side of Figure 7 illustrate the clustering or grouping of poles. The preferred orientations of groups or sets of planes can be readily seen by contouring the clusters. Stereonets are constructed by equal-area projection (a Lambert or Schmidt net) or by equal-angle projection (a Wulff net), and may be constructed to equatorial or polar projections. The equal-area projection, or Schmidt net, retains correct areal distribution of projected poles and is used for statistical analysis and contouring of groups of discontinuities. The equal-angle, or Wulff net, is most commonly used for analysis and graphical solutions of structural problems due to the ease of plotting the traces of planes in the

* For example, see Banks (1977), op. cit.

projection. Discontinuity orientation can be simply displayed on a joint rosette, which shows the number of joints (discontinuities) occurring in each sector of a 360° compass face. Discussions of the construction and use of stereographic projections can be found in Billings (1954); Ragan (1973); Hoek and Bray (1974); Coates (1970); Goodman (1976); and Hendron, Cording, and Aiyer (1980). John (1968) explains in detail the use of stereographic projection of discontinuity data in stability analyses of slopes in jointed rock, and includes the application of factors of safety and active and passive forces. Another useful discussion can be found in Attewell and Farmer (1976), but it should be recognized that the upper hemisphere projection is used. Figure 9 illustrates the simplified analysis of the stability of a wedge formed by the intersection of two planar discontinuities with a slope (example of Coates (1970)). In the example, two joint planes, one striking N10°E and dipping 60°NW (Plane 1 in Figure 9) and the other striking N30°E dipping 40°SE (Plane 2), intersect to form a potential wedge in a slope face striking N75°E and dipping 35°SE. The joint poles are also shown. The intersection of the joint planes, line OJ (Figure 9), plunges 11 deg and extends beyond the trace of the slope plane on the stereonet. Therefore, sliding of the wedge is possible. If Point J were inside the slope projection, the intersection would have a steeper plunge than the slope and sliding of the wedge could not occur.

36. The attitude (orientation) of the joint planes (Figure 9) could have been recorded numerically as, for example, with joint plane number 1, 190°, 60° if a convention were used whereby strike is recorded as an azimuth clockwise from north, dip direction is understood to be strike plus 90°, and dip is between 0° and 180°. Or the attitude could be recorded as 280°, 60°, whereby 280° is the dip direction (in lieu of "strike plus 90°") using azimuth.

37. Recommended descriptors for orientation. Other conventions are sometimes used, however, but until a standard convention is established for recording attitudes numerically for automatic data processing and analysis, the practice of recording strike, dip, and direction of dip should be continued. The geologist should be aware, however, that specific projects may require the recording of discontinuity orientations in numerical (azimuthal) notation. Pole diagrams (stereonet plotting) of discontinuity distribution and frequency are an efficient and well known method of displaying absolute

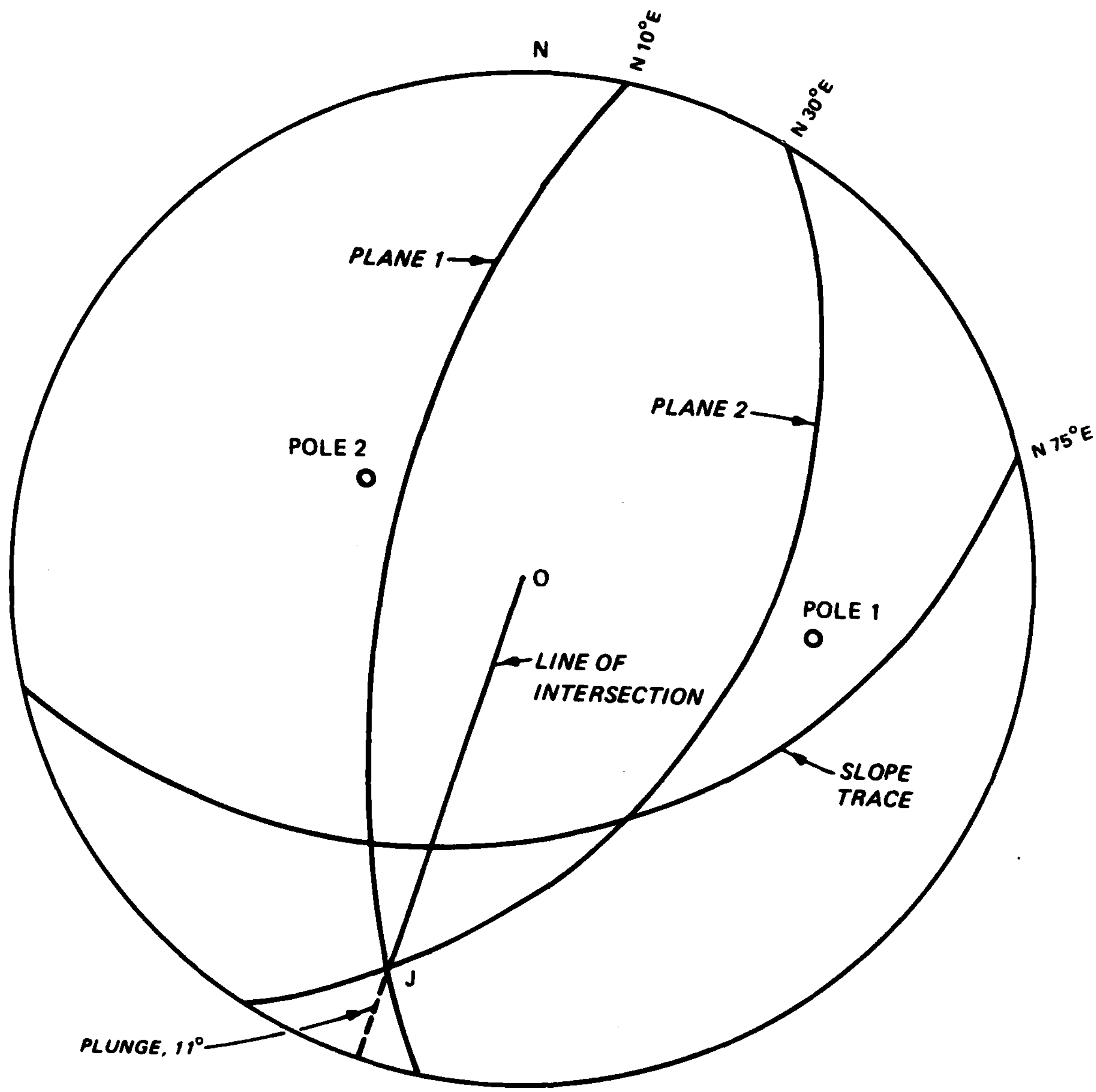

Figure 9. Stereonet analysis of joint wedge/slope stability problem (after Coates 1970)

orientation and orientation relative to surface and subsurface excavations, and their use in geotechnical reports is encouraged. Field methodology for determining the orientation of discontinuities in rock slopes is presented in ETL 1110-2-300 (OCE 1983).

PART III: ROCK WEATHERING

Definition

38. Weathering is the disintegration and decomposition of rock in place by mechanical and chemical processes. Rock is attacked by weathering agents on exposed surfaces, such as excavation walls and natural outcrops and along joints and other discontinuities that extend into the rock mass. The zone of weathering is most pronounced near the discontinuities and exposed surfaces. The degree of weathering in a rock mass depends on (a) the area of exposed surface, (b) the age of the exposed surface, (c) the extent of access to the rock mass along discontinuities and through pores, (d) the chemical composition (mineral content) and texture of the rock, (e) the environment or climate, and (f) the position and chemistry of the ground water. A useful discussion of the relative susceptibilities of the common rocks and minerals to breakdown by weathering is presented by Dornbusch (1982).

39. Mechanical, or physical, weathering of rock occurs primarily by (a) freeze expansion (or frost wedging) of water that seeps into pores and open discontinuities, particularly in temperate climates; (b) thermal expansion and contraction from severe daily temperature variations, especially in arid regions; and (c) cycles of wetting-drying, particularly in the clay-rich rocks. Chemical weathering occurs by the reaction of water, acids and bases, oxygen, and carbon dioxide with mineral constituents of the rock. Iron sulfides combine with oxygen to form the commonly occurring red oxides of iron by the process of oxidation. Carbon dioxide dissolved in water readily dissolves soluble carbonates such as limestones and dolomites to produce the networks of caves and solution-enlarged discontinuities of karst regions. Many clay minerals, which are significant in stability of geotechnical structures, are formed from silicates of the igneous rocks by the addition of water (hydrolysis) under certain conditions to form hydrous compounds. For example, feldspars, common igneous minerals, alter in the presence of water to illite or kaolinite, common clay minerals. The absorption of free water into the mineral structure (hydration) also produces a kind of mechanical weathering by expansion of the structure when a mineral undergoes growth by recrystallization. For example, the hydration of anhydrite to reform gypsum produces a volume change of as much as 30 to 60 percent (Robinson 1982). Clay minerals such as

montmorillonite filling the space between discontinuity walls may absorb water and contribute to expansion and mechanical breaking of the rock mass.

Previous and Existing Classifications

40. The weathering of rock is recognized by a decrease in the luster of the rock's minerals, discoloration of the rock, separation of rock crystals or grains along their boundaries, increased friability, and a general decrease in competency or compressive strength. Infiltrating water may stain discontinuity surfaces or bring in material to fill open discontinuities. The degree of weathering present in a rock mass can be classified on the basis of simple qualitative visual and physical inspection. Saunders and Fookes (1970) reviewed weathering processes and earlier (pre-1970) classification schemes of several workers. The system widely published more recently is based on work by the Task Committee for Foundation Design Manual of the Committee on Shallow Foundations of the Soil Mechanics and Foundations Division of the American Society of Civil Engineers (ASCE) (1972). Table 14 is the classification used in Bieniawski (1979) and is an abbreviated version of the ASCE Task Committee (1972) similar to the versions of the Geological Society Engineering Group Working Party (1977) and AEG (1978). It describes weathered rock on the basis of appearance and feel. The terminology for degree of decomposition (weathering) used by Little (1969) in his scheme (Saunders and Fookes 1970) is similar to that of Bieniawski (1979), but Little based his field recognition on relative strength of hand samples and the degree of difficulty in excavation. Little also evaluated the weathering classes by their effects on engineering works.

Recommended Descriptors

41. Although the descriptive terms of Table 14 are subjective in nature, they can be rendered locally objective if the user will observe fresh (unweathered) cores or other samples of the rock mass and use the fresh rock as a standard of comparison for weathered rock. This report recommends the descriptive terminology of Table 14 for field description of the weathering condition of rock samples and rock masses. The field inspector should record the depth of weathering from exposed surfaces where possible and the thickness of the

weathered zone around discontinuities as well as the degree of weathering. The determination of the decrease in strength of the weathered rock should be made using the methods presented in paragraphs 9-14.

PART IV: ROCK QUALITY DESIGNATION

Definition

42. Rock Quality Designation (RQD), or Modified Core Recovery, was developed by Deere, Merritt, and Coon (1969) as a means of describing the condition of the rock mass from core borings. The RQD is obtained by measuring the cumulative (total) length of intact NX core pieces 4 in. long or longer and dividing by the sampling depth (Figure 10). The USACE stipulates that RQD be applied to NX core only. Other investigators (Bieniawski 1979, and Franklin, Broch, and Walton 1971) apply RQD to NX or larger cores. The quotient is expressed as a percentage and is used to classify the rock quality as very poor to excellent (Table 15).

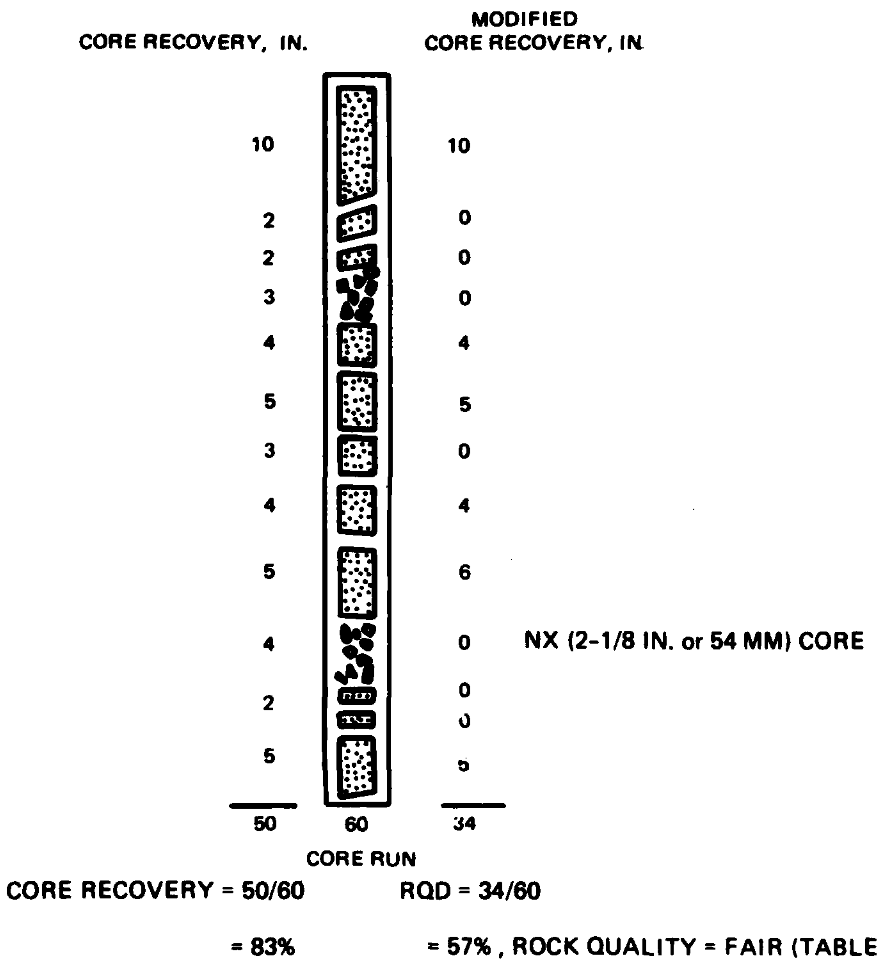

Figure 10. Rock quality determination from modified core recovery (Deere, Merritt, and Coon 1966)

Importance and Previous Use

43. The RQD has been accepted widely as a means of estimating rock quality from cored rock. Bieniawski suggests that the RQD is a quick, inexpensive index for rock core quality but is insufficient to adequately describe the rock mass quality alone because it disregards discontinuity orientation, aperture or tightness, and condition (filling, roughness, etc.). However, Bieniawski uses the RQD as a factor in his Geomechanics rock mass classification. The Geological Society of Great Britain Engineering Group Working Party (1970) elected to retain RQD as a rock quality index. Barton, Lien, and Lunde (1974) incorporate RQD in their Rock Mass Quality rating (Q) of rock mass classification. Most tunnel support design and analysis, for example, require RQD as input, but RQD may or may not be used in foundation or slope stability investigations.

Recommended Use

44. The determination of RQD is somewhat more time consuming than standard core recovery measurements but is a specified procedure for most WACE Districts and Divisions. The RQD as defined by Deere, Merritt, and Coon (1969) (Table 15), or a slightly modified version for metric use in which 0.1-m (10 cm) core pieces instead of 4-in. pieces are counted, should be used when RQD is required. Following current USACE practices, RQD should be applied to NX core only.

PART V: GROUND-WATER CONDITIONS

Definition and Importance

45. The effects of ground water in surface and subsurface excavations are manifested as (a) seepage or inflow through pores and along discontinuities, (b) strength-reducing pore pressure in excavation slopes and along potential planes of weakness, and (c) softening or weakening of saturated rocks and filling materials. Ground-water inflow in underground excavations inhibits excavation activities and may wash out loose, saturated materials in pores and filled discontinuities. Water trapped in the rock mass against impermeable or low permeability barriers can create a buildup in pore pressures which can lead to instability in excavation slopes, tunnel walls, and along potential failure planes in foundations. Pore pressure reduces the effective stress on potential planes of failure and thereby lowers the shear strength of the rock mass. The relationship is shown mathematically in the familiar expression:

$$\tau = c + (\sigma - u) \tan \phi$$

where

τ = the shear strength (or shear stress required to cause sliding along a plane)

c = the cohesion of the rock or soil particles

σ = the normal stress component of load on the plane

u = the pore (uplift) pressure produced by the head of ground water

ϕ = the angle of internal friction along the potential failure plane

The term $(\sigma - u)$ is the effective stress on the plane resulting from the reduction in normal stress by the pore pressure. As stated in Hoek and Bray (1974), ϕ and c are not much affected by the mere presence of water (water content) in hard rock, sands, and gravels. Instead, the shear strength characteristics of those materials are defined more by water pressure, u, than by water content. Slope stability, for example, is influenced more by a small volume of water trapped under high pressure within the rock mass than by a large volume of water discharging freely from the excavation face.

Previous and Existing Classifications

46. Bieniawski (1979) included ground-water condition in his Geomechanics Classification for rock mass rating. Ground-water condition, accounting for as much as 10 percent of the rock mass rating in his system, is assessed by estimating the rate of inflow (discharge) and by a subjective description such as dry, damp, wet, dripping, or flowing. Barton, Lien, and Lunde (1974) used a Joint Water Reduction Factor, J_w, in their Q-System of rock mass rating. J_w is a measure of the water (pore) pressure, which is responsible for reducing the effective stress along planes of potential failure. Barton et al. also recognized the ability of water under pressure to soften and wash out clay-filled discontinuities.

47. Tests on rock by Colback and Wiid (1965) and by Broch (1974) showed a general reduction in compressive strength and point-load index with increasing water contents of the rocks tested. Broch (1979) suggested in further tests that a reduction in ϕ also occurred. The deleterious effects of water content on strength have been shown only in laboratory tests of intact rock specimens. For practical considerations of stability of rock masses the effects of pore pressure and not water content should be given priority. Pore pressures within the rock mass can be monitored by the installation of piezometers if the piezometers are properly located. Nevertheless, the preliminary analysis of seepage and pore pressure conditions benefits from early field recognition and description of water conditions along discontinuities encountered in the exploration phase.

Recommended Descriptors

48. The ISRM (1978) suggests a system of seepage ratings for describing the water conditions of filled and unfilled discontinuities in tunneling or surface exposures (Table 16). The ratings are based on simple observations of the amount of water present and on field estimates of discharge and relative water pressure. This report recommends the use of the field descriptors presented in Table 16 for preliminary assessment of water conditions. The systems of Bieniawski (1979) and Barton, Lien, and Lunde (1974) for rock mass rating are accepted as USACE guidance.

PART VI: ROCK DENSITY

Definitions and Importance

49. The rock density, or unit weight, is commonly used to determine the load that the rock mass exerts on a structure and the stress that exists at a point within the rock mass. Rock density is defined in several ways:

- a. Grain density, the ratio of the weight of dry solids to the volume of solids (converts to specific gravity, Gs , by dividing by the unit weight of water).
- b. Dry density, γ_d , the ratio of the weight of the dry solids to the total specimen volume (includes pores).
- c. Saturated density, γ_{sat} , the ratio of the weight of the saturated specimen to the total specimen volume (pores filled with water).
- d. Bulk, or wet density, γ_{wet} , the ratio of the weight of the specimen at its natural or sampled water content to the total specimen volume (also wet unit weight).

Wet density is the quantity most often used to estimate the rock load and is also the most readily obtained because it requires only the weight and volume of the intact specimen at its natural (as-sampled) water content.

Recommended Use

50. Unit weight is usually determined in the laboratory but it should be calculated in the field on rock cores if a diamond saw to square the core ends and a good scale are available (horizontally bedded sedimentary rocks may not require sawing). The maximum range of variation of unit weight in nature is only a little more than a factor of 2. The Rock Testing Handbook, Method No. 109-80 (WES 1980) describes the procedure for determining the effective (as-received) unit weight of a rock specimen.

PART VII: SUMMARY AND RECOMMENDATIONS

51. The significant rock characteristics discussed and the classification and descriptive systems recommended in this report are summarized below:

Significant Rock Characteristics	Recommended Descriptive System
Rock type	Common geologic rock name with qualifying phrase, if necessary (Table 2 and paragraph 8)
Strength	Unconfined compressive strength from point-load test (Table 4 and paragraph 14)
Discontinuity spacing	Table 8 and paragraph 20
Discontinuity aperture	Use "tight" or "open." Include actual aperture measurements (paragraph 22)
Discontinuity filling	Minimum and maximum filling thickness; mineralogy of filling; grain size (paragraph 28)
Discontinuity roughness	Table 12 and paragraph 31
Discontinuity orientation	Strike, dip, direction of dip; use of pole diagrams (stereographic projection) is encouraged; (Figures 9 and 10 and paragraph 37)
Weathering	Table 14 and paragraph 41
RQD	Figure 10, Table 15, and paragraph 44
Ground-water conditions	Table 16 and paragraph 48
Rock density	Calculate wet unit weight (paragraph 50)

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Table 1

Working List of Properties for Rock Mass Classification

Engineering Application	Rock Type	RQD	Discontinuities: Condi-tion*	Orien-tation	Spacing	Weather-ing	Compres-sive Strength	Abrasion Resist/ Hardness	Density or Unit Wt	Joint/ Pore Water	Slake Dura-bility	Permea-bility	Fric-tion Angle ϕ	V Ratio**	Seis-mic Data	State of Stress
Slope Stability		△	△	△	△				△	△	△	△	△			
Rippability	△	△			△	△		△	△						△	
Drillability	△					△	△	△	△							
Tunnel Boring	△	△			△	△	△	△		△						
Tunnel Support		△	△	△	△	△	△		△	△				△		△
Foundations	△	△	△	△	△		△			△			△			
Subsurface Blasting	△	△	△	△	△	△	△		△	△						
Surface Blasting	△	△	△	△	△	△	△		△	△						
Quarry Excavation	△	△			△	△	△		△							
Dynamic Re-sponse (Quake, Blast Resist.)	△			△	△				△						△	△

NOTE: Properties or conditions used in rock mass classification systems. △ indicates consideration in given application.

* Roughness, aperture, filling.

** $\frac{V_{field}}{V_{lab}}$ = velocity index of Deere (1969).

Table 2

A Classification of Common Rock Types

Common Igneous Rocks

	Light-Colored (Acidic)		Intermediate		Dark-Colored (Basic)	Ultra-Basic (Composed wholly of dark minerals, often only one mineral)
	Qtz Rich	Qtz Deficient	Qtz Rich	Qtz Deficient		
	——GRADATIONAL CHANGE DEPENDING ON KIND AND AMOUNT OF FELDSPAR PRESENT——→					
Coarse Texture (Plutonic or intrusive, phaneritic)	Granite	Syenite	Qtz Monzonite to Qtz Diorite (Tonalite)	Monzonite to Diorite	Gabbro	Peridotite Pyroxenite Dunite Others
Contrasting Texture (Porphyritic; coarse crystals in fine matrix)	Granite Porphyry			Monzonite Porphyry		
	Rhyolite Porphyry			Latite Porphyry	"Diabase" or "Dolerite"	
Fine Texture or Glassy (Volcanic or extrusive; lava formers; aphanitic)	Rhyolite (Obsidian is a glassy form of Rhyolite)	Trachyte	Qtz Latite to Dacite	Latite to Andesite	Basalt[(1)]	

Pegmatites are igneous rocks with large crystals. Pegmatites are usually granitic and tabular.
(1) Dark fine grained igneous rocks are often called "traprock"

Common Sedimentary Rocks

Clastic Rocks (Mechanical)				Non-Clastic (Chemical)	
Coarse Grained		Fine Grained		Organic	Inorganic
(>1/16 or 0.06mm, Wentworth >0.075mm, USCS, Soils)		(<1/256 or 0.0004mm, Wentworth <0.075mm, USCS, Soils)			
Conglomerate	(Rudites)	Shales	(Argillites or Lutites)	Most Limestone($CaCO_3$)	Gypsum
Breccia	(Rudites)	Claystones	(Argillites or Lutites)	Coal(Lignite, Bituminous)	Anhydrite
Sandstones	(Arenites)	Mudstones	(Argillites or Lutites)	Dolomite($Ca\text{-}MgCO_3$)	Rock Salt
Siltstone (1/16 to 1/256mm, Wentworth)					
Volcanic Breccia (volcani-clastic)		Tuff (volcani-clastic)		Diatomaceous Earth (Silicate)	Some Limestones
Glacial Till (Tillite)					Chert

MARLSTONE
($35\text{-}65\%$ $CaCO_3$,
$65\text{-}35\%$ Clay)

Common Metamorphic Rocks

Isotropic (Non-Foliated, Massive)	Anisotropic (Foliated or Banded)
Quartzite	Schist
Marble	Gneiss
Hornfels	Slate
	Phyllite

Table 3

Rock Hardness Classification of the Core Logging Committee, South Africa Section (from Association of Engineering Geologists 1978)

Classification	Field test	Range of minimum uniaxial compressive strength (MPa)
Very soft rock	Can be peeled with a knife, material crumbles under firm blows with the sharp end of a geological pick	1 to 3 (145 to 435 psi)
Soft rock	Can just be scraped with a knife, indentations of 2 to 4 mm with firm blows of the pick point	3 to 10 (435 to 1450 psi)
Medium hard rock	Cannot be scraped or peeled with a knife, hand-held specimen breaks with firm blows of the pick	10 to 25 (1450 to 3625 psi)
Hard rock	Point load tests must be carried out in order to distinguish between these classifications. These results may be verified by uniaxial compressive strength tests on selected samples	25 to 70 (3625 to 10,150 psi)
Very hard rock		70 to 200 (10,150 to 29,000 psi)
Extremely hard rock		200 (29,000 psi)

Table 4

Rock Compressive Strength Classifications (from Deere and Miller (1966) and Bieniawski (1979)

Description	Uniaxial Compressive Strength: Bieniawski (1979)*	Uniaxial Compressive Strength: Deere and Miller (1966)
Very high strength	>30,000 psi (>200 MPa)	>32,000 psi
High strength	15,000-30,000 psi (100-200 MPa)	16,000-32,000 psi
Medium strength	7,500-15,000 psi (50-100 MPa)	8,000-16,000 psi
Low strength	3,500-7,500 psi (25-50 MPa)	4,000-8,000 psi
Very low strength	150-3,400 psi (1-25 MPa)	<4,000 psi

* Bieniawski gives the metric (SI) measurements listed here in parentheses.

Table 5

Rock Strength Classification (from Coates 1970)

Description	Uniaxial Compressive Strength, psi
Very weak	<5,000
Weak	5,000-10,000
Strong	10,000-25,000
Very strong	>25,000

Table 6

Stratification Thickness Classification (from McKee and Weir 1953)

Thickness Term	Thickness	Splitting Property Terms
Thinly laminated (lamination)	2 mm	Papery
Laminated (lamination)	<2 mm - 1 cm	Platy or shaly
Very thin-bedded (bed)	1 cm - 5 cm	Flaggy
Thin-bedded (bed)	5 cm - 60 cm	Slabby
Thick-bedded (bed)	60 cm - 120 cm	Blocky
Very thick-bedded	>120 cm	Massive

Table 7

Discontinuity Spacing Scheme (from Deere 1964)

Joint Spacing Term	Spacing/Thickness	Bed Thickness Term
Very close	<2 in.	Very thin
Close	2 in. - 1 ft	Thin
Moderately close	1 ft - 3 ft	Medium
Wide	3 ft - 10 ft	Thick
Very wide	>10 ft	Very thick

Table 8

Discontinuity Spacing Recommended for USACE Use (modified from Deere 1964)

Joints	Spacing/Thickness	Bedding
	1/2 in.	Laminated
Very close	1/2 in. - 2 in.	Very thin
Close	2 in. - 1 ft	Thin
Moderately close	1 ft - 3 ft	Medium
Wide	3 ft - 10 ft	Thick
Very wide	>10 ft	Very thick

Table 9

Some Proposed Discontinuity Aperture Classifications

Geological Society (Great Britain) Engineering Group Working Party (1977)		Core Logging Committee South Africa Section, AEG (1978)		Bieniawski (1979)	
Description	Aperture mm	Description	Aperture mm	Description	Aperture mm
Wide	>200				
Moderately Wide	60-200				
Moderately Narrow	20-60				
Narrow	6-20	Very Wide	5-25+	Very Wide	10-25
Very Narrow	2-6	Wide	1-5	Open	2.5-10
Extremely Narrow	>0-2	Narrow	0.1-1	Moderately Open	0.5-2.5
Tight	0	Very Narrow	0-0.1	Tight	0.1-0.5
		Closed	0	Very Tight	<0.1

Table 10

Materials Filling Discontinuities and Associated Problems

(modified from Brekke and Howard 1972)

Material Filling Discontinuity	Potential Problems
Swelling clay (montmorillonite, illite, attapulgite)	Subject to volume change in varying moisture conditions. May produce swelling conditions when confined. May cause lifting of excavation surfaces and foundations
Inactive clay	Represents weak material between discontinuity walls, with low shear resistance if thick enough. Can be washed out, resulting in open discontinuity
Low-friction metamorphic minerals (chlorite, talc, graphite, serpentine)	Low resistance to sliding, especially when wet
Crushed rock fragments or breccia; sandlike gouge	May ravel or run out of exposed discontinuity. Permeability may be high
Calcite, gypsum	Soluble, may later produce larger apertures than initially measured. May be relatively weaker than wall rock

Table 11

Discontinuity Roughness Classification Schemes

Bieniawski (1979)	AEG (1978)	Barton, Lien, and Lunde (1974)	Geol. Soc. Working Group (1977)
Very rough	Very rough	Rough or irregular, undulating	Very rough
Rough	Rough	Smooth, undulating	Small steps
Slightly rough	Medium rough	Slickensided, undulating	Defined ridges
Smooth	Slightly rough	Rough or irregular, planar	Rough
Slickensided	Smooth	Smooth, planar	Smooth
		Slickensided, planar	Slickensided
			Polished

Table 12

Bieniawski's Discontinuity Roughness Classification

(from Bieniawski 1979)

Description	Definition
Very rough	Near vertical steps and ridges occur on the discontinuity surface
Rough	Some ridge and side-angle steps are evident; asperities clearly visible; discontinuity surface feels very abrasive
Slightly rough	Asperities are distinguishable and can be felt
Smooth	Surface appears smooth, feels smooth
Slickensided	Visual evidence of polishing

Table 13

Effect of Joint Strike and Dip Orientations in Tunneling

(from Bieniawski 1979)

Strike Perpendicular to Tunnel Axis				Strike Parallel to Tunnel Axis		Dip 0^{o}-20^{o} Irrespective of Strike
Drive with Dip		Drive Against Dip				
Dip 45^{o}-90^{o}	Dip 20^{o}-45^{o}	Dip 45^{o}-90^{o}	Dip 20^{o}-45^{o}	Dip 45^{o}-90^{o}	Dip 20^{o}-45^{o}	
Very favorable	Favorable	Fair	Unfavorable	Very unfavorable	Fair	Unfavorable

Table 14

Classification of Degree of Weathering of Rocks

(after Bieniawski 1979)

Unweathered:	No visible signs of weathering; rock fresh; crystals bright.
Slightly weathered rock:	Discontinuities are stained or discolored and may contain a thin filling of altered material. The discoloration may extend into the rock from the discontinuity surfaces to a distance of up to 20 percent of the discontinuity spacing.
Moderately weathered rock:	Slight discoloration extends from discontinuity planes for greater than 20 percent of the discontinuity spacing. Discontinuities may contain filling of altered material. Partial opening of grain boundaries may be observed.
Highly weathered rock:	Discoloration extends throughout the rock and the rock material is friable. The original texture of the rock generally has been preserved, but separation of the grains or crystals has occurred.
Completely weathered rock:	The rock is totally discolored and decomposed and friable. The external appearance of the rock sample is that of soil. Internally, the rock structure is partially preserved but grains and crystals have completely separated.

Table 15

Rock Quality Designation (RQD) as an Index of Rock Quality (from Deere, Merritt, and Coon 1969)

RQD, percent	Description of Rock Quality
0-25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

Table 16

Field Observations of Seepage Conditions for Filled and Unfilled Discontinuities

(after International Society for Rock Mechanics 1978)

Seepage Rating	Description, Unfilled Discontinuities	Description, Filled Discontinuities
I	Discontinuity very tight and dry, water flow along it does not appear possible	Filling materials heavily consolidated* and dry; significant flow appears unlikely due to very low permeability
II	Discontinuity is dry with no evidence of water flow	Filling materials damp, but no free water present
III	Discontinuity is dry but shows evidence of water flow, e.g., staining	Filling materials are wet, occasional drops of water
IV	Discontinuity is damp but no free water is present	Filling materials show signs of washout; continuous flow of water (estimate discharge)
V	Discontinuity shows seepage; occasional drops of water, but no continuous flow	The filling materials are washed out locally; considerable water flow along washout channels (estimate discharge and describe pressure, i.e., low, medium, high)
VI	Discontinuity shows continuous flow of water (estimate discharge and describe pressure, i.e., low, medium, high)	Filling materials washed out completely; high water pressures (estimate discharge and describe pressure, i.e., low, medium, high)

* Presumably, "consolidated" implies that low void ratio has been achieved.

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