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16. Abstract

The U.S. Geological Survey (USGS), under subcontract to the National Bureau of Standards (NBS) on FHWA contract no. FHWA-7-3-0001, performed the geologic tasks required by the contract. The portion of the project reported in this interim report is part of Phase I requiring documentation of features and conditions which influence stability of natural and manmade slopes in earth materials. The features and conditions described include discrete primary and secondary features or discontinuities such as bedding surfaces, joints, and foliations as well as less distinct anisotropies in an otherwise physically uniform mass. Discussion of secondary factors contributing to slope instability such as rainfall, slope steepness and aspect, and vegetation also is included. Triggering by earthquakes has not been included, nor have mud and debris flows and soil creep unless they have been inseparably grouped by authors with other types of mass movement. Also, rockfalls, rock glaciers, and topples were not investigated. All other mass movement of soil and rock such as earthflows, slumps, and rock or block slide failures are considered to be varieties of landslides and are included in this report. The literature on interaction of landslide-causing factors was reviewed and is summarized.

17. Key Words
Slope stability, discontinuities, rock classification, material, moisture, slope, vegetation

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Preface

This literature-derived compilation of factors that influence the stability of slopes is part of a 2-year investigation funded by the U.S. Department of Transportation, Federal Highway Administration, under Federal Highway Administration Order No. 7-3-0001. The contract, "High resolution sensing techniques for slope stability studies," was awarded to the Electromagnetics Division of the National Bureau of Standards for initiation on October 1, 1976. The manuscript was completed December 1977 and only literature available before that date has been included.

The U.S. Geological Survey is a subcontractor to the National Bureau of Standards for the performance of geological tasks relating to the investigation. The portion of the project reported herein is part of the Phase I obligation of the U.S. Geological Survey requiring documentation of features and conditions which influence stability of natural and manmade slopes in earth materials.

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Introduction

The purpose of this study is to document from the literature those features and conditions which may influence the stability of natural and manmade slopes in rock and soil masses. The report has provided input into the evaluation by the National Bureau of Standards of geophysical methods suited to the high-resolution surface detection of subsurface features and conditions that contribute to failure of slopes. Contract time constraints permitted only the English-language literature to be searched in the preparation of this report. It is recognized that a considerable body of literature has not been examined. However, the volume of literature represented by this report would indicate that no serious major exclusions of fundamental concepts or features are present.

The features and conditions described include discrete primary and secondary features such as bedding surfaces, joints, and foliations which affect rock mass stability, as well as less distinct anisotropies, or disruptions, in an otherwise physically uniform mass, such as variations in material type and moisture content which influence the stability of rock and soil masses. Secondary factors such as rainfall, slope steepness and aspect, and vegetation which in turn influence equilibrium are included. In general, short-duration landslide triggering mechanisms such as earthquakes have not been included. An exception is the effect of stresses transferred to slope materials by wind action on trees. This wind-loading factor is included as it is directly related to the presence or absence of vegetation on a slope.

A few restrictions have been placed on the types of mass movement considered because of the nature of the study. Mud and debris flows and soil creep are not included except where inseparably grouped by authors with other types of mass movement. Rockfalls, rock glaciers, and topples have not been investigated. All other mass movements of soil and/or rock such as earthflows, slumps, and rock or block slide failures are considered to be varieties of landslides and are included in this report.

At the outset certain other definitions are pertinent to the report. Of primary importance is the term "discontinuity." It denotes a distinct break in the physical continuity of earth materials and includes bedding surfaces, joints, and foliations. If separated, the intervening space may be open or filled. Projections from the surfaces are called "asperities" and they contribute to the roughness of a given surface. "Soil" is used almost exclusively in the geotechnical sense to define residual or transported earth materials overlying bedrock which are a product of mechanical disintegration and chemical decomposition of preexisting rock. The surface soils of the agronomist are thereby included. Certain rocks such as some shales which are rock-like from compaction of fine-grained sediments may be treated as soil or soil-like materials in the report.

The report is divided into two main parts. The first and larger part is concerned with discontinuities. The second part deals with the influence of material type, moisture content, slope, vegetation, and combinations of these on landsliding.

An overview of discontinuities in earth materials

General

Knowledge of the types, occurrence, and features of discontinuities or surfaces of separation (Deere, 1964), is of major geotechnical significance in the study of stability of soil and rock slopes. The influence of discontinuities on the stability of soils or of materials geologically classed as rock which have soil-like properties (clay shale) has been observed by Terzaghi (1936), Skempton (1964), Anderson and Schuster (1970), and Fleming, Spencer, and Banks (1970). Terzaghi (1962), Voight (1968), John (1970a), Piteau (1970, 1971), and Cruden (1975) are among the many investigators who have noted that the stability of a rock mass is more a function of the presence and nature of discontinuities within the mass than of the strength of the rock unaffected by discontinuities. Failures in soil are not so dependent upon the presence of discontinuities because the soil mass is intrinsically weak, whereas in rock, the opposite is true (Patton and Deere, 1971; Piteau, 1971). In the pages that follow, discontinuities, for the most part, will be discussed independently of whether they occur in soil or rock masses.

Most discussions of the influence of discontinuities on slope stability typically restrict listings of kinds of discontinuities to those occurring in rock masses for the reason given above. An exception involves fissures, which in the geotechnical literature are small discontinuities common to overconsolidated clays and clay shales. Although the bulk of the literature is involved with "rock formations" from the geologist's standpoint, these materials are soil-like in their properties and the principles of soil mechanics rather than rock mechanics are applied. The discontinuities of geotechnical importance are relatively planar in form, a feature which contributes to the structural weakness of a discontinuous rock mass.

The most comprehensive lists of discontinuities which influence slope stability are those of Deere (1964), Hagerman (1966), Cruden (1975), Kulhawy (1975), and Goodman (1976). Other less comprehensive lists have been prepared by Jennings and Robertson (1969), Fleming, Spencer, and Banks (1970), Krohn and Slosson (1976), and Kohlbeck and Scheidegger (1977). The dates of these papers indicate the relatively recent recognition of discontinuities as significant factors in the stability of rock slopes.

Kinds of discontinuities

Of the discontinuities associated with slope failures, joints and bedding were noted most often in the literature reviewed for this study. Joints occur as fractures in all rock types from a variety of causes, and ideally show no visible evidence of displacement (Price, 1959; Gary and others, 1972). Fissures as used in the geotechnical literature are similar to joints but are much smaller in scale (Attewell and Farmer, 1976) and typically are slickensided. Bedding is a nearly universal discontinuity in sedimentary rocks which visibly separates two adjacent beds or layers. The terms "bedding plane" and "bedding surface" are synonymous with bedding and are used by some to differentiate between planar and irregular bedding (Gary and others, 1972; Pettijohn, 1975).

Metamorphic rocks commonly contain discontinuities which are products of regional metamorphic stresses. As used in geotechnical literature these are cleavage, schistosity, and foliation (or banding or gneissosity), listed in order of decreasing ease of splitting. Cleavage is the regular planar discontinuous structure common to slate which results from reorientation of small platy minerals in very fine and fine-grained sedimentary rocks which have been weakly metamorphosed. Schistosity is the relatively planar structure similar to cleavage which is caused by the parallel alinement of platy and prismatic minerals such as mica formed during metamorphism. Foliation is the term often used in a restricted sense in the geotechnical literature for the development of segregated bands of coarser, more equant minerals, as in gneiss, which result from metamorphism of a higher degree than that which formed schistosity. Information relating to broader definitions and uses of these terms in the geologic literature may be found in Verhoogen and others (1970) and Gary, McAfee, and Wolf (1972). Selected comparisons which may cause problems are discussed in the next section.

Faults and shear zones are discontinuities that have resulted from fracturing and measurable displacement along the fracture or fractures. Displacement may range from a few centimeters to thousands of meters. Faults may occur singly or as groups of faults. Differing degrees of polishing and striation called slickensides, angular crushed rock called breccia and mylonite, and clay-like, pulverized rock called gouge will occur along faults and within zones of faulted material. Where faulting has been closely spaced and parallel, a zone of crushed and brecciated rock may be formed which is referred to as a shear zone. The crushed zone of Kulhawy (1975) probably is a shear zone. Additional information about faults and fault-related features may be found in introductory texts on structural geology (Billings, 1972; Spencer, 1977) and Krynine and Judd's geotechnical text (1957).

Other discontinuities of lesser importance noted in the geotechnical literature often with restricted meanings include contacts (Jennings and Robertson, 1969; Cruden, 1975; Goodman, 1976), unconformities and veins (Cruden, 1975), and seams (Kulhawy, 1975). Contacts are discontinuities that normally occur at the boundaries of igneous rocks with adjacent rocks. Unconformities are surfaces on which sedimentary rocks overlie igneous, metamorphic, or other sedimentary rocks as a result of lengthy erosional or nondepositional periods. Veins are fractures filled with intruded material, usually of igneous origin. In the geotechnical literature, seams usually refer to joints that have been filled with other material such as clay (Goodman, 1976). They may also represent weak clay zones in a sedimentary sequence or minor faults or altered zones along other discontinuities (Brekke and Howard, 1972).

A discontinuity which seldom appears in lists of discontinuities is a preexisting failure surface which may be present in soil or rock masses. The reasoning probably is that naturally occurring discontinuities in unfailed materials are of primary importance in evaluating slope conditions. The geotechnical literature contains numerous examples, however, of the important part that old failure surfaces play in the reactivation of slope movement. Gray, Ferguson, and Hamel (1974) have included old sliding surfaces with the other discontinuities that control the engineering behavior of sedimentary rocks. They also considered an old failure surface to be the only discontinuity affecting soil masses. Thus, they should not be neglected and can be the most significant factor when itemizing discontinuities that are conducive to slope failure.

The various discontinuities noted above are named for the most part on the basis of genesis. A genetic classification can create problems as incorrect naming may imply other features or details that may not be present. In addition, it may not be possible to identify the genesis of a particular discontinuity making naming difficult. Deere (1964) foresaw this dilemma and suggested the use of the term "fracture" in such an event.

Features associated with discontinuities

In order to quantify the characteristic features of discontinuities in a way not possible with a more descriptive, genetic terminology, modern workers make a number of measurements or estimates concerning discontinuities. These are of great importance in the quantitative analysis of slope stability.

The most important measurable features associated with discontinuities from the geotechnical standpoint are orientation, spacing, continuity, surface roughness, and coating or filling material (fig. 1). Although each will be treated in greater detail later, general information is given here for the reader to note their value and need for measurement.

Orientation (or attitude) refers to the positioning of discontinuities in three-dimensional space. This is of special importance as discontinuities inclined in the same direction as a slope or cut are normally more susceptible to failure than those inclined into a slope or cut. In addition, discontinuities which are inclined at such an angle that they intercept (daylight) a slope or cut are more unstable than those that do not. Orientation typically involves measurement of the dip and strike of the various discontinuities encountered. The need for such orientation measurements is noted by Jennings and Robertson (1969), Franklin, Broch, and Walton (1971), Müller and Hofmann (1971), Robertson (1971), Goodman (1972, 1976), N. Barton (1973), Bieniawski (1973), Heuzé (1974), and Call, Savely, and Nicholas (1976).

The spacing frequency or intensity of occurrence of discontinuities is a measure of the degree to which a rock mass is subdivided or broken into discrete masses and is an important rock mass strength parameter (John, 1962; Piteau, 1970, 1971, 1972; Franklin and others, 1971; Müller and Hofmann, 1971; Brekke and Howard, 1972; N. Barton, 1973; Bieniawski, 1973; Heuzé, 1974; Call and others, 1976; Goodman, 1976). Franklin, Broch, and Walton considered spacing of discontinuities to be of particular importance as compared to the other joint features.

The continuity of a discontinuity, or the two-dimensional length over which it creates a single structural defect, is closely associated with its orientation and surface roughness. All combine to greatly influence the stability of a rock mass. Piteau (1970, 1971) has considered continuity second only to orientation in importance. Overlap (fig. 1) influences the ease with which failure occurs between adjacent joints and, depending on the spacing, may effectively extend the length of a structural defect. Goodman (1972) used the term "imbrication" for overlap. Others who have stressed the importance of the continuity of discontinuities are Jennings and Robertson (1969), Franklin, Broch, and Walton (1971), Goodman (1972, 1976), Piteau (1972), Bieniawski (1973), Heuzé (1974), and Call, Savely, and Nicholas (1976).

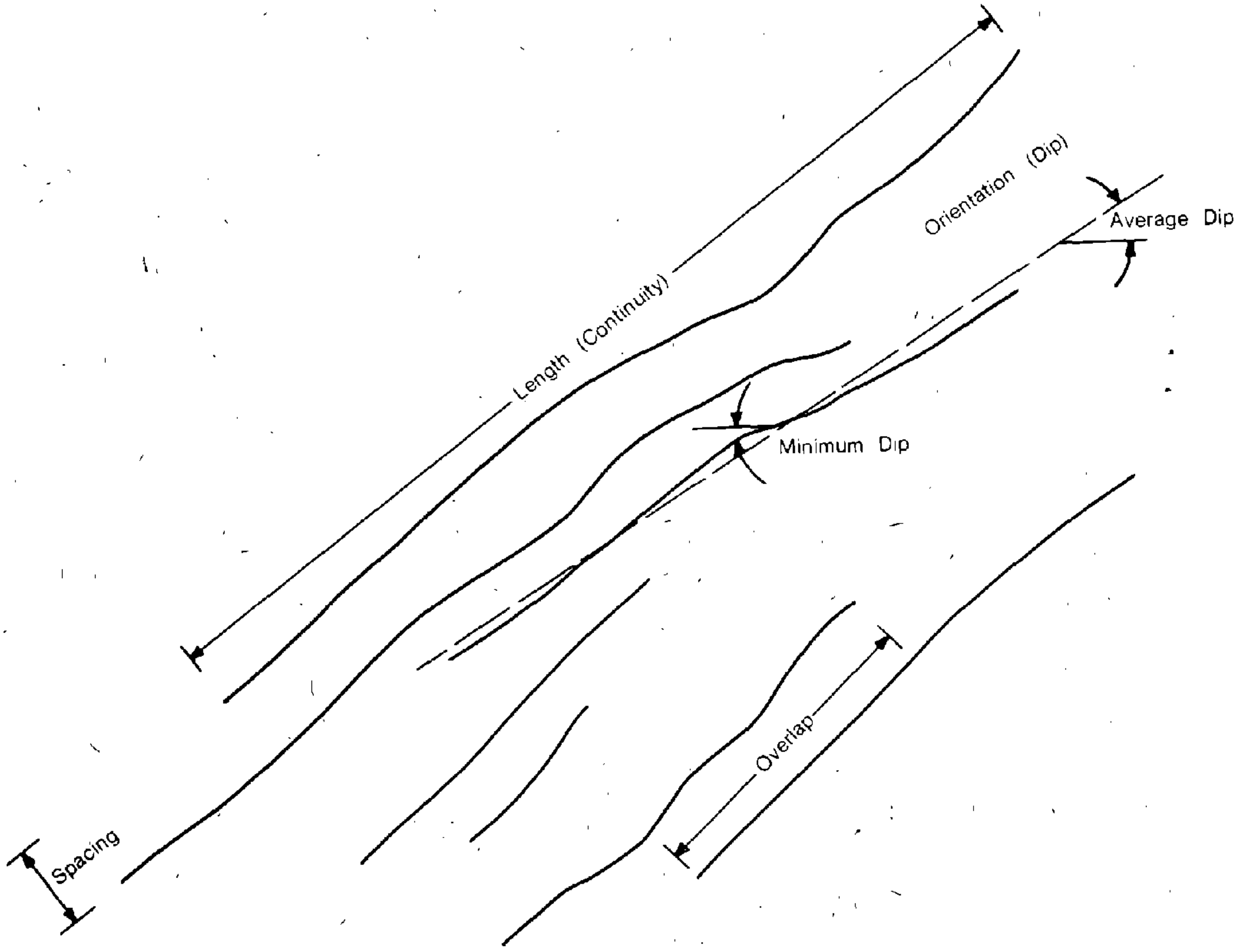

Figure 1. Joint characteristics.
Modified from Call, Savely, and Nicholas, 1976.
Monograph on Rock Mechanics Applications in Mining,
S. J. Green, W. S. Brown, W. A. Hustrulid, eds.,
AIME, New York, 1977.

The roughness or irregularity of a discontinuity, primarily joints and bedding surfaces (Deere, 1964), is of prime importance in determining or estimating the frictional resistance to movement along the discontinuity (Deere; Jennings and Robertson, 1969; Piteau, 1970, 1971, 1972; Franklin and others, 1971; Robertson, 1971; Brekke and Howard, 1972; Goodman, 1972, 1976; N. Barton, 1973; Barton and others, 1974; Call and others, 1976).

The physical properties and amount of material that may coat or fill discontinuities is an important factor in evaluating the strength of a rock mass, as with increasing thickness, a weak filling material gradually eliminates the influence of surface waviness and roughness. The gouge that often occurs associated with faults is normally included when filling material is considered. The following papers provide insight into the interaction of coating and filling materials with the other measurable parameters being considered: Jennings and Robertson (1969), Piteau (1970, 1971, 1972), Franklin, Broch, and Walton (1971), Brekke and Howard (1972), Barton, Lien, and Lunde (1974), Call, Savely, and Nicholas (1976), and Goodman (1976).

The scale at which discontinuities occur has been considered by some investigators. Fleming, Spencer, and Banks (1970) considered fissures, slickensides, and some joints to be small-scale features, and bedding planes, faults, unconformities, and some joints to be large-scale features. They felt that the mass strength of a material is influenced by the small-scale discontinuities whereas the large-scale features exert control over the location of failure surfaces. Earlier, Müller (1964a) had stated that joints with large continuity were necessary for the development of a failure surface in rocks. Fissures (or microfissures) were classified as flaws rather than geniune discontinuities by Brekke and Howard (1972). They classified other discontinuities on the basis of the cross-section size in which a feature would occur; that is, squares approximately 0.6 m, 6 m, 61 m, and 610 m on a side ranging from bedding and foliation to major faults. The scale classification of Heuzé and Goodman (1972) is essentially identical.

While not a discontinuity feature per se, weathering or alteration may reduce the strength and thus the stability of a rock mass as decomposition takes place along a discontinuous surface or as the intact compressive strength is reduced by weathering that pervades the rock mass. Bieniawski (1973) has provided a set of criteria for estimating the degree of weathering that includes both discontinuities and the intact rock mass. Because of its thoroughness it is included here:

> Unweathered: No visible signs of weathering. Rock fresh, crystals bright. Few discontinuities may show slight staining.

> Slightly weathered rock: Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material. Discontinuities are discolored and discoloration can extend into rock up to 10 mm from discontinuity surfaces.
>
> Moderately weathered rock: Slight discoloration extends through the greater part of the rock mass. The rock material is not friable (except in the case of poorly cemented sedimentary rocks). Discontinuities are stained and/or contain a filling comprising altered material.
>
> Highly weathered rock: Weathering extends throughout rock mass and the rock material is partly friable. Rock has no lustre. All material except quartz is discolored. Rock can be excavated with geologist's pick.
>
> Completely weathered rock: Rock is totally discolored and decomposed and in a friable condition with only fragments of the rock texture and structure preserved. The external appearance is that of a soil.

Cording and others (1975) also recognized the importance of weathering or alteration in rock-mass strength and proposed a visual rating system similar to but not as well defined as that by Bieniawski (1973). Their scale ranged from 1 to 5 as follows: (1) no weathering or alteration, (2) slight weathering on joint surfaces, (3) moderate weathering on joint surfaces; (4) slight weathering of mass, and (5) intense weathering of mass.

Others who have introduced weathering as a factor to be evaluated are Wahlstrom (1973) and Barton, Lien, and Lunde (1974). Wahlstrom considered pervasive alteration of a rock mass while Barton, Lien, and Lunde dealt with alteration associated with joint openings and filling material. Bieniawski (1973), Wahlstrom, and Barton, Lien, and Lunde have used their evaluations of weathering in systems of classifying rock-mass strength, which are described later.

N. Barton (1973) noted that the shear strengths of weathered joints are lower than for those in unweathered rock given the same degree of roughness of the joint surfaces. This is due to the reduction in the compressive strength of the weathered rock which affects the shearing strength of the asperities.

Terminology

The naming of the different kinds of discontinuities suffers from a lack of consistency in the geotechnical literature. At the center of the problem is the term "joint." Goodman (1976) has written that "joint" is loosely used in an engineering context for all or part of a family of discontinuities in rock masses.

Many investigators such as Deere (1964), Patton (1966a, b), Fleming, Spencer, and Banks (1970), Cruden (1975), and Kulhawy (1975) used the term as it was defined earlier; that is, a fracture along which there has been no displacement. Others such as Piteau (1970, 1971, 1972), and Patton and Deere (1971) have avoided the problem by considering discontinuities to be planes or zones of weakness or defective planes in rock rather than using the term in a more inclusive sense as noted by Goodman above.

Many have broadened or altered the meaning of the term "joint." McMahon (1968a) considered a joint to be any naturally occurring rock fracture with less than 0.6 cm of clay or other soft coating material. N. Barton (1973) defined a rock joint as a mechanical discontinuity of geologic origin that intersects near-surface rock masses. Bieniawski (1973) included all discontinuities whether joints, faults, bedding planes, or other surfaces of weakness. Most recently Kohlbeck and Scheidegger (1977) have defined a joint to be any crack or fracture in an exposed rock outcrop. Brekke and Howard (1972) used the term "joint" as a scale term regardless of whether there has been movement or not, while noting that many geologists reserve the term for typical extension discontinuities without any perceptible movement.

Attewell and Farmer (1976) and Goodman (1976) have distinguished between discontinuities and fissures on the basis of scale. Attewell and Farmer noted that both geologists and engineers look upon small-scale discontinuities as fissures while engineers depart from restrictive terminology at larger scale and regard all large discontinuities as cracks or joints depending upon whether they are open or closed, respectively. They concluded that "joint" is best used for all large-sized systematic and continuous discontinuities and "fissures" for all small discontinuities. Goodman considered fissures to be rock specimen features whereas joints are larger scale features.

Brekke and Howard (1972) have contributed further to the terminology problem by stating that discontinuities in general are surfaces of rupture resulting from failure from natural forces as well as from blasting and stress relief from excavation. Thus no discontinuous primary sedimentary features such as bedding planes are included. Franklin, Broch, and Walton (1971) grouped joints, faults, and bedding planes as fractures regardless of their origin. By comparison, Hagerman (1966) defined bedding surfaces as "primary joints" and included fractures under "rupture joints."

The geotechnical literature that considers soils or nonindurated (soil-like) rock has similar variations in terminology involving joints and fissures. In Skempton's work (1964) on the London Clay (an overconsolidated "rock" formation of Eocene age), both terms were used with scale as the implicit differentiation. Later, Fookes and Denness (1969) used only the term "fissure" for discontinuities in materials similar to the London Clay. Fleming, Spencer, and Banks (1970) applied all of the commonly used terms such as fissures, joints, bedding planes, faults, etc., to the Bearpaw, Claggett, and Pierre Shales, which are all overconsolidated fissured clay shale formations. Anderson and Schuster (1970) encountered the fissure and joint terminology problem while working with fissured, overconsolidated clay shales interbedded in the Columbia River Basalt Group. They noted that when working with soils or soil-like materials, geologists used "joints" for closed discontinuities with no displacement and "fissures" for open, large discontinuities. Engineers typically used "cracks" for large, open discontinuities and "fissures" for small discontinuities.

The geotechnical terminology of discontinuities in metamorphic rocks is in a similar state of disarray. Geologically, foliation is a general term that applies to the planar arrangement of textural or structural features in any kind of rock (but most commonly metamorphic rocks) that is conducive to breakage along near-parallel planes (Billings, 1972; Gary and others, 1972; Spencer, 1977). Cleavage is the tendency of a rock to split along secondary, closely spaced structures (Gary and others). Cleavage or rock cleavage may be in the form of a fracture cleavage from closely spaced joints or slaty cleavage from the parallel arrangement of platy minerals from low-grade metamorphism (Billings; Spencer). Cleavage, thus, is a variety of foliation in contrast to the usual geotechnical usage referred to earlier. Schistosity is also a variety of foliation due to the parallel, planar arrangement of minerals (Gary and others). It occurs in metamorphic rocks from the process of recrystallization (Billings). It is displayed in schist and gneiss rock types with the latter having more equant and coarser grains with greater mineral segregation or banding as noted in the preceding section (Verhoogan and others, 1970). Over the years the terms "schistosity" and "gneissosity" have been applied by many to the degree of foliation exhibited by schists and gneisses, respectively (Gary and others).

In the geotechnical literature Deere (1964), Kulhawy (1975), and Goodman (1976) have used cleavage, schistosity, and foliation for degrees of foliation ranging from slate to schist to gneiss as noted earlier. Heuzé and Goodman (1972) used just cleavage and foliation in classifying discontinuities. Cruden (1975) did not misuse "foliation" as he used only cleavage, schistosity, and gneissosity, with cleavage used in the general geologic sense; that is, closely spaced, parallel surfaces. Berkman and Ryall (1976) in their "Field Geologists' Manual" have compounded the problem further, especially as it regards geological terminology, by preferring the terms "bedding," "foliation," or "cleavage" in place of stratification, schistosity and gneissosity, or microfissuring. In their system of classifying common defects in rock masses, cleavage is both a feature of layering or fracturing while bedding and foliation are exclusively layering terms. For the individual interpreting the literature, it can be assumed safely that all of the terms refer to the ease with which rocks split. Further refinement must be a function of the context in which the terms are used.

In review, an investigator must be aware of the variations in terminology that exist when involved with discontinuities and their characteristics. When writing or presenting data, the definitions being used should be stated. When using the work of others, the terminology being used must be understood so that incorrect assumptions will not be made concerning the nature of a given discontinuity.

Classifications involving discontinuities

General

With increasing construction in rock in post-World War II years, there also has been an increasing need for measures of rock-mass quality. Stability of rock slopes is a function of rock-mass quality. Discontinuities play the dominant role in the development of such measures or classifications. Some rock-mass quality classifications have been designed specifically for a particular application such as that by Wickham, Tiedemann, and Skinner (1972) for tunnel construction. Other systems such as those of Deere and others (1967) and Barton, Lien, and Lunde (1974) are more universally applicable even though originally designed to describe drill-hole cores and for tunnel design, respectively. Coates (1964), Watkins (1971), and Barton, Lien, and Lunde all noted the need for classification systems that give estimates of rock-mass quality which are independent of the kind and size of rock excavation. The changing involvement of the various discontinuities and their characteristics in geotechnical classification can be seen best with a historical review of some of the more common ones. Classifications were descriptive at the outset and have evolved into more quantitative ones.

Classification schemes

As early as 1946, Terzaghi developed a descriptive classification of rocks for use in estimating tunnel support requirements. The classification is as follows:

> Intact rock--rock with no joints.
>
> Stratified rock--rock with little strength along bedding surfaces.

Moderately jointed rock--rock mass jointed but cemented or strongly interlocked.

Blocky and seamy rock--jointed rock mass without any cementing of joints and weakly interlocked blocks.

Crushed rock--rock that has been reduced to sand-sized particles without any chemical weathering.

Squeezing rock--rock containing a considerable amount of clay.

Swelling rock--rock that squeezes primarily from mineral swelling.

Although qualitative, Terzaghi's classification showed his perception of the various factors that control rock-mass strength.

Another descriptive classification involving joints was that of Hodgson (1961) who among others had noted that freshly exposed joint surfaces often have faint ridges oriented in plumelike or radial patterns. He divided such joints into systematic and nonsystematic joints. The systematic joints occur in sets within which joints of each set would be parallel or subparallel and would bear the surface features noted above. Nonsystematic joints have random, curvilinear traces in plan and section and lack the oriented surface features.

In 1962, John recognized the need for systematizing the spacing of joints in rock. As used, jointing included all of the related foliation features noted in metamorphic rocks. Joint spacings were categorized from "very close" to "occasional" over a spacing range of 0.5-1000 cm as shown in figure 2. With the addition of a four-part weathering classification and associated compressive strengths (fig. 2), this comprised the first quantitative classification scheme of rock-mass strength. John considered joint spacing and weathering to be the most important factors that govern rock-mass strength. The interaction of these factors, shown by the shaded patterns in figure 2, permits visual assessment of rock-mass strength. The jointing and strength scales are logarithmic. This permits a graphical weighting of the factors in proportion to their relative importance.

Deere (1964) modified the joint spacing classification of John to provide more spacing units as shown in table 1. In the same paper Deere proposed descriptive terminology for the thickness of bedding units between visible and prominent bedding planes. It is reproduced as table 2. Also in the same paper, Deere noted the need to measure and record the lengths of pieces of core obtained from core drilling. This culminated later in his Rock Quality Designation (RQD) classification which conceptually has its basis in frequency of discontinuity spacing.

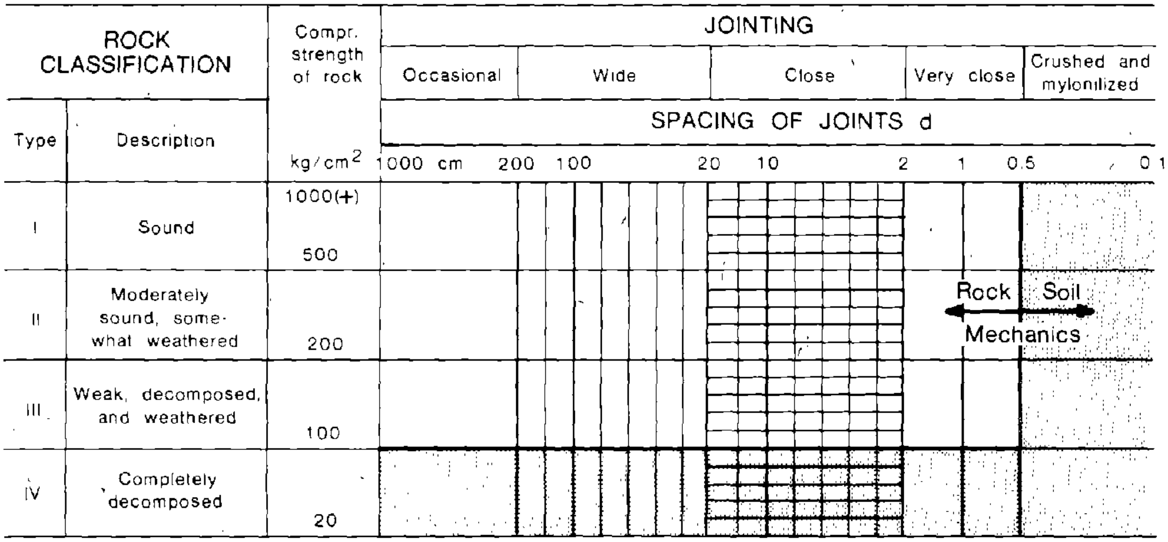

Figure 2. Rock-mass classification of John (1962).
An approach to rock mechanics, K. W. John, p. 11,
Proc. Soil Mech. and Foundation Engr. Div., ASCE, 1962.

Table 1. Descriptive terminology for joint spacing. Deere, 1964. Reproduced with permission from Springer-Verlag.

Descriptive Term	Spacing of Joints (English)	(Metric)
Very close----------------	Less than 2 in.	Less than 5 cm
Close--------------------	2 in. - 1 ft	5 cm - 30 cm
Moderately close---------	1 ft - 30 ft	30 cm - 1 m
Wide---------------------	3 ft - 10 ft	1 m - 3 m
Very wide----------------	Greater than 10 ft	Greater than 3 m

Table 2. Descriptive terminology for thickness of bedding units. Deere, 1964. Reproduced with permission from Springer-Verlag

Descriptive Term	Thickness of Beds (English)	(Metric)
Very thin----------------	Less than 2 in.	Less than 5 cm
Thin---------------------	2 in. - 1 ft	5 cm - 30 cm
Medium-------------------	1 ft - 3 ft	30 cm - 1 m
Thick--------------------	3 ft - 10 ft	1 m - 3 m
Very thick---------------	Greater than 10 ft	Greater than 3 m

In his classification of rocks for rock mechanics, Coates (1964) included a joint spacing classification of rock masses. Under his "continuity of rock substance," he divided a rock into "solid" with joint spacings greater than 1.8 m; "blocky" with joints 7.6 cm to 1.8 m apart, and "broken" for rock broken into fragments less than 7.6 cm.

In 1967, Deere and others wrote that the behavior of rock masses is governed by both the properties of the intact rock mass and by the naturally occurring geological discontinuities in the rock mass. They noted that the importance of each of these factors depends on a ratio between the dimensions of the excavation and the spacings of the discontinuities. The resulting classification, the Rock Quality Designation (RQD) is based on the recovery of rock core over 10.2 cm long. This has been referred to as a modified core recovery procedure. The RQD value is the percentage obtained by dividing the total of all core pieces (≥10.2 cm) by the length cored. All smaller pieces are assumed to be the result of closely spaced discontinuities, shearing, faulting, or weathering, all of which decrease the quality. Descriptive names were given to groupings of modified RQD values; that is, very poor, 0-25; poor, 25-50; fair, 50-75; good, 75-90; and excellent, 90-100. The reader is referred to the 1967 paper for an evaluation of the problems associated with the use of RQD values.

Komarnitskii (1968) devised a genetic classification of zones and planes of weakness as they influence slope stability. The features that occur naturally are classed as lithogenetic, tectonic, and exodynamic. Lithogenetic features are those that result from the formation of sedimentary rocks such as bedding and some joints. The tectonic group includes joints and faults of tectonic origin along with disconformities and unconformities and joints and faults which result from subsidence. His exodynamic class includes features resulting from ancient landslides, filtration, or leaching, karst or sink features, and buried erosion surfaces. The usefulness of such a descriptive classification has not been tested widely in geotechnical applications.

The classification of discontinuity-related features devised by Fookes and Denness (1969) is primarily for fissured overconsolidated clay shales. It utilizes measurements of the area of fissures per unit volume and size of intact blocks of shale. The classifications in tables 3 and 4 are based on attributes that are difficult to measure in the field. Skempton, Schuster, and Petley (1969) have used intensity of fissures as the number of discontinuities per unit volume.

Table 3. Size classification of fissures.
Modified from Fookes and Denness, 1969.
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Type	Size (area)
Very large fissure-----------------	≥100 sq. m
Large fissure---------------------	1 -100 sq. m
Normal fissure--------------------	0.01- 1 sq. m
Small fissure---------------------	1 -100 sq. cm
Very small fissure-----------------	≤1 sq. cm

Table 4. Fissure intensity classification.
Modified from Fookes and Denness, 1969.
Reproduced with permission from Geotechnique.

Intensity type	Area of fissures per unit volume (sq. m/cu. m)	Average size of intact blocks
Very low	≤3	≥1 cu. m
Low	3- 10	0.027-1 cu. m
Moderate	10- 30	0.001-0.027 cu. m
High	30-100	27-1000 cu. cm
Very high	100-300	1-27 cu. cm
Excessive	≥300	≤1 cu. cm

Fookes and Denness (1969) subdivided surface geometry on the basis of the ratio of the length of the discontinuity (L) to the radius of curvature of that discontinuity (R) or L/R, as shown in table 5.

Table 5. Surface geometry classification of fissures.
Modified from Fookes and Denness, 1969.
Reproduced with permission from Geotechnique.

Type	Description L/R range	Occurrence	Size	Orientation	Frequency	Restrictions
Planar	$\leqslant\pi/8$	Ubiquitous	Any	Any	Very common	None
Semi-curved	$\pi/8-\pi/4$	Ubiquitous	Any	Any	Common	None
Curved	$\geqslant\pi/4$	Ubiquitous	Any	Any	Common	None
Hinged	Combination of planar and semi-curved/curved	Ubiquitous	Any	Any	Rare	None
Semi-undulose	Combination of two or more alternately convex and concave semi-curved	Ubiquitous	Any	Any	Fairly common	None
Undulose	Combination of two or more alternately convex and concave curved	Ubiquitous	Any	Any	Fairly common	None
Conchoidal	As a conchoid	Ubiquitous	<1 m sq.	Any	Rare	None

In classifying the surface roughness, Fookes and Denness (1969) departed from the convention established by Patton (1966a) who developed the concept of first- and second-order roughness that is described in a later section. They compared roughness to a standard sandpaper roughness scale, shown in table 6, which is analogous to Patton's second-order roughness.

Table 6. Surface roughness classification of fissures.
Modified from Fookes and Denness, 1969.
Reproduced with permission from Geotechnique.
[Leaders (---), mean no information]

Type	Description		Occurrence	Size	Orientation	Frequency	Restrictions
	Sandpaper grade	D/L range					
Slickensided	<00 (shiny when dry, some directional features)	---	Mainly in shear surfaces	Any	Parallel to shear zones	Rare	Fine grained rocks
Very smooth	<00	---	Ubiquitous	Any	Any	Common	Fine grained rocks
Smooth	00-01	---	Ubiquitous	Any	Any	Common	None
Slightly rough	01-02	---	Ubiquitous	Any	Any	Common	None
Rough	02-03	---	Ubiquitous	Any	Any	Common	None
Very rough	03-04	---	Ubiquitous	Any	Any	Common	None
Pock marked	>04	≤1/10	Ubiquitous	Any	Any	Common	None
Pitted	---	1/10-1/2	Ubiquitous	Any	Any	Common	None

In 1970, Goodman developed a qualitative classification for joints in rock. Surface characteristics and origin formed the basis of the classification as follows:

Healed joints and incipient joints

Clean, smooth fractures

Clean, rough fractures

Filled joints, sheared zones, shale partings,
and smooth bedding

Dry or slightly moist

Wet, thin

Wet, thick

Müller and Hofmann (1971) considered the influence of discontinuities on both the mobility of rock masses and strength of rock masses. In the first case, they set up four classes of rock from the mechanical behavior standpoint. These are quasi-monolith, jointed rock, cracked rock, and shattered rock-mylonite ranging from a single body system to a loose mass. The relationship of these classes to the degree of jointing (or spacing) and the degree of connection of joints (or separation as defined by Müller and Hofmann) is reproduced as figure 3. In this figure it can be seen that the least chance for mobility falls in the area of least jointing and least continuity of jointing.

Müller and Hofmann (1971) classified the strength of a rock mass on the basis of joint spacing, as before, and the strength of the homogeneous rock, as defined by compressive strength and degree of weathering. The classification is similar to that proposed by John in 1962. Rock masses were judged to range from strong rock to very weak rock on the basis of these two parameters as shown in figure 4. Sound rock with little jointing is appropriately classed as strong rock. Both scales are logarithmic as first used by John.

Franklin, Broch, and Walton (1971) developed a classification of rock-mass quality based on discontinuity (or fracture) spacing and rock strength as determined from cores. They did so following consideration of such other discontinuity measures as orientation, continuity, irregularity, tightness, and filling material. They stated that "other things being equal, a strong mass consists of large blocks with few fractures; a weaker rock mass has smaller blocks grading ultimately into soil materials." They also considered spacing to be more easily measured than the other properties mentioned.

Figure 5 illustrates their universal rock-quality classification system ranging from extremely high (EH) to extremely low (EL) as a function of strength and discontinuity spacing plotted on log scales. The terminology on the top and right sides of the diagram are categories suggested by the Geological Society of London.

The log scale concept was also used by Watkins (1971) to apply to the spacing of both bedding and jointing. The classification is reproduced as table 7 in which Watkins compares it with others that have been devised to describe discontinuity spacings. The "Binnie" classification at the bottom of table 7 avoids commonly used terms such as flaggy, massive, and blocky as they have more than one meaning.

In 1972, Clayton and Arnold prepared fracturing density and degree of weathering classifications for the evaluation of the stability of granitic slopes in the area of the Idaho batholith in northwestern United States. The five fracture density classes are reproduced as table 8. The rock weathering classes are characterized by a comprehensive combination of factors as follows:

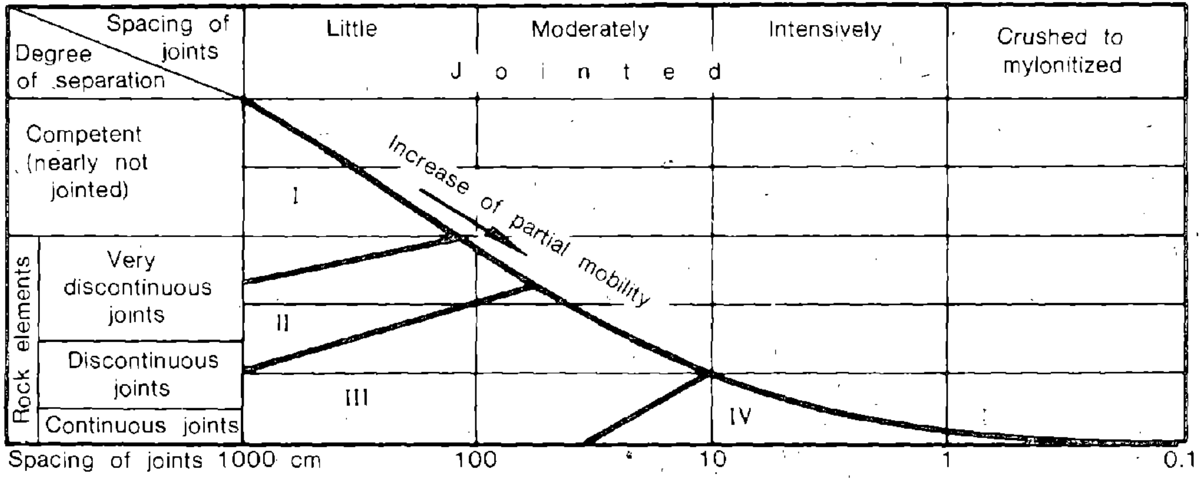


Figure 3. Rock-mass mobility classification. I, quasimololith; II, jointed rock; III, cracked rock; IV, shattered rock. Modified from Müller and Hofmann, 1971. Proceedings of Symposium on the Theoretical Background to the Planning of Open Pit Mines with Special Reference to Slope Stability, Johannesburg, 1970. Published 1971 for The South African Institute of Mining and Metallurgy by A. A. Balkema, Cape Town.

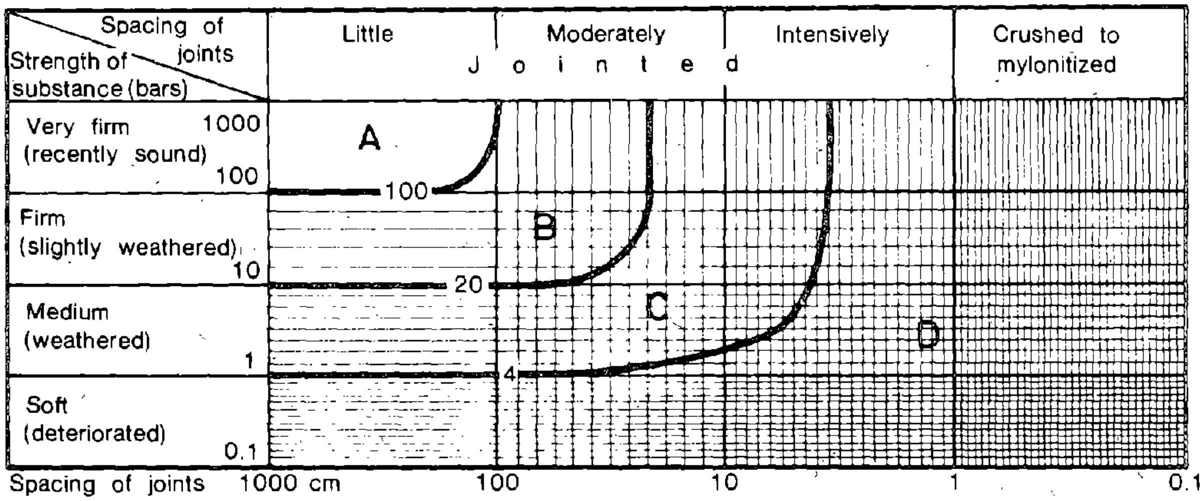


Figure 4. Rock-mass classification of Müller and Hofmann (1971). A, strong rock; B, medium rock; C, weak rock; D, very weak rock. Modified from Proceedings of Symposium on the Theoretical Background to the Planning of Open Pit Mines with Special Reference to Slope Stability, Johannesburg, 1970. Published 1971 for The South African Institute of Mining and Metallurgy by A. A. Balkema, Cape Town.

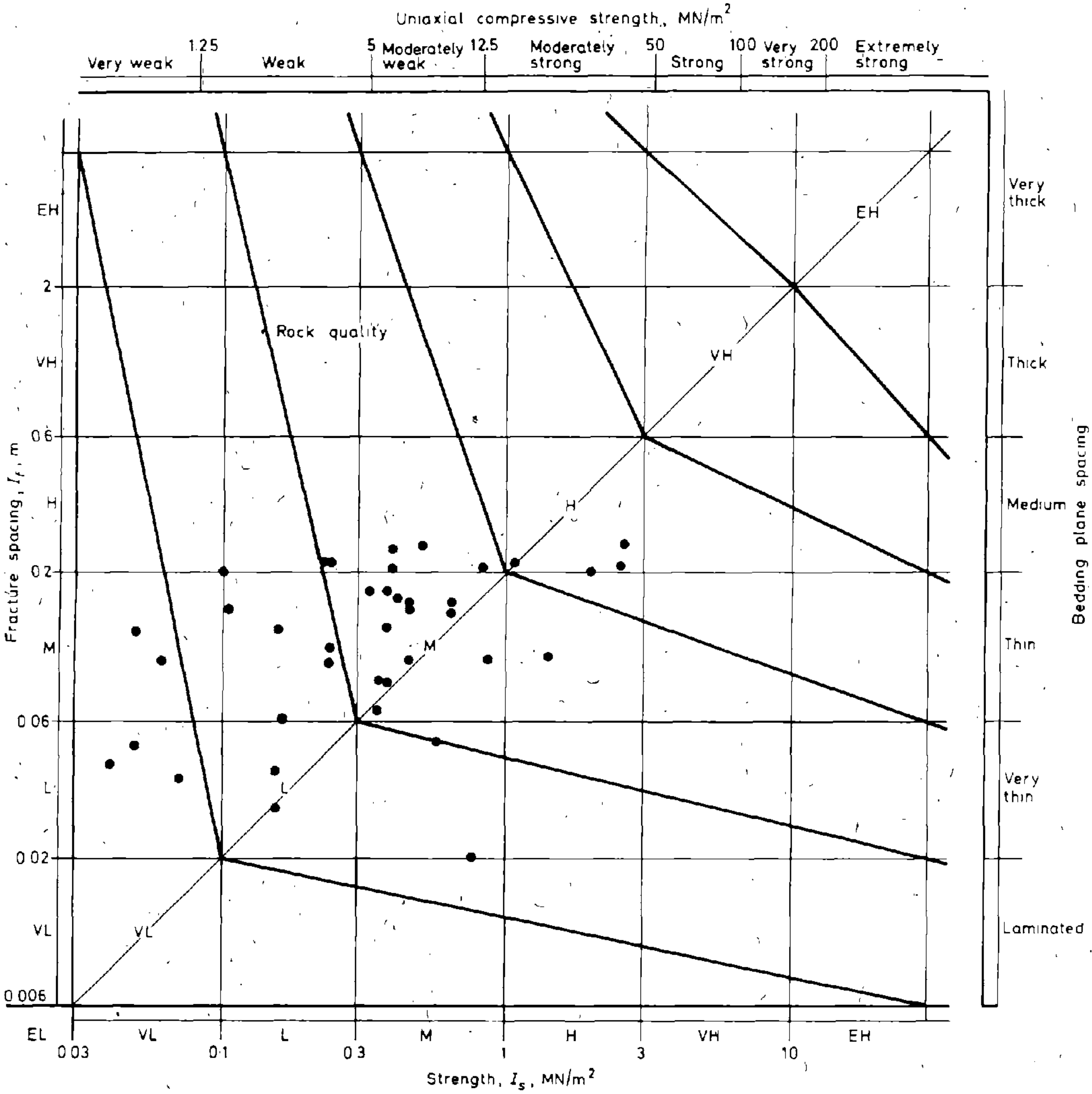


Figure 5. Rock quality classification. EH, extremely high; VH, very high; H, high; M, medium; L, low; VL, very low; I_s, point-load index strength. Modified from Franklin, Broch, and Walton (1971). Copyright Institution of Mining and Metallurgy.

Table 7. Classifications for discontinuity spacings. Modified from Watkins, 1971. Reproduced with permission from The Quarterly Journal of Engineering Geology.

1 2 6 10 2 6 10^2 2 6 10^3 2 6 10^4mm

Bedding	Fissile	Shaly	Flaggy	Massive					Payne (1942)
Bedding	Thinly laminated	Laminated	Very thin	Thin	Thick	Very thick		Geology	McKee & Weir (1953)
Bedding	Thinly laminated	Thickly laminated	Very thin	Thin	Medium	Thick	Very thick		Ingram (1954)
Jointing	Crushed	Very close	Close	Wide	Occasional				John (1962)
Jointing	Very close	Close	Moderately close	Wide	Very wide			Engineering geology	Deere (1964)
Bedding	Very thin	Thin	Medium	Thick	Very thick				
Jointing	Shattered	Very close	Close	Mod-erate	Wide	Massive			Eastaff Personal communication to Watkins (1963)
Jointing	Broken	Blocky	Solid						Coates (1964)
Mechanical Analysis	Sand	Fine gravel	Medium gravel	Coarse gravel	Cobbles	Boulders		Soil mechanics	Anonymous (1957)
Jointing	Comminuted	Shattered	Very close	Close	Moderate	Wide	Very wide	Geo-technical	Binnie & Partners (1969) unpublished
Bedding	Thinly laminated	Laminated	Very thin	Thin	Medium	Thick	Very thick		

Class 1, Unweathered Rock.--Unweathered rock will ring from a hammer blow; cannot be dug by the point of a rock hammer; joint sets are the only visible fractures; no iron stains emanate from biotites; joint sets are distinct and angular; biotites are black and compact; feldspars appear to be clear and fresh.

Class 2, Very Weakly Weathered Rock.--Very weakly weathered rock is similar to class 1, except for visible iron stains that emanate from biotites; biotites may also appear "expanded" when viewed through a hand lens; feldspars may show some opacity; joint sets are distinct and angular.

Class 3, Weakly Weathered Rock.--Weakly weathered rock gives a dull ring from a hammer blow; can be broken into "hand-sized" rocks with moderate difficulty using a hammer; feldspars are opaque and milky; no root penetration; joint sets are subangular.

Class 4, Moderately Weathered Rock.--Moderately weathered rock may be weakly spalling. Except for the spall rind, if present, rock cannot be broken by hand; no ring or dull ring from hammer blow; feldspars are opaque and milky; biotites usually have a golden yellow sheen; joint sets indistinct and rounded to subangular.

Class 5, Moderately Well Weathered Rock.--Moderately well weathered rock will break into small fragments or sheets under moderate pressure from bare hands; usually spalling; root penetration limited to fractures, unlike class 6 rock where roots penetrate through the rock matrix; joint sets are weakly visible and rounded; feldspars are powdery; biotites have a light golden sheen.

Class 6, Well Weathered Rock.--Well weathered rock can be broken by hand into sand-sized particles (grus); usually so weathered that it is difficult to determine if rock is spalling, roots can penetrate between grains; only major joints are preserved and filled with grus; feldspars are powdery; biotites may appear silver or white in thin flakes.

Class 7, Very Well Weathered Rock.--Very well weathered rock has feldspars that have weathered to clay minerals and rock is plastic when wet, no resistance to roots.

Table 8. Fracture spacing classification. Clayton and Arnold, 1972.

Class	Distance between fractures (ft.)	(cm.)	Density
1	6	180	Very low
2	4 to 6	120 to 180	Low
3	1.5 to 4	45 to 120	Medium
4	.5 to 1.5	15 to 45	High
5	.5	15	Very high

Brekke and Howard (1972) classified discontinuities according to scale. The essentially identical classification of Heuzé and Goodman (1972) is shown as figure 6 as it is more complete. This approach, although qualitative, provides an investigator with a means of estimating the influence of various discontinuities on a cut or excavation of a particular size.

The Rock Structure Rating (RSR) of Wickham, Tiedemann, and Skinner (1972) illustrates a rock quality classification designed for determining support requirements in tunneling which can be applied to slope stability problems in rock. Most tunnel stability problems are affected by the same factors as those which have been discussed earlier; that is, orientation, continuity, surface roughness, spacing, etc. The main difference is in the influence of cross-section shape and size on stability encountered in tunneling.

By 1972, Wickham, Tiedemann, and Skinner had developed two generations of the RSR concept. The second generation (RSR no. 2) requires fewer factors and improves factor interaction. The weighted geologic factors contributing to both RSR calculations and their interdependency is included as table 9 in which RSR no. 1 and RSR no. 2 determination is shown. The determination of factor values used in table 9 is illustrated in tables 10-13.

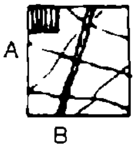
Name		Scale of Observation	Typical Spacing	Main Modes of Origin
Macro and Microfissures	5.1 cm 	Lab sample	< 25mm	Alteration and extension fracturing
(may be taken as a rock block characteristic)				
Partings; bedding joints; cleavage foliation	61cm 	In situ test block	25mm-5.0cm	Extension fracturing
Joints (A) Seams (B)	6.1m 	Exploratory excavation	5.0cm-6.1m	Extension fracturing and shear failure
Minor faults, crushed zones; sheared zones	61m 	Completed cutting	6.1m-61m	Shear failure
Major faults	610m 	Mountain range	> 61m	Shear failure

Figure 6. Classification of discontinuities according to scale. Modified from Three-dimensional approach for design of cuts in jointed rock, F. E. Heuzé and R. E. Goodman, p. 403, in Stability of rock slopes, Proc. 13th Symposium on rock mechanics, 1972.

Table 9. Comparison of RSR no. 1 and no. 2 factors. Modified from G. E. Wickham, H. R. Tiedemann, and E. H. Skinner, Support determinations based on geologic predictions, 1972, RETC Proceedings, K. S. Lane and L. A. Garfield, eds., AIME, New York, 1972, p. 51. [←→ denotes interaction of factors]

Geologic Factors

RT - Rock Type
CA - Core Analysis
SV - Seismic Velocity Ratio
JO - Joint Orientation
RF - Folding & Discontinuities
MF - Major Faults
JS - Joint Seal
CT - Cover over Tunnel
WF - Water Inflow
RM - Rock Modulus
RH - Rock Hardness
JP - Joint Pattern (Spacing)

RSR #1		RSR #2		
Factors	Max. Value	Parameters		Max. Value
RT ←→ CA	35	RT → RF	"A"	30
RT ←→ SV	10	+		
RT ←→ JO	9	JP → JO	"B"	50
RT ←→ RF	14	+		
RT ←→ MF	13	WF ←→ JS	"C"	20
RT ←→ JS	3			
RT ←→ CT	2			
RT ←→ WF	4			
RT ←→ RM	10			
RSR Value	100	RSR Value		100

Table 10. Core analysis factor (CA) used in RSR no. 1. Modified from Wickham and Tiedemann, 1972. [RQD, Deere's evaluation; fracture frequency, fractures per 30.5 cm of core; visual inspection, individual judgment]

Rock Type	Rock Quality Designation				
	0-25%	25-50%	50-75%	75-90%	90-100%
	Fracture Frequency (fractures per 30.5 cm)				
	>4.5	3-4.5	2-3	1-2	<1
	Visual Inspection				
	Very poor	Poor	Fair	Good	Very Good
Igneous	6	16	24	30	35
Sedimentary	4	10	16	24	35
Metamorphic	5	12	18	27	35

Table 11. Parameter "A" of RSR no. 2 classification (see table 9). G. E. Wickham, H. R. Tiedemann, and E. H. Skinner, Support determinations based on geologic predictions, 1972, RETC Proceedings, K. S. Lane and L. A. Garfield, eds., AIME, New York, 1972, p. 52

		Geologic structure		
Basic rock type	Massive	Slightly faulted or folded	Moderately faulted or folded	Intensely faulted or folded
Igneous	30	26	15	10
Sedimentary	24	20	12	8
Metamorphic	27	22	14	9

Table 12. Parameter "B" of RSR no. 2 classification (see table 9). G. E. Wickham, H. R. Tiedemann, and E. H. Skinner, Support determinations based on geologic predictions, 1972, RETC Proceedings, K. S. Lane and L. A. Garfield, eds., AIME, New York, 1972, p. 53

Average Joint Spacing (meters)	Strike ⊥ to Axis					Strike ∥ to Axis		
	Direction of Drive							
	Both	With Dip		Against Dip		Both		
	Dip of Prominent Joints[1]							
	1	2	3	2	3	1	2	3
<0.5-0.15 (Closely Jointed)	14	17	20	16	18	14	15	12
.15-.31 (Moderately Jointed)	24	26	30	20	24	24	24	20
.31-.61 (Moderate to Blocky)	32	34	38	27	30	32	30	25
.61-1.22 (Blocky to Massive)	40	42	44	36	39	40	37	30
>1.22 (Massive)	45	48	50	42	45	45	42	36

[1] 1 = <20°, 2 = 20°-50°, 3 = 50°-90°.

Table 13. Parameter "C" of RSR no. 2
classification (see table 9).
G. E. Wickham, H. R. Tiedemann, and E. H. Skinner,
Support determinations based on geologic predictions,
1972, RETC Proceedings, K. S. Lane and L. A. Garfield,
eds., AIME, New York, 1972, p. 54.

Anticipated Water Inflow (gpm/305 m)	Sum of Parameters A + B					
	20-45			46-80		
	Joint Condition[1]					
	1	2	3	1	2	3
None	18	15	10	20	18	14
Slight (200 gpm)	17	12	7	19	15	10
Moderate (200-1000 gpm)	12	9	6	18	12	8
Heavy (1000 gpm)	8	6	5	14	10	6

[1] 1 = tight or cemented, 2 = slightly weathered, 3 = severely weathered or open.

The use of Deere's RQD in the first-generation (no. 1) RSR development is shown in table 10. Tables 11-13 show how the three streamlined parameters A, B, and C used in RSR no. 2 are derived. Of interest is the important part discontinuities play in both systems. We find included such items as joint spacing, orientation, separation, and frequency combined with bedding, jointing, and faulting. Lacking are the influence of surface roughness and filling materials.

In 1973, Bieniawski, in a comprehensive paper on classification of rock masses, modified the older rock mass classifications proposed by John (1962) and Müller and Hofmann (1971). In it he retained the log-log plotting of joint spacing and rock strength. As modifications he divided the spacing scale into five subdivisions rather than the four of Müller and Hofmann and, following John, used uniaxial compressive strength of the rock samples as a more definitive measure of strength. In addition, ranges of cohesion and friction angle for the classes of rock-mass strength were given. The modified diagram is shown in figure 7.

In the same paper, Bieniawski (1973) proposed his geomechanical classification of jointed rock masses which is based on Deere's practical and simple Rock Quality Designation (RQD). A better term than "jointed" might have been "discontinuous" as his term "joint" includes joints, faults, bedding planes, and other surfaces of weakness. The classification divides rock masses into five categories or classes each having similar degrees of weighted rock mass characteristics. The parameters used to define these categories are those that contribute to the behavior of a discontinuous rock mass. These are RQD, weathering, intact rock strength, joint spacing, joint separation, joint continuity, orientation, and ground-water inflow. Determination of the relative

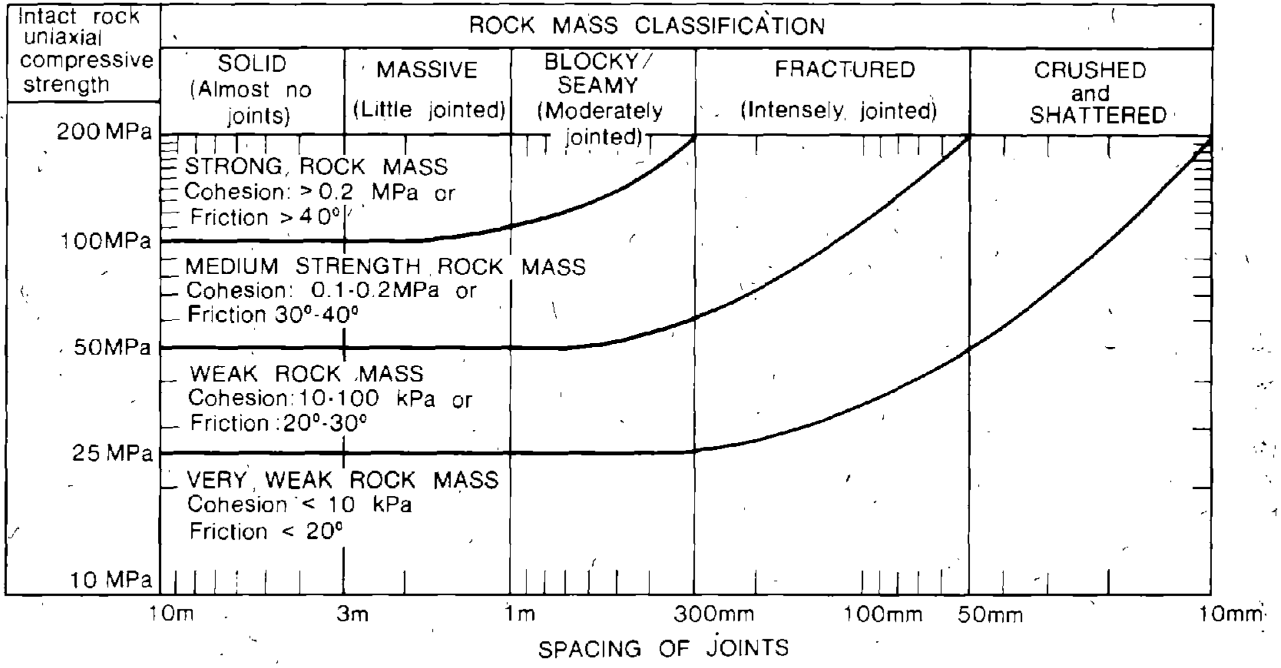


Figure 7. Rock-mass strength classification of Bieniawski (1973). MPa, megaPascals; kPa, kiloPascals. Reproduced with permission from South African Institution of Civil Engineers.

importance or weighting of these parameters except for weathering on the assessment of rock-mass strength was based on work by Wickham, Tiedemann, and Skinner (1972). A qualitative degree of weathering classification was proposed which was based on recommendations of several geological and geotechnical groups (see section on features associated with discontinuities). The geomechanical classification proposed by Bieniawski is shown as table 14 and his individual parameter rating system in table 15. Bieniawski advised caution in the use of the classification in the case of shales and other swelling materials unless some measure of slake durability (Cording and others, 1975) or "weatherability" is added as another parameter. In this classification the influence of discontinuities is quite obvious. However, the important strength factors of surface roughness and joint filling material are absent or not compensated for, omissions which could affect the use of the classification in rock-slope stability evaluation.

Table 14. Geomechanics classification of Bieniawski (1973). Reproduced with permission from South African Institution of Civil Engineers.

Item	Class No. and its description	1 Very good	2 Good	3 Fair	4 Poor	5 Very poor
1	Rock quality RQD (%)	90-100	75-90	50-75	25-50	<25
2	Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Complete weathered
3	Intact rock strength, MPa	>200	100-200	50-100	25-50	<25
4	Spacing of joints	>3m	1-3 m	0.3-1 m	50-300 mm	<50 mm
5	Separation of joints	<0.1 mm	<0.1 mm	0.1-1 mm	1-5 mm	>5 mm
6	Continuity of joints	Not continuous	Not continuous	Continuous no gouge	Continuous with gouge	Continuous with gouge
7	Ground water inflow (per 10 m of adit)	None	None	Slight <25 L/min	Moderate 25-125 L/min	Heavy >125 L/min
8	Strike and dip orientation	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable

Table 15. Individual weighted ratings used in
geomechanics classification.
Bieniawski, 1973. Reproduced with permission
from South African Institution of Civil Engineers.

Item	Parameter		Class				
			1	2	3	4	5
1	Rock quality RQD		16	14	12	7	3
2	Weathering		9	7	5	3	1
3	Intact rock strength		10	5	2	1	0
4	Spacing of joints		30	25	20	10	5
5	Separation of joints		5	5	4	3	1
6	Continuity of joints		5	5	3	0	0
7	Ground water		10	10	8	5	2
8	Strike and dip orientations	Tunnels	15	13	10	5	3
		Foundations	15	13	10	0	-10

Also in 1973, Wahlstrom proposed a procedure for rating competency or rock quality for tunnel construction. It is based on the characteristics of the original unfractured and unaltered rock, the effects of faulting, the influence of joints on competency, and the effects of rock alteration on strength. The characteristics are weighted according to Wahlstrom's evaluation of their impact on rock-mass strength. He noted that an engineering classification of rock from drill-hole information must be based on subjective appraisal of the variable with a degree of geological intuition. In addition, all of the desired data may not be available for an explicit determination of rock quality.

Wahlstrom's outline for rating rock competency is reproduced as figure 8. The numerical ratings shown are based on subjective estimates of the categories. In the case of jointing, for instance, the criteria for the rating are spacing, number of sets, and whether wet or dry. The joint rating is obtained without regard to surface roughness or orientation. The classification is admittedly quite qualitative. However, Wahlstrom wrote that the use of the rating scheme would serve the special purpose of emphasizing the need for the collection of definitive data.

Barton, Lien, and Lunde (1974) developed a classification which recognized the advantages of the rock-mass strength classifications proposed by Wickham, Tiedemann, and Skinner (1972) and Bieniawski (1973) as well as the omission of quantitative input concerning surface roughness and strength of joint filling material. They also added rock load which is of importance in tunneling.

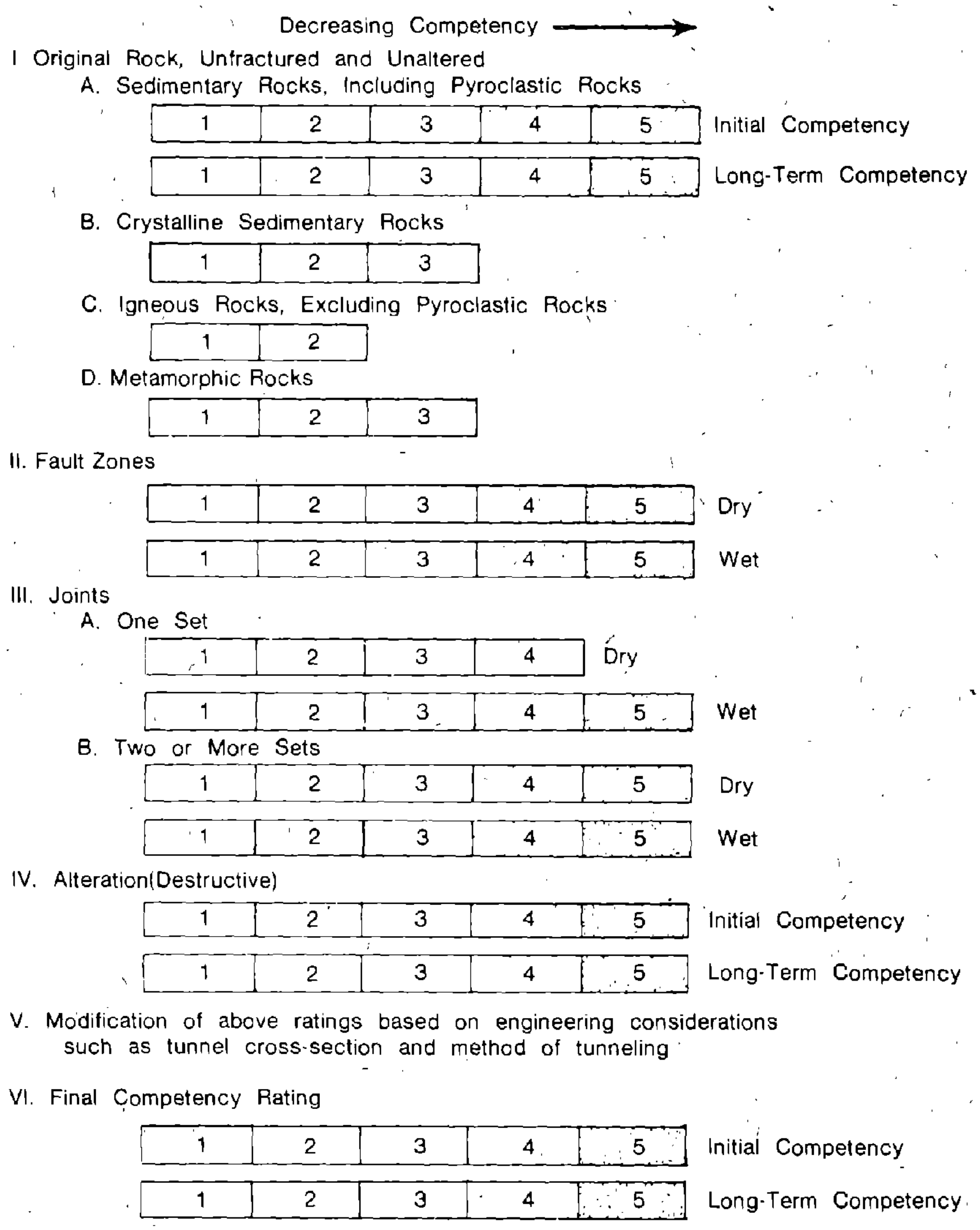


Figure 8. Rock-mass quality classification of Wahlstrom (1973). Tunneling in Rock, E. E. Wahlstrom, in Developments in geotechnical engineering 3, Elsevier Scientific Publishing Co., 1973.

Six parameters were chosen to evaluate rock-mass quality. These are RQD, number of discontinuity sets (J_n), surface roughness (J_r), filling and wallrock alteration (J_a), water conditions (J_w), and a stress-reduction factor (SRF). Tables 16, 17, and 18 show the parameters, their subdivisions, and the weighting applied to them for comparison with earlier classifications by other investigators.

Table 16. RQD, joint set, and joint roughness descriptions and ratings.
Modified from Barton, Lien, and Lunde, 1974.
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I.	Rock Quality Designation	(RQD)	
A.	Very poor	0- 25	Note: (i) Where RQD is reported measured as <10 (including 0) a nominal value of 10 is used to evaluate Q in Eq. (1) (ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate
B.	Poor	25- 50	
C.	Fair	50- 75	
D.	Good	75- 90	
E.	Excellent	90-100	
II.	Joint Set Number	(Jn)	
A.	Massive, no or few joints	0.5-1.0	
B.	One joint set	2	
C.	One joint set plus random	3	
D.	Two joint sets	4	
E.	Two joint sets plus random	6	
F.	Three joint sets	9	
G.	Three joint sets plus random	12	
H.	Four or more joint sets, random, heavily jointed, "sugar cube," etc.----------	15	
J.	Crushed rock, earthlike-------	20	
III.	Joint Roughness Number (a) Rock wall contact and (b) Rock wall contact before 10 cms shear	(Jr)	
A.	Discontinuous joints------------	4	Note: (i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m (ii) Jr = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are favorably oriented
B.	Rough or irregular, undulating--	3	
C.	Smooth, undulating--------------	2	
D.	Slickensided, undulating--------	1.5	
E.	Rough or irregular, planar------	1.5	
F.	Smooth, planar------------------	1.0	
G.	Slickensided, planar------------	0.5	
	(c) No rock wall contact when sheared		
H.	Zone containing clay minerals thick enough to prevent rock wall contact--------	1.0 (nominal)	
J.	Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact------------------	1.0 (nominal)	

Table 17. Joint alteration and joint water reduction descriptions and ratings.
Barton, Lien, and Lunde, 1974. Reproduced with permission from Springer-Verlag.

IV.	Joint Alteration Number	(Ja)	ϕr (approx.)	
	(a) Rock wall contact			
A.	Tightly healed, hard, non-softening, impermeable filling, i.e. quartz or epidote	0.75	(---)	Note: (i) Values of (ϕ)r are intended as an approximate guide to the mineralogical properties of the alteration products, if present
B.	Unaltered joint walls, surface staining only	1.0	(25°-35°)	
C.	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	(25°-30°)	
D.	Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20°-25°)	
E.	Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum, and graphite, etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2 mm or less in thickness)	4.0	(8°-16°)	
	(b) Rock wall contact before 10 cms shear			
F.	Sandy particles, clay-free disintegrated rock, etc.	4.0	(25°-30°)	
G.	Strongly over-consolidated, non-softening clay mineral fillings. (Continuous, <5 mm in thickness)	6.0	(16°-24°)	
H.	Medium or low over-consolidation, softening, clay mineral fillings. (Continuous, <5 mm in thickness)	8.0	(12°-16°)	
J.	Swelling clay fillings, i.e. montmorillonite. (Continuous, <5 mm in thickness.) Value of Ja depends on percent of swelling clay-size particles, and access to water, etc.	8.0-12.0	(6°-12°)	

Table 17. Joint alteration and joint water reduction descriptions and ratings (continued)

IV.	Joint Alteration Number--cont. (c) No rock wall contact when sheared	(Ja)	ϕr (approx.)	
K. L. M.	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6.0,8.0 or 8.0-12.0	(6°-24°)	
N.	Zones or bands of silty- or sandy clay, small clay fraction (non-softening)	5.0		
O. P. R.	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10.0,13.0 or 13.0-20.0		(6°-24°)
V.	Joint Water Reduction Factor	(Jw)	Approx. water pressure (kg/cm)	
A.	Dry excavations or minor inflow, i.e. <5 1/min. locally	1.0	<1	Note: (i) Factors C to F are crude estimates. Increase Jw if drainage measures are installed (ii) Special problems caused by ice formation are not considered
B.	Medium inflow or pressure occasional outwash of joint fillings	0.66	1.0- 2.5	
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10.0	
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5-10.0	
E.	Exceptionally high inflow or water pressure at blasting, decaying with time	0.2-0.1	>10.0	
F.	Exceptionally high inflow or water pressure continuing without noticeable decay	0.1 -0.05		>10.0

Table 18. Stress reduction factor description and rating.
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VI.	Stress Reduction Factor (a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated	(SRF)	Note: (i) Reduce these values of SRF by 25-50 percent if the relevant shear zones only influence but do not intersect the excavation
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0	
B.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation <50 m)	5.0	
C.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation >50 m)	2.5	
D.	Multiple shear zones in competent rock (clay-free) loose surrounding rock (any depth)	7.5	
E.	Single shear zones in competent rock (clay free) (depth of excavation <50 m)	5.0	
F.	Single shear zones in competent rock (clay free) (Depth of excavation >50 m)	2.5	
G.	Loose open joints, heavily jointed or "sugar cube," etc. (any depth)	5.0	

Table 18. Stress reduction factor description and rating (continued)

VI. Stress Reduction Factor--cont.				(SRF)
(b) Competent rock, rock stress problems	σ_c/σ_1	σ_t/σ_1		
H. Low stress, near surface	>200	>13	2.5	(ii) For strongly anisotropic stress field (if measured) when 5< σ_1/σ_3 <10, reduce σ_t and σ_c to to 0.8 σ_c and 0.8σ_t; when σ_1/σ_3>10, reduce σ_c and σ_t to 0.6 σ_c and 0.6 σ_t where: σ_c = unconfined compression strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses
J. Medium stress	200-10	13-0.66	1.0	
K. High stress, very tight structure (usually favorable stability, may be unfavorable to wall stability)	10-5	0.66-0.33	0.5-2.0	
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10	
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20	
(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures				
N. Mild squeezing rock pressure			5-10	
O. Heavy squeezing rock pressure			10-20	
(d) Swelling rock; chemical swelling activity depending on presence of water				(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H)
P. Mild swelling rock pressure			5-10	
R. Heavy swelling rock pressure			10-15	

From these parameters, Barton, Lien, and Lunde (1974) have devised an equation which defines rock-mass quality, Q, as follows:

$$Q = (RQD/J_n).(J_r/J_a).(J_w/SRF) \quad .$$

The term RQD/J_n represents the overall structure of the rock mass. J_r/J_a defines the roughness and degree of joint alteration or influence of filling material and is a fair approximation of mass shear strength. J_w/SRF is admittedly a complicated qualitative factor that defines "active stresses" in the rock mass. Values of Q and the six parameters for representative case histories are provided in the 1974 paper. As in the other rock-quality classifications, the presence and nature of discontinuities play the dominant role. As noted earlier, such measures of rock-mass quality are directly applicable to rock-slope stability analysis.

For the reader who does not have access to the geotechnical literature, attention is directed to Goodman's text titled "Methods of Geological Engineering in Discontinuous Rocks" (1976). In it can be found summaries of the various kinds of discontinuities, their characteristic features, relative importance, and input into the various systems for classifying rock masses.

Features of the more important discontinuities

Joints

General

As noted earlier, the term "joint" has multiple meanings in geotechnical usage. In order to explore the influence of joints on slope stability and the characteristics which determine this influence, the broad meaning will be used; that is, that a joint refers to all or part of a family of discontinuities occurring in rock masses (Goodman, 1976). Specific features associated uniquely with bedding surfaces, foliation, and shearing, considered to be "joints" in this context, will be covered in later sections. It bears repeating that the discontinuities discussed will primarily influence and be a part of rock masses. Soils that have not previously failed inherently have strengths that permit failure to occur without significant influence from discontinuities within the soil mass (Piteau, 1971). A good summary of jointing from the geotechnical viewpoint has been provided by Attewell and Farmer (1976).

The importance of jointing in its broadest sense in contributing to the instability of rock slopes cannot be overemphasized as joints so greatly affect rock mass strength. The reduction in strength of intact rock by jointing determined by Lane and Heck (1964) is shown graphically in figure 9. Hamel (1974) has written that comprehensive and reliable information on discontinuities is probably the single most important factor in evaluating slope stability. This has been recognized by many

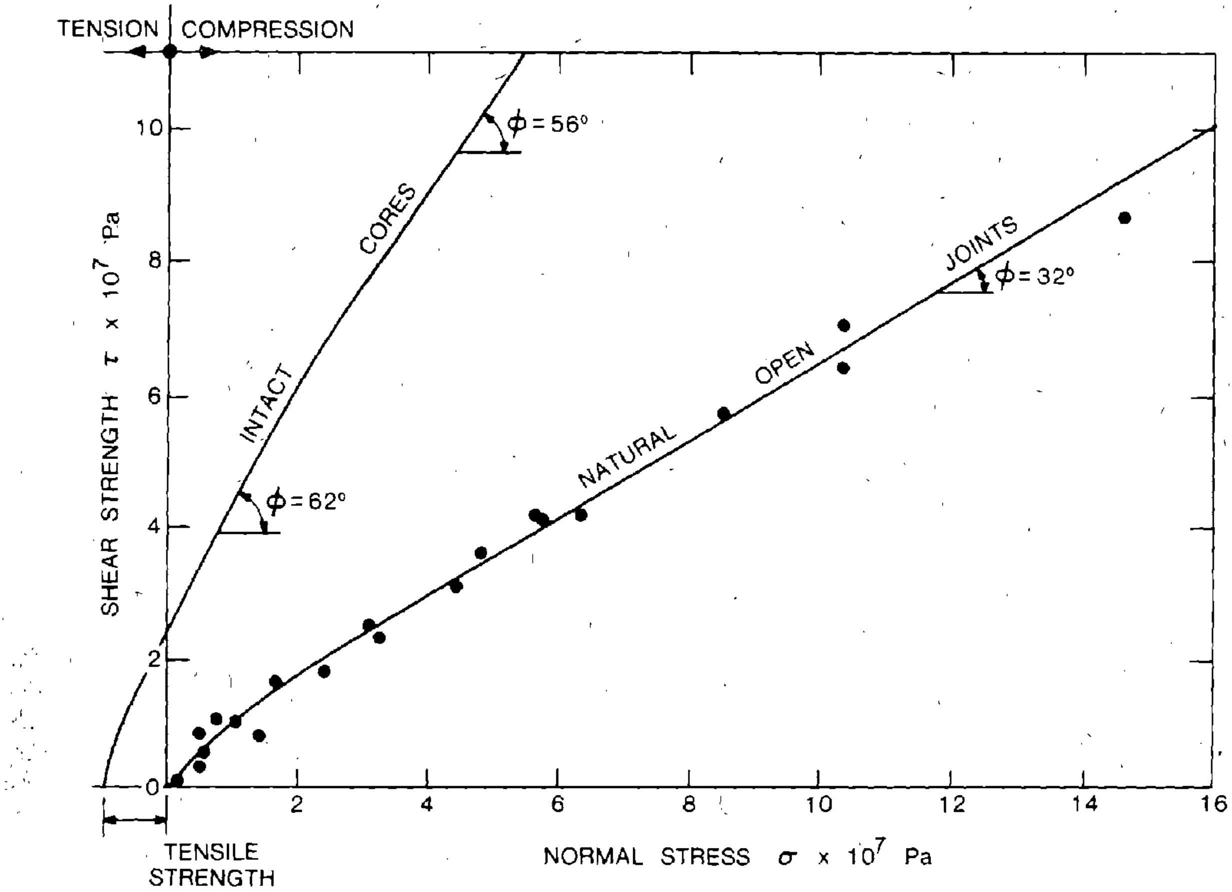


Figure 9. Comparison of Mohr strength envelopes of intact cores and natural open joints in quartz monzonite. Modified from Triaxial testing for strength of rock joints, K. S. Lane and W. J. Heck, in Proc. 6th Symposium on rock mechanics Rolla, Mo., 1964.

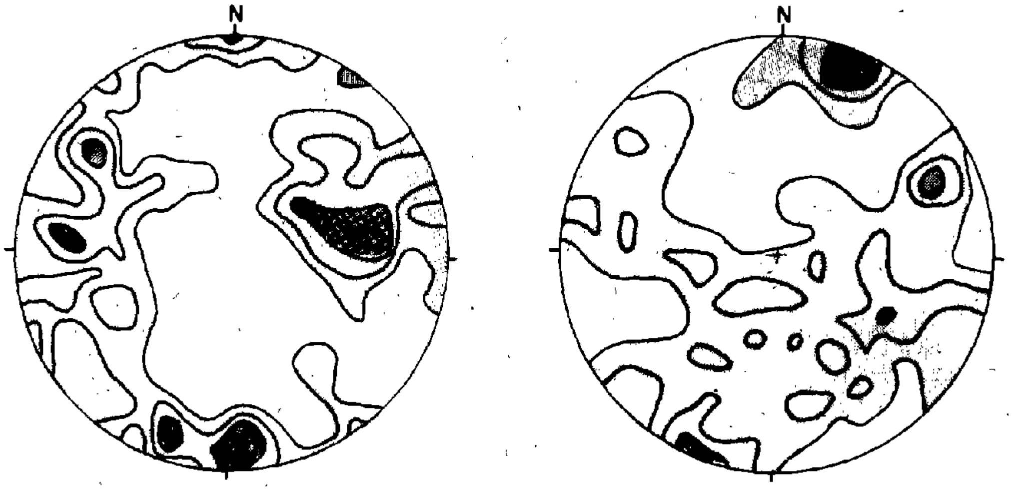
other investigators such as Terzaghi (1962), Philbrick (1963), Denkhaus (1965), Deere and others (1967), Piteau (1971), and Goodman (1972). Müller (1964a) concluded that a failure surface in rock would not develop except where large joints are present. He wrote that such failures could occur to depths of 150 m. Lo, Lee, and Gelinas (1972) reported an average depth of 200 m for the failure surface at Vaiont. Documentation of the importance of jointing in major slope failures in rock has been provided by Müller (1964b), Hamel (1972), Cruden and Krahn (1973), Londe (1973), and Cruden (1976). Esu (1966) has written of the control exerted by joints on slides in overconsolidated clay shales, indicating that their influence is not limited to materials classed geotechnically as rocks.

Many factors relating to joints contribute to the stability of slopes. Included are such things as orientation, spacing, continuity, surface characteristics, and nature of filling material if present (fig. 1). As observed by Goodman (1972), these are features that often are bypassed in purely geological descriptions. Each will be discussed in the sections that follow. Each factor as well as combinations of the factors, greatly influence the strength of rock masses as noted earlier and summarized by Bieniawski (1973) and Cording and others (1975). Weathering or alteration of rock along joints, while not a joint characteristic, may reduce the strength of the jointed rock mass (Terzaghi, 1946; N. Barton, 1973; Barton and others, 1974). Terzaghi (1936) reported strengths of jointed and fissured clays being as little as one tenth of that of intact samples of the same material. Skempton, Schuster, and Petley (1969) have obtained similar results. The combined factors create problems of detection, discrimination, and projection when drilling and geophysical methods are employed for exploratory purposes.

Orientation

Of the several factors that are related to jointing, orientation or attitude (dip and strike) must rank as the most important, for without favorable orientation of discontinuities relative to a potential failure direction, the other factors normally would have greatly reduced influence (Piteau, 1972). McMahon (1968a) noted that rock masses with irregularly oriented joint systems have a greater degree of block interlock and less mechanical anisotropy than those masses with regularly oriented joints. The anisotropy referred to increases with increased regularity of orientation because preferred directions of weakness are generated.

McMahon (1968a) has shown that the degree of irregularity or randomness of joints may be shown graphically on an equal-area projection of joint poles as seen in figure 10. From such a plot, he devised a joint dispersion index as a quantitative measure of randomness of joint distribution. The index is defined as the area enclosed by a given joint concentration contour on a plot expressed as a percentage of the area covered by the same contour of a random sample from a uniform distribution of an equal number of points.



REGULAR JOINT PATTERN

JOINT DISPERSION INDEX = 50%

IRREGULAR JOINT PATTERN

JOINT DISPERSION INDEX = 90%

Figure 10. Equal-area stereographic projections of poles for regular and irregular joint patterns. Modified from B. K. McMahon, Indices related to the mechanical properties of jointed rocks, status of Practical Rock Mechanics, N. E. Grovenor and B. H. Pauldings, eds., AIME, New York, 1968, p. 123.

The clustering of poles representative of various systematic planar discontinuities such as joints and bedding surfaces makes the stereographic projection a useful means of obtaining average orientation data by graphic means. The three joint sets and bedding shown in figure 11 from Piteau and Martin (1977) illustrate the usefulness of this form of data presentation. The inherent three-dimensional characteristics of pole plots provides a considerable advantage over other plotting techniques such as joint rosettes where only direction of planar orientation is shown. The structural geology texts by Billings (1972) and Spencer (1977) may be referred to for details. Robertson (1971) described a rectangular plotting method and compared it to the stereographic projection.

Influence of two or more preferred orientations of joints on slope stability was noted by Broili (1967) at the site of the Vaiont slide. McMahon's joint dispersion index as well as the equal area pole plot both identify quantitatively the presence of more than one preferred direction of jointing. Rawlings (1968) noted that the intersection of several joints observed in exposures could define a wedge-shaped block subject to failure. Recognition f the interaction of multiple planes of failure has led to graphical analyses of the stability of jointed rock by many investigators such as Heuzé and Goodman (1972), Hoek, Bray, and Boyd (1973), and Londe (1973) following the definitive work of John (1968, 1970b, 1971).

The prominence of one or more preferred directions of jointing has led to the definition of the joint set in which the joints in a given set roughly parallel one another (fig. 12). Jennings and Robertson (1969) and Heuzé (1974) emphasized the importance of groups or sets of discontinuities and the fact that all such measurements result in a statistical measure of their central tendencies; that is, mean or average orientation and range of value. The spatial distribution of multiple joint sets may be referred to as systems of joints (McMahon, 1968a; Piteau, 1971; Goodman, 1976). It is common to find joint sets forming regional patterns. Such patterns will be discussed later in this section. Piteau and Russell (1972) have used the cumulative sums technique for the prediction of the average orientation of a joint set.

Piteau (1971) has separated various combinations of joints (or failure surfaces) into failure modes such as the block failure mode and the wedge failure mode depending on the number and angular intersection of the involved joints. This is similar to Goodman's (1972, 1976) interest in the shape of blocks formed by intersecting discontinuities. Kohlbeck and Scheidegger (1977) have written that three joint sets are required for a "fundamental rock cell or fundamental joint-parallelepiped."

McMahon (1974) has used the term "mode" in a different way but still involving orientation of joints. He has proposed four modes

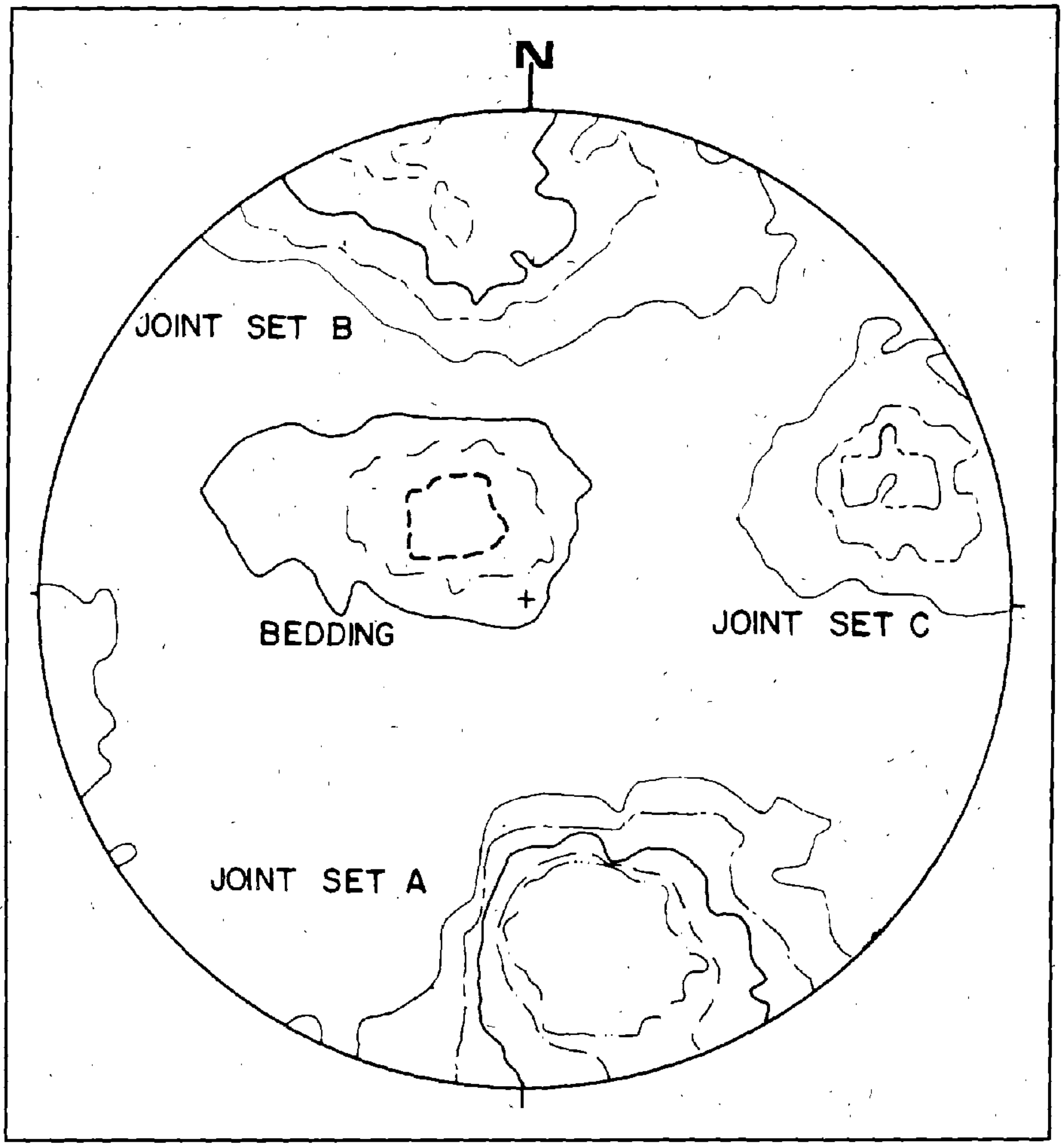


Figure 11. Equal-area stereographic projection of poles for bedding and multiple joint sets. Piteau and Martin, 1977. Reproduced with permission from Canadian Institute of Mining.

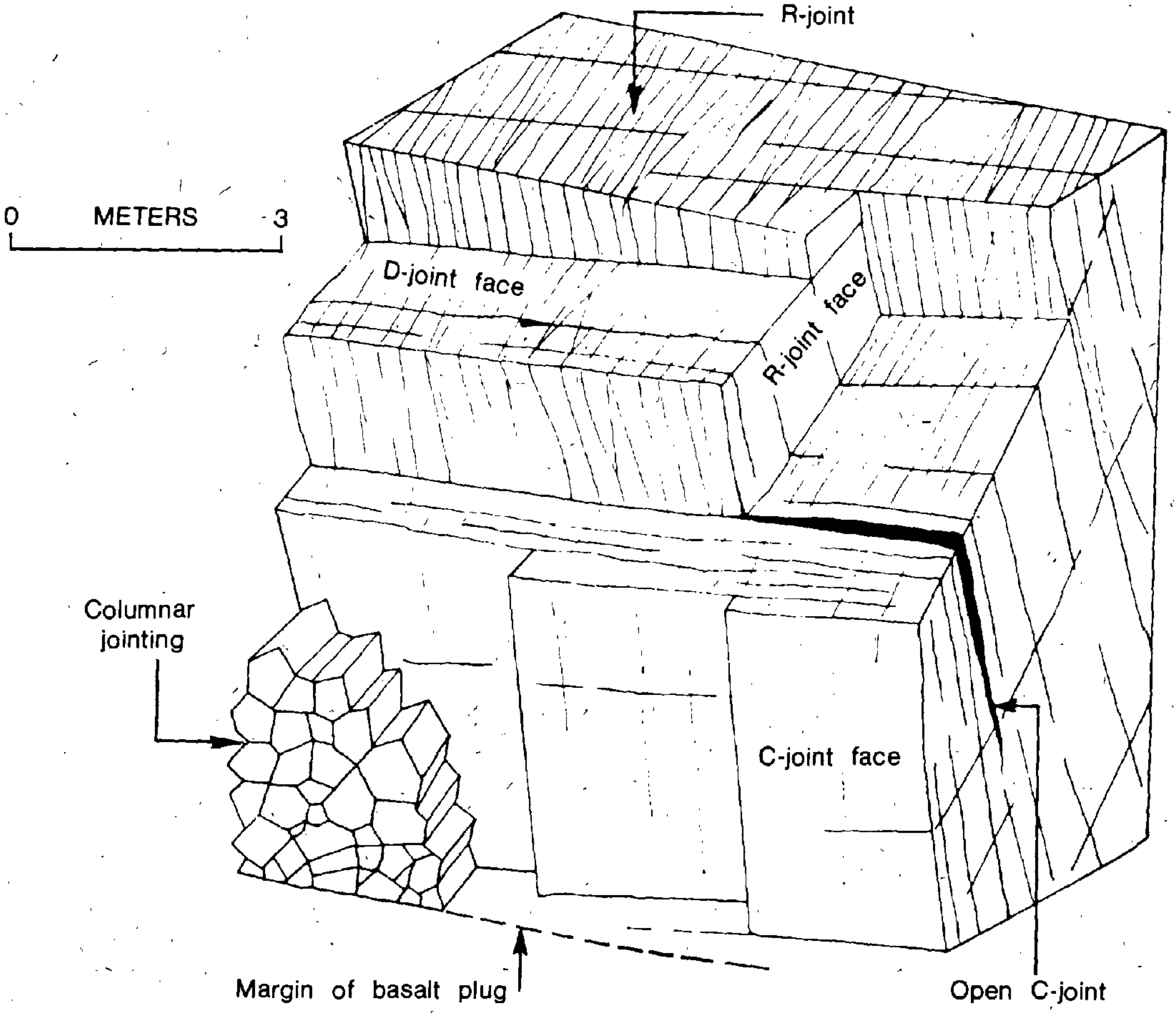


Figure 12. Block diagram illustrating the relationship between the various joint sets. Scale approximate. Modified from Price and Knill, 1967. Reproduced with permission from Geotechnique.

viewed two-dimensionally, normal to a slope, with from one to three sets of joints suitably oriented to create instability downslope. From these modes he has proposed the critical dip concept in which the dip is measured in the direction of movement of the slide mass on the flattest of the joints at a state of critical equilibrium.

The orientation of jointing relative to a slope whether natural or excavated, is of paramount importance when one is concerned with slope stability (Calder, 1970). The two factors of primary importance are whether the joints (or discontinuities) intersect (or daylight) the slope or excavated surface at angles less than the surface angle and, if they do, whether the dip angles of the joints exceed the angles of friction along the joint surface (Lacy, 1963; Patton, 1966a; Deere and others, 1967; Brawner and Gilchrist, 1970; Calder, 1970). While most of the geotechnical literature is concerned with excavated slopes, Hadley (1964, 1974) has shown the influence of erosional oversteepening of slopes having discontinuities dipping with the slope on the Hebgen or Madison Canyon slide in Montana. Piteau (1971) has noted the importance of observing naturally formed slopes so that stability estimates may be made from the natural angles relative to the orientation of joints within the rock mass.

Exceptions to the rule of daylighting of joints have been noted by some investigators. Philbrick (1963) has compared two cut slopes for a given joint set and bedding surface arrangement. For horizontal bedding, he proposed that joints dipping into a slope could create greater instability than joints oriented parallel to the slope. Similar situations have been described by O'Neill (1963) in foliated metamorphic rocks and Bukovansky, Rodríguez, and Cedrún (1974) in interbedded sediments. In both cases the dip of the primary discontinuities was into the slope. It would appear from the descriptions, however, that there were less obvious joints normal to the principal features (and thus daylighted) which contributed to the slope failures.

In the case cited by Bukovansky, Rodríguez, and Cedrún (1974) jointing perpendicular to bedding was noted as a factor. The occurrence of joints perpendicular to bedding regardless of the latter's orientation also has been noted by Henkel (1961), Esu and Calabrisi (1969), Skempton, Schuster, and Petley (1969), Hamel (1972), and Briggs, Pomeroy, and Davies (1975). Such jointing is common and can play an important role in slope stability in sedimentary rocks. Muller's (1964b) analysis of the slope failure at Vaiont and that of the Frank slide by Cruden and Krahn (1973) are examples.

The presence of jointing parallel to valley walls has caused many slope failures (Branthoover, 1974). Stress relief resulting from stream and glacial erosion is generally accepted as the cause (Aisenstein, 1965; Bjerrum and Jørstad, 1968; Knill, 1968; Gray and others, 1974; Attewell and Farmer, 1976). As topographically controlled stress-relief features, such joints may occur independently of local structure and rock type. They are restricted to shallow depths but can be expected to develop with excavation of highly stressed rocks. Because these features parallel the surface, they may escape notice in a field investigation (Branthoover, 1974). Fissure orientation relative to surface topography and ice loading also has been observed in glacial till (McGown and others, 1974).

Aside from the orientation of joints that is the result of stress relief associated with erosion or excavation, joint orientation is normally associated with structural or tectonic causes. Joints appear to be better developed in competent rocks and poorly developed in incompetent materials, a relationship that is probably related to the elastic moduli of the materials (Price, 1959). The three prominent sets reported by Mahtab, Bolstad, and Kendorski (1973) in competent igneous rock support this view. Price also has stated that joint orientation in igneous rocks is determined by stress conditions during emplacement of the molten material.

The fact that multiple sets of joints occur in rock masses is an expression of the tectonic origin of the stress fields that define the preferred orientations. Such oriented joints were considered to be systematic joints by Hodgson (1961). Piteau (1971) has named areas having a systematic arrangement of joints "structural regions." Robertson (1971) considered them to be zones of similar strength. The regularity and predictability of the joint sets within a structural region have led Piteau, Robertson, and Steffen and Jennings (1974) to call them "design joints". They are more commonly referred to as "regional joints". Jennings and Robertson (1969) observed that such joints can be grouped into a limited number of joint sets and that all joints in a given set will be identical within a statistical range.

Many investigators have noted the relationship between inclined failure-causing discontinuities and a fold or fault origin of the inclination (Henkel, 1961; Lacy, 1963; Hadley, 1964; Broili, 1967; Rawlings, 1968; Cruden and Krahn, 1973; Cruden, 1976). Patton (1966a) in his comprehensive work on discontinuities collected orientation data for discontinuities in stable, unstable, and failed slopes in sandstone and limestone. From his sample populations, he found that in sandstone the approximate average dip of discontinuities for stable slopes was 20°. This increased to 30° for unstable slopes and to 38° for failed slopes. Average dip values for carbonate rock were approximately 24°, 37°, and 44°, respectively.

Obviously other factors such as surface roughness influence such values. However, such measurements do indicate the predictive limits that may be placed on orientations given sufficient knowledge of material, nature of the surface, etc. Investigators also have observed the influence of the orientation of discontinuities on the stability of overconsolidated clays. Esu (1966) noted that the failure of such materials upon excavation was controlled directly by the type and orientation of joints. The regularly oriented (sets) and continuous joints were most susceptible to failure.

Anderson and Schuster (1970) found that five of nine failures observed in overconsolidated clays interbedded with basalt flows had failed along oriented fissures. Skempton, Schuster, and Petley (1969) reported the occurrence of two orthogonal joint sets as well as sheet jointing in the London Clay. The latter developed in response to stress relief as with rock masses (Fookes and Denness, 1969). Joint sets in overconsolidated clays which are perpendicular to bedding also have been noted by Esu and Calabrisi (1969). All of these similarities to features found in the sedimentary rocks are to be expected, for with few exceptions, the overconsolidated clays are geologic units interbedded with the geotechnically defined rock units. Fissure orientation relative to surface topography and ice loading has also been observed in glacial till (McGown and others, 1974).

Spacing

The spacing of joints in a rock mass is an important factor in assessing the stability of the mass. Müller and Hofmann (1971) have related spacing to what they term "mobility" of a rock mass (see fig. 3). The smaller the spacing, the more mobile or less strong the rock mass. Voight (1968) has cautioned that where discontinuity spacing is great, the gross behavior of the rock mass will be strongly influenced by the intact rock properties.

N. Barton (1973) has observed the close relationship between joint orientation and spacing in the failure of jointed rock masses and thus, rock mass strength. Deere's RQD concept (1964; Deere and others, 1967) and the other rock mass classification systems based on it testify to the importance of discontinuity spacing on rock-mass strength.

The distribution of spacing distances throughout a rock mass has led investigators to examine the frequency of occurrence (or density or intensity) of jointing. Jointing may be evenly distributed, clustered, or randomly spaced. Priest and Hudson (1976) have prepared a detailed evaluation of these variations in frequency and their influence on calculated values of RQD. Figure 13 illustrates these variations. Earlier, Deere and others (1967) had recognized a linear relationship between RQD and fracture frequency for unweathered rock (fig. 14).

The intensity of jointing is a measure of frequency utilized by Piteau (1971) for the number of joints per unit distance measured normal to the strike of a joint set. Cording and others (1975) have observed that discontinuities in a given set tend to occur in swarms or groups at regular intervals.

The actual measurements taken of joint spacing reveal a wide range of values from a centimeter or less to many meters (Esu and Calabrisi,

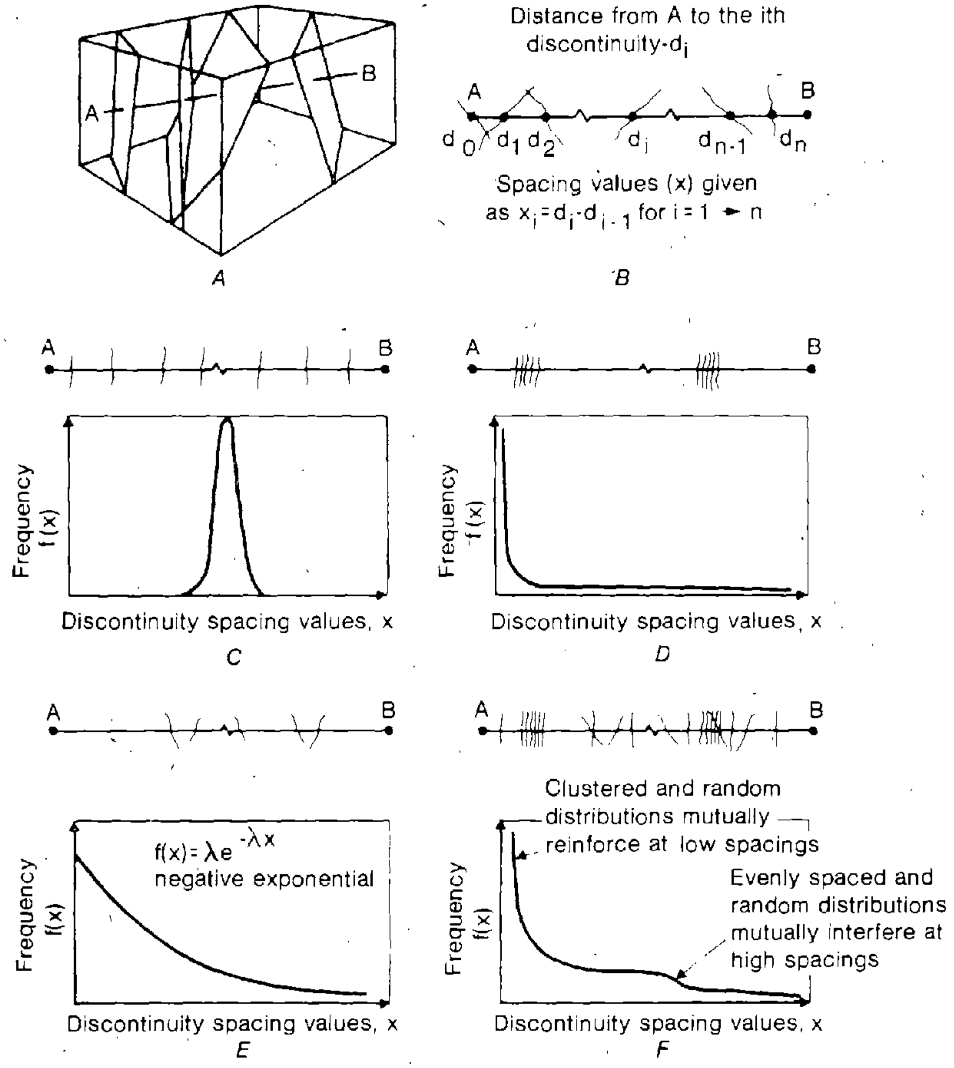


Figure 13. Theoretical discontinuity spacing distribution. A, discontinuity intersection points along a straight line (AB) through the rock mass; B, scanline (measuring tape) on exposed rock face; C, fairly evenly spaced distribution; D, clustered distribution; E, random distribution; F, combination of distributions. Reprinted from Internat. Jour. Rock Mechanics and Mining Sci. and Geomechanics Abs., Vol. 13, S. D. Priest and J. A. Hudson, Discontinuity spacing in rock, 1976.

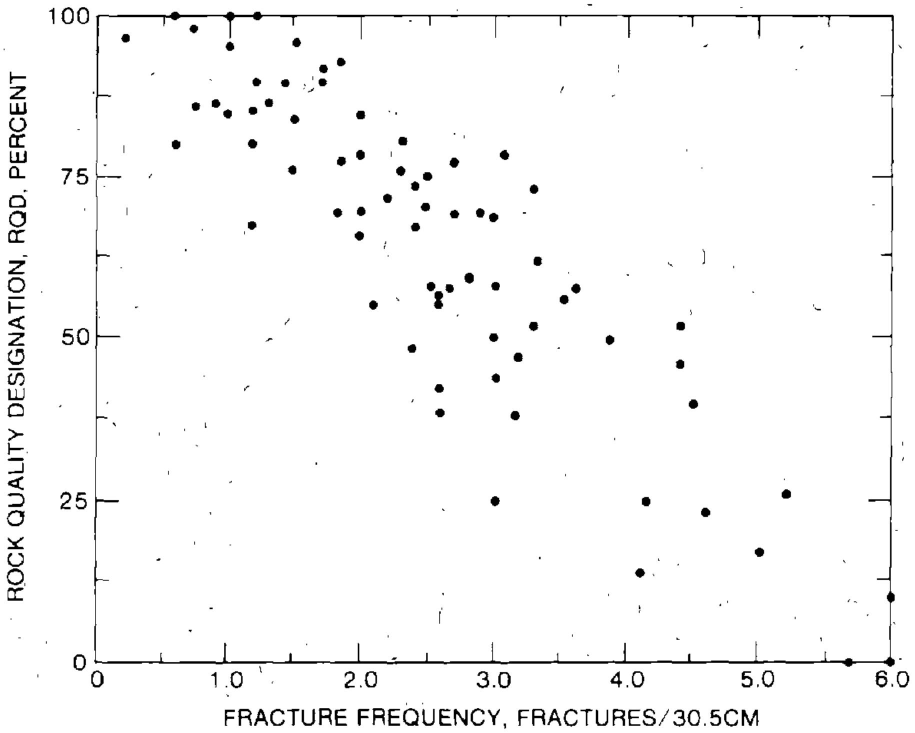


Figure 14. Correlation of fracture frequency and RQD. Modified from D. U. Deere, A. J. Hendron, Jr., F. D. Patton, and E. J. Cording, Design of surface and near-surface construction in rock, Failure and Breakage of Rock, C. Fairhurst, ed., AIME, New York, 1967.

1969; Taylor, 1970; Sherrell, 1971; Gray and others, 1974; Priest and Hudson, 1976). Deere's spacing classification (1964) referred to earlier (table 1) provides a good basis for judging the normal range of spacings for rock masses in general. The distribution of thousands of spacing widths in limestone, sandstone, mudstone, and chalk has been examined by Priest and Hudson. The average spacing width was greatest in sandstone and least in mudstone, with the dominant spacing in all materials measured in centimeters. Others have observed the same decrease in spacing with a decrease in grain size of the clastic sedimentary rocks (Sherrell; Gray and others). Given the same material, Gray, Ferguson, and Hamel have reported that joints resulting only from tectonic stresses as compared to overburden stress, for instance, are spaced more widely.

In the preceding section on orientation, the influence of stress relief on joint development in a variety of materials was discussed. Near-surface increase in frequency was explicitly or implicitly stated in the examples. In an additional example, Snow (1970) observed an average spacing of 1.7 m in near-surface exposures of granite. This average spacing increased to over 10 m at a depth of approximately 100 m in the same material. The same formation will change from a discontinuous mass to a continuum with increasing depth (Goodman, 1976). Thus prior to excavation, joint spacing from all causes can be expected to increase with depth except where influenced by faulting. This is a factor that should influence prediction of joint frequency at depth from measurements taken at surface or near-surface exposures.

Continuity

Although it may appear to be paradoxical, discontinuities are themselves discontinuous through a rock mass (Taylor, 1966). Cording and others (1975) have defined "continuity" as the average length of a discontinuity in a selected direction. The continuity of jointing ranks with orientation, spacing, and surface roughness as one of the primary joint characteristics influencing rock-slope stability as discussed in an earlier section. Piteau (1970, 1971) ranked continuity second only to orientation in importance. Bieniawski (1973) utilized a ranking of continuities ranging from "very good" where not continuous to "very poor" where continuous with gouge filling. For joints oriented in a direction that favors slope failure, continuity obviously reduces the frictional resistance offered to movement by the rock mass (Jennings and Robertson, 1969).

Because joints are typically discontinuous, the development of a continuous sliding surface requires the development of tension or shear failures in the intervening rock portions between joints (Müller and Hofmann, 1971; Piteau, 1971; Goodman, 1972). Thus a jointed rock mass possesses strength from the unfailed portions of the jointed mass which introduces the factor of tensional or shear strength of the rock into stability calculations. The rock blocks bounded by joints will usually not be able to move until failure planes propagate through the intact rock, or irregular step-like failure surfaces develop along adjacent, overlapping, or intersecting joints (Cording and others, 1975). Müller (1964a, b) and Sherrell (1971) have described the development of a similar step-like failure surface by the intersection of joint sets and bedding surfaces.

Some variations of terminology concerning continuity of joints exist in the geotechnical literature. Piteau (1970, 1971, 1972) has made a transition from use of the term "continuity" to "joint size" in his papers. Müller and Hofmann (1971) have used the term "degree of separation" as a measure of the continuity, or conversely, the discontinuity of jointing.

Piteau (1970) has devised a one-dimensional coefficient of joint continuity, k, as a quantitative measure of continuity. The quantity

$$k = \frac{\Sigma a}{\Sigma a + \Sigma b}$$

where a and b as shown in figure 15 are measured distances along the discontinuous joint. Conversely the coefficient of discontinuity for the rock would be

$$(1-k) = \frac{\Sigma b}{\Sigma a + \Sigma b}$$

The intact strength of the rock occupying the distances labeled "b" in figure 15 contributes greatly to the stability of the slope. This has been referred to as "rock bridge" in the geotechnical literature (Müller and Hofmann, 1971; Steffen and Jennings, 1974). In cases where the value of the coefficient of continuity, k, has been conservatively assumed as 1, resulting in a calculated factor of safety less than 1, the cut slope may not fail because of the contribution made by the rock bridge. Steffen and Jennings have noted that stability in such cases has been maintained by the use of the presplitting method so that the rock bridge has not been disturbed by uncontrolled blasting.

Actual measurements of continuity rarely appear in the literature. Bell (1976) has reported joints in metamorphic rocks extending continuously for tens of meters. Skempton, Schuster, and Petley (1969) measured joints as long as 5.5 m in the London Clay. Discontinuity of joints, however, must be assumed when dealing only with surface exposures, drill core, or geophysical data even though long continuous surfaces occur.

Surface characteristics

The nature of the bounding surfaces of a discontinuity plays an important role in the stability of both rock and soil masses as it affects sliding resistance (Patton, 1966a). The strength along a joint is very dependent on the roughness of the joint surface (N. Barton, 1973; Goodman, 1972, 1976). Dodds (1966) stated that the strength that can be mobilized along an irregular surface is dependent on the size of the irregularities, the yield point of the rock composing the

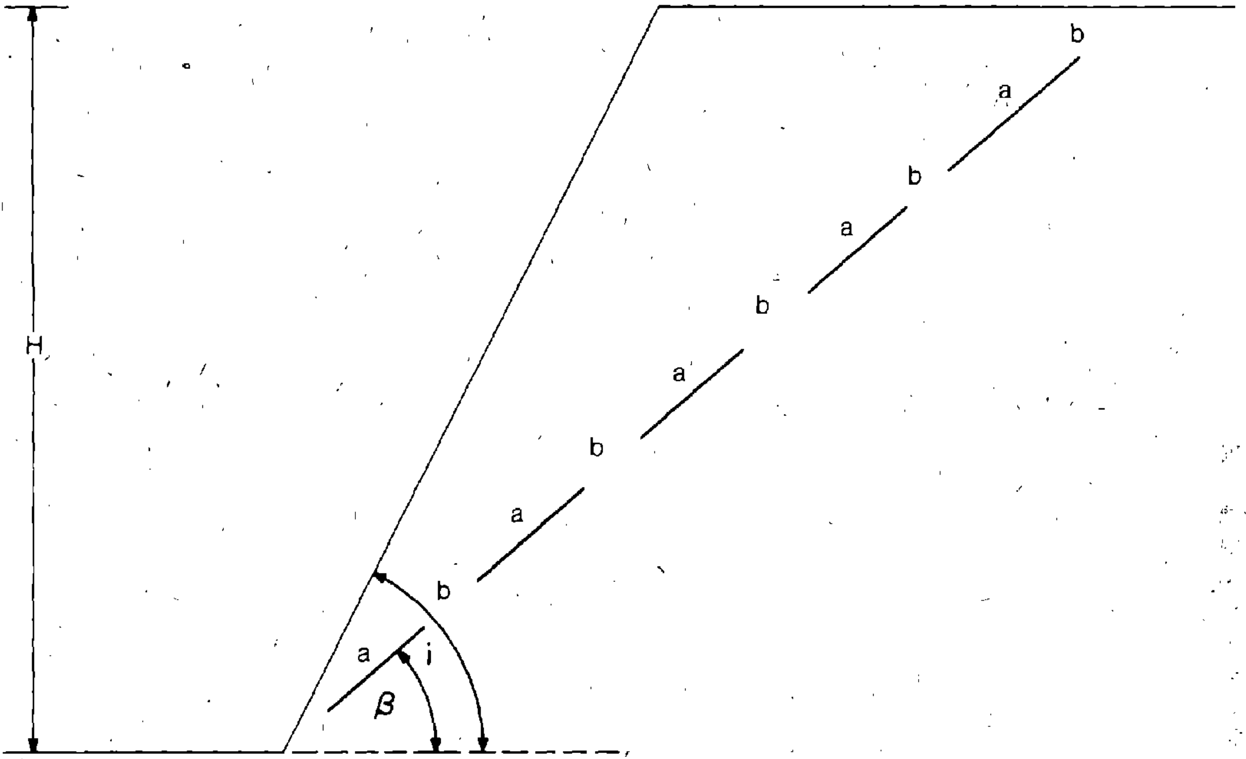


Figure 15. Measurement of coefficient of joint continuity parameters. a, joint segment length; b, distance between joint segments; i, slope angle; β, joint dip. Piteau, 1970a. Courtesy International Association of Engineering Geology from Proc. 1st Int. Cong., IAEG, Paris, 1970; 1, 404 (1970).

irregularities, and the inclination of the irregular surface relative to the movement direction. Cording and Deere (1972) have suggested a relationship between joint properties, including surface characteristics and strength, as follows:

> Low-strength joints--shear zones with gouge, continuous planar joint surfaces, often slickensided. Clay-filled joints.
>
> Medium-strength joints--continuous to semi-continuous joint planes. Surfaces smooth. Occasionally thin gouge or slickensides, but not prominent.
>
> High-strength joints--joints tight, and wavy to irregular. The friction angle, , depends on the angle between the general joint surface and the steepest irregularities. (This relationship will be discussed later in this section.)

Note the similarity of this rating system with the more detailed one of Barton, Lien, and Lunde, (1974) shown earlier on table 15 in the section on classification systems.

Some investigators use the term "character" to define the attributes of a joint relating to the degree of roughness and the presence and type of filling or coating material, if present (Brekke and Howard, 1972). The importance of surface roughness is greater for stronger rocks than for weaker rocks. In the former, more resistance is offered by the asperities before shearing takes place (Patton, 1966a). The nature of irregularities along a failure surface in soil is of little consequence, whereas it may determine the difference between stability or failure in rock (Patton and Deere, 1971).

All kinds of discontinuities have surface features that affect rock-mass strength (Taylor, 1966; Skempton and others, 1969; Patton and Deere, 1971; Sherrell, 1971). Their surfaces have been described as smooth, planar, undulating, slickensided, rough, irregular, and wavy (Taylor, 1966; Knill, 1968; Piteau, 1970; Sherrell, 1971). Deere (1964) proposed a qualitative classification for such terms, utilizing degree of planeness for the geometric features (plane, curved, irregular) and degree of smoothness for the surface irregularities (slick, smooth, rough). Patton (1966a), in his classic study of discontinuous surfaces, defined two classes of surface irregularities based on their magnitude (figure 16). In the geotechnical literature (Piteau, 1970; Goodman, 1976) the first-order irregularities are now called waviness, and the second-order, roughness.

Of the two factors, waviness and roughness, waviness, or first-order irregularity, has the greater effect on slope stability (Patton, 1966a; Deere and others, 1967). The asperities that constitute roughness, or second-order irregularities, may be sheared off, providing a smoother surface, whereas for two adjacent blocks to move over a wavy, or first-order surface, there must be relative displacement or dilation normal to the surface for the opposing wavy sides to move parallel to the surface (Piteau, 1970).

Given the same normal stress on the surface, the controlling factor will be the angle i, or waviness angle (Call and others, 1976), shown in figure 16b. This may be seen in terms of the factor safety (FS) of a block on an inclined surface (fig. 17a), where

$$FS = \frac{W \cos\beta \tan\phi}{W \sin\beta} = \frac{\tan\phi}{\tan\beta}$$

β = slope angle, W = weight, and ϕ = the angle of sliding friction. For a FS = 1, $\tan\phi = \tan\beta$, or the friction angle equals the discontinuity slope angle. If there is an irregular surface, as in figure 18, the average angle i of the first-order irregularities, or roughness angle, is subtracted from β so that the resulting angle, β-i, effectively increases the factor of safety. Some investigators add the angle i to ϕ, accomplishing the same thing. From this application we can see that the slope angle, β, may be increased until $\beta = \phi$+i before the instability threshold is reached (Cording and others, 1975). The increase in the friction angle, ϕ+i, which results in the equivalent angle of friction, or shearing resistance (Patton and Deere, 1971), is due in part to the dilation that occurs in the sliding mode (Rengers, 1970). This dilation has been shown by Hoek and Bray (1974) in figure 19. The effect of irregularities on shearing resistance may be different in opposite directions on the same surface, owing to the asymmetry or orientation of the irregularities. Patton and Deere reported that the equivalent friction angle may vary by 15 or more, depending on the steepness or gentleness of the angle i in the direction of movement. In either case, the strength along a joint surface increases above a minimum value as the angle i increases (Patton, 1966a).

More recently, Barton (1976) and Goodman (1976) stated that the increase in strength with an increase in the angle i occurs only at low normal stresses at which dilation can occur easily. At higher normal stresses, the strength of the wallrock asperities controls the joint shear strength. This is the result of the high normal stress reducing or prohibiting the dilatant displacement caused by the upper block moving or riding up on the irregular surface, as shown in figure 20 (Barton, 1971; Goodman, 1974; Hoek and Bray, 1974). The normal stress-shear stress relationships along a smooth surface and an irregular surface in dilation and asperity shearing modes are illustrated in this figure (Hoek and Bray, 1974).

The mean roughness angle i may be obtained by measuring many dips on the surface of a joint and using the mean value (Goodman, 1976). Earlier, Goodman (1974) had proposed plotting the poles of these

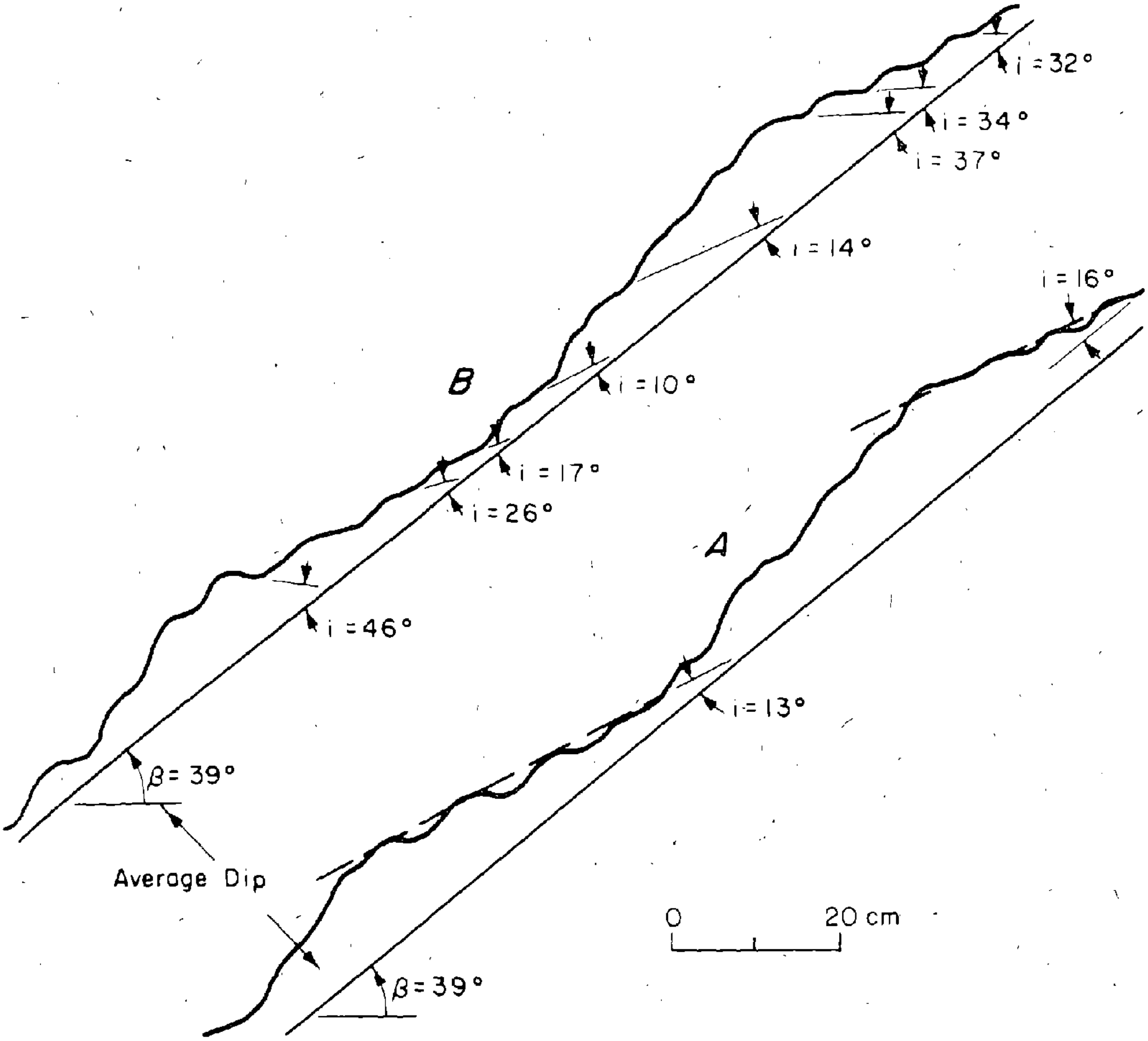


Figure 16. Irregularities on a discontinuity. A, first order; B, second order. Modified from D. U. Deere, A. J. Hendron, Jr., F. D. Patton, and E. J. Cording, Design of surface and near-surface construction in rock, Failure and Breakage of Rock, C. Fairhurst, ed., AIME, New York, 1967.

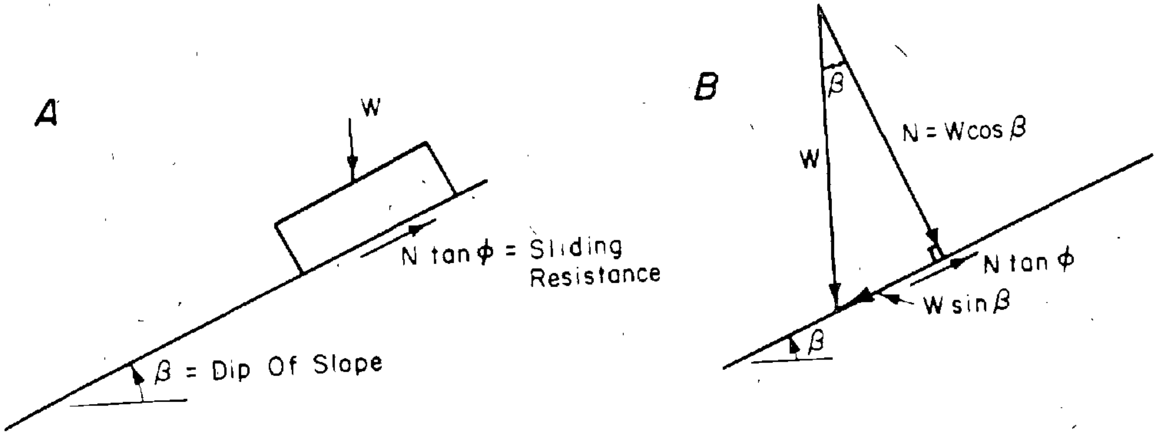


Figure 17. Forces acting on a block resting on a relatively smooth inclined surface. A, sliding block on inclined plane; B, force diagram for part A; N, normal stress; ϕ, friction angle. From D. U. Deere, A. J. Hendron, Jr., F. D. Patton, and E. J. Cording, Design of surface and near-surface construction in rock, Failure and Breakage of Rock, C. Fairhurst, ed., AIME, New York, 1967.

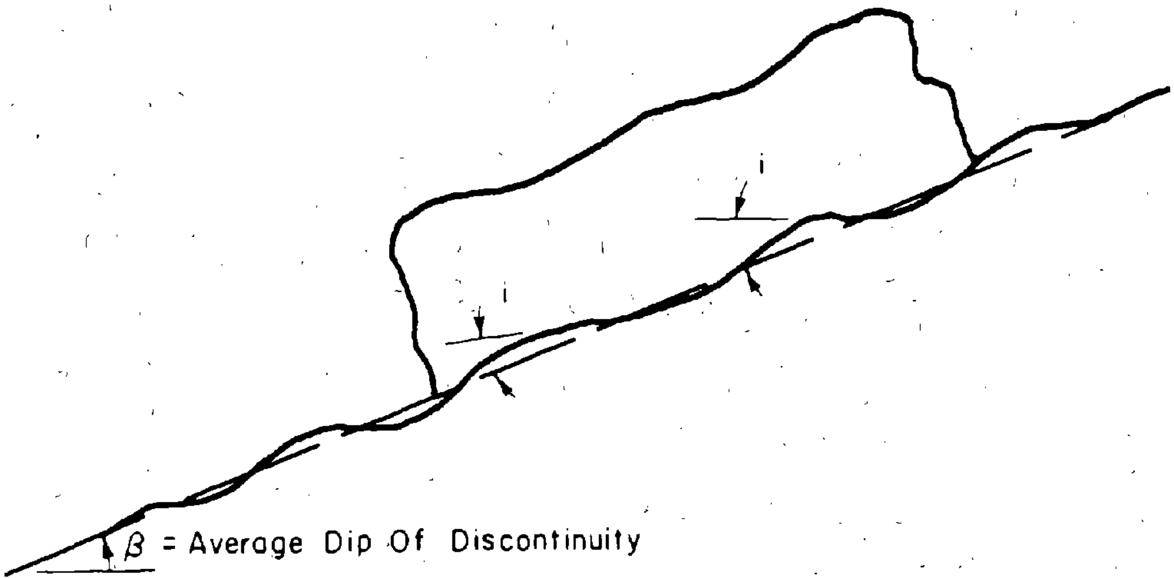


Figure 18. Sketch of a rock mass resting on an inclined irregular discontinuity. From D. U. Deere, A. J. Hendron, Jr., F. D. Patton, and E. J. Cording, Design of surface and near-surface construction in rock, Failure and Breakage of Rock, C. Fairhurst, ed., AIME, New York, 1967.

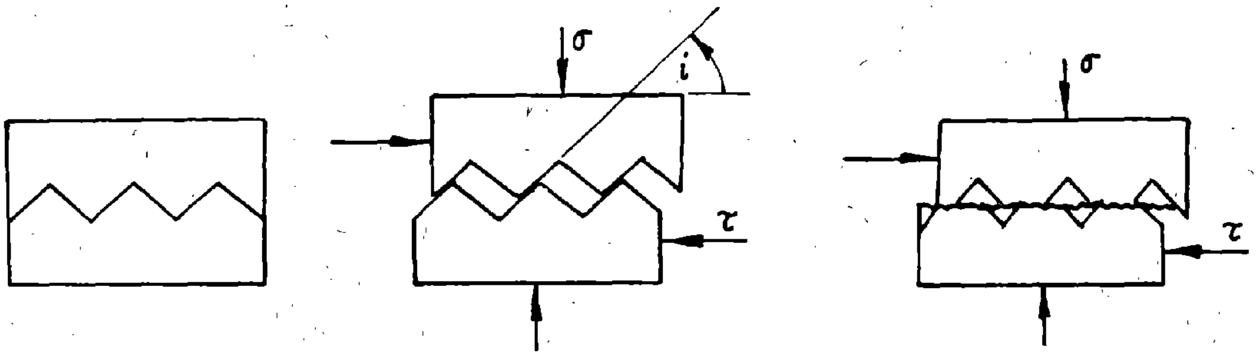


Figure 19. Dilation and asperity shearing on irregular
τ, shear stress; σ, normal stress; i, angle of inclination
of surface to horizontal. From E. Hoek and J. Bray,
Rock Slope Engineering, Institution of Mining and Metallurgy,
London, 1974.

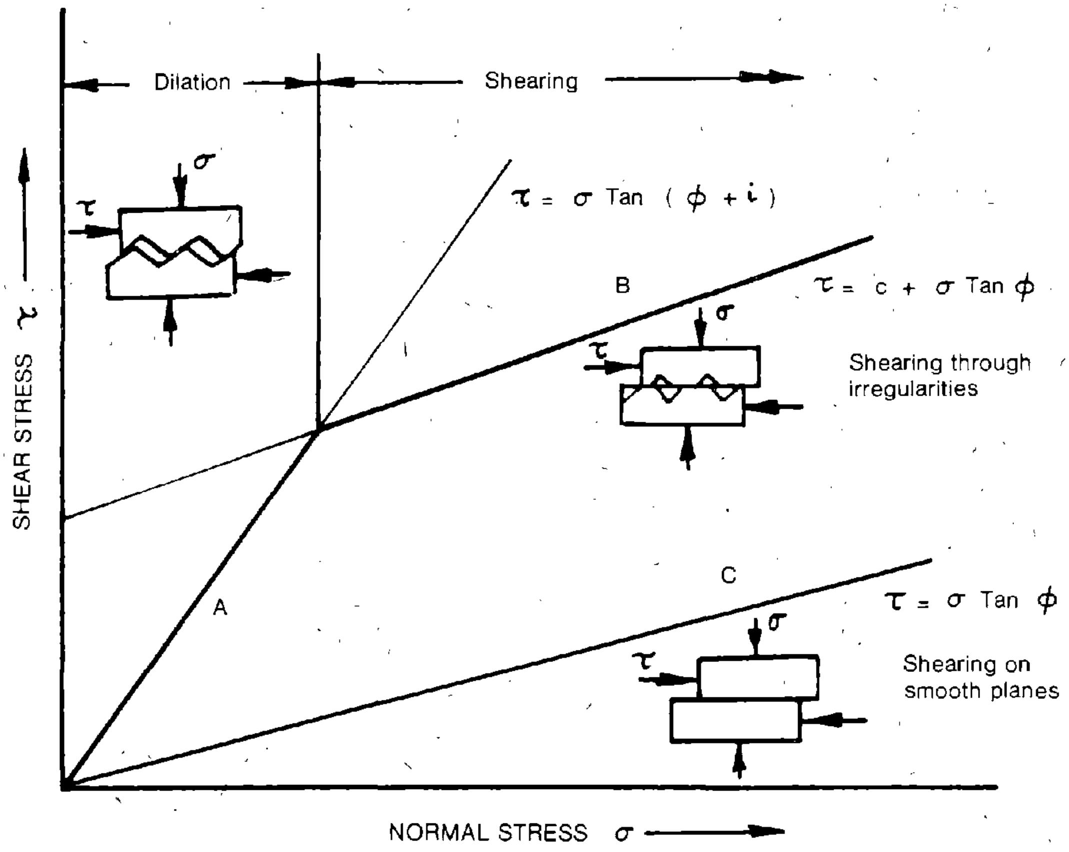


Figure 20. Simplified relationships between shear strength and normal stress for rough surfaces. ϕ, friction angle; i, inclination of surface irregularity to plane surface. From E. Hoek and J. Bray, Rock Slope Engineering, Institution of Mining and Metallurgy, London, 1974.

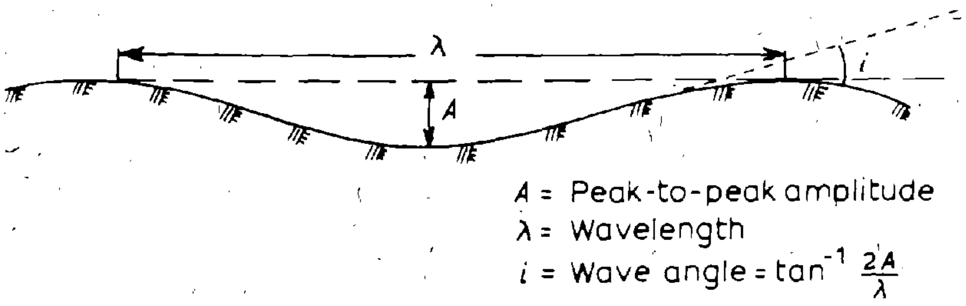


Figure 21. Joint waviness measures. From Principles of Engineering Geology, P. B. Attewell and I. W. Farmer, Chapman and Hall Ltd., Publishers, 1976.

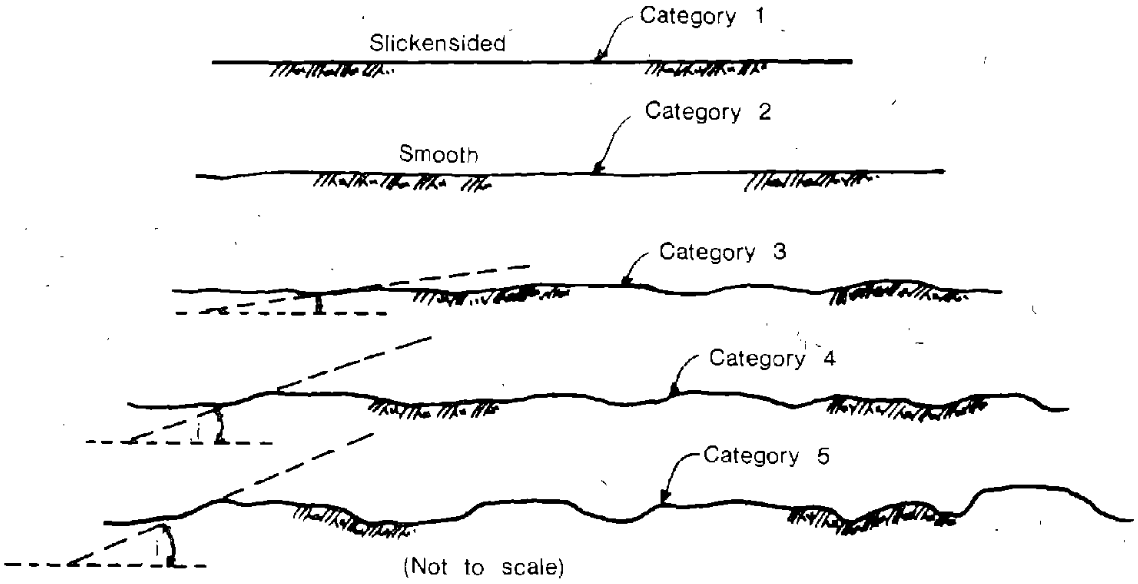


Figure 22. Illustration of relative roughness. Modified from Planning Open Pit Mines, Proceedings of the Symposium on the Theoretical Background to the Planning of Open Pit Mines with Special Reference to Slope Stability, Johannesburg, 1970. Published 1971 for The South African Institute of Mining and Metallurgy by A. A. Balkema, Cape Town.

surfaces on a stereonet and defining roughness from the scatter of the plotted poles. The graphical measurement of angle i is shown in figure 21.

Waviness amplitudes as great as 3 m have been recorded on previously failed surfaces (Stout, 1971). Amplitudes of 0.6 m or less probably are more in order for joint surfaces (Hamel, 1974). Piteau (1971) has proposed five qualitative categories of roughness (or waviness) based on increases in the roughness angle i or λ as shown in figure 22. Cording and others (1975) have divided waviness (first-order irregularity) into four classes based on actual measurements of i as shown in table 19. They place a range of 0.25-10.0 m for the wave lengths for such features.

Table 19. Joint waviness classification.
Cording and others, 1975.

Angle i	Waviness class
0°- 5°	Planar
5°-10°	Slightly wavy
10°-20°	Wavy
>20°	Very wavy

Measurement of waviness and roughness may be conducted by mechanical, optical and photogrammetric means in addition to the physical measurement of dips on an exposed surface mentioned earlier. Rengers (1970) has used a mechanical profilograph to measure amplitudes on joint surfaces up to 2 m in size. For smaller amplitudes on hand specimens a stereodepth measuring microscope was used. This utilized a floating dot measuring system and the horizontal and vertical movements were recorded on an xy plotter.

Barton (1971), Wickens and Barton (1971), and Hoek and Bray (1974) have described a stereophotogrammetric method using a phototheodolite. Stereoscopic pairs of photographs are taken of surface exposures from two distances for examination of large- and small-scale surface features. Ross-Brown, Wickens, and Markland (1973) have developed a terrestrial photogrammetric procedure using a phototheodolite. The attitudes of planes fitted to the surfaces of a stereoscopic model are measured with a clinometer in the field. The method gives a measure of waviness.

Coulson (1972) measured surface roughness with a Brush Surfanalyzer 1200 System. He obtained "true" profiles and arithmetic average roughness values. Hoek and Bray (1974) have recommended the measurement of first- and second-order roughness by using 42 cm and 5.5 cm diameter plates placed on the large- and small-scale roughnesses. A geological compass is used for measuring the inclination angles.

The degree of roughness or second-order irregularity that develops on a joint is in part a function of the origin of the joint. Generally there are two ways in which the joint forms aside from causative agents such as relief of tectonic stresses, desiccation stresses, and overburden stresses. These are by extension or tension on the mass and shearing of adjacent blocks.

Rupturing by tension creates characteristically rough and clean surfaces. Shearing results in smooth surfaces with varying degrees of detritus from the shearing action (Brekke and Howard, 1972). The fracture surfaces resulting from tension also tend to have appreciable waviness whereas those generated by shearing will tend to be more planar (N. Barton, 1973). Hodgson (1961) has described the second-order roughness patterns caused by tension fracturing as plume-like or radial in appearance.

Slickensides on the surfaces of fissures in overconsolidated clays long have been assumed to have resulted from shearing. While shearing may cause such surface features in this material, a common cause now is considered to be from volume change without well-defined shearing through the mass (Skempton, 1964; Fookes and Wilson, 1966; Smith and others, 1967; Focht and Sullivan, 1969). Smith, Albee, and Jahns (unpublished data, 1967) have related the occurrence of the shiny surfaces to texture, grain size, composition, and amount of montmorillonite present in the material.

Weathering or change in physical and chemical properties of a material can significantly influence the strength along a joint that otherwise is controlled by surface roughness and strength of wall rock asperities. Skempton (1964) and Skempton, Schuster, and Petley, (1969) have described strength loss from softening of stiff fissured clays from water movement along joints and fissures. N. Barton (1973) has written that the shear strength of weathered rock joints will be lower than those with the same roughness in the same kind of rock due to the reduction in compressive strength of the altered rock. The depth of weathering into joint walls will depend on the composition and permeability of the wall rock assuming the presence of water (N. Barton).

As noted in an early section, an old slip or failure surface constitutes as viable a discontinuity as those normally described in the literature because of its influence on slope stability (Gray and others, 1974). It is appropriate at this point to examine some of the features of these important surfaces.

As seen in the field, failure surfaces exhibit a wide range of thicknesses. Oliveira (1972) reported a well-defined slickensided surface in a plastic organic clay layer. Failure surfaces in a clay-rich glacial till were described by Nasmith (1964) to be from several millimeters to 2.5 cm thick. Reported thicknesses of failure surfaces in clay-rich colluvium range up to 5 cm (Hamel and Flint, 1972; Gray and others, 1974). The failure-prone sensitive clays in Quebec have exhibited surfaces from several millimeters to no more than 2.5 cm thick (Conlon, 1966; Lo, 1972). The measured thicknesses of failure surfaces in the stiff, fissured clays range from a few microns to several centimeters (Morgenstern and Tchalenko, 1967; Broms, 1975). Similar measured thicknesses in failed glacial lake clays range from sharp interfaces to failure zones up to 1.8 m thick (Haug and others, 1977).

In rock masses the slip or failure surface may become a zone of failure. Failures in southern California range from paper-thin seams to shear zones "many feet thick" (Leighton, 1966). Shear-zone thicknesses in sedimentary rocks have been measured from several centimeters to about a meter thick (Dodds, 1966; Wahlstrom and Nichols, 1969). The material in the zone is composed of crushed rock material, with composition and size range depending on the rock types involved and the magnitude of the movement. Crushing of the rock will occur to a much greater extent between walls of rough joints or faults than where there are smooth, planar surfaces (Barton, 1976). Bell (1976) has reported a failure zone in schist 10-25 cm thick, consisting of clay, mica, and crushed schist. Deere and others (1967) have emphasized the importance of recognizing the presence of crushed or mylonitic zones from earlier failures when examining an area. It is not uncommon to observe the failure zone along or parallel to the bedding or foliation surfaces in sedimentary and metamorphic rocks (Fookes and Wilson, 1966; Leighton; Bell).

Minerals and rock fragments may exhibit a high degree of orientation in and adjacent to a failure surface. Hamel and Flint (1972) have noted that platy claystone and shale fragments are commonly alined parallel with the failure movement. Henkel and Yudhbir (1966) have reported the same orientation in clay shales. In a silty clay shear zone, Morgenstern and Tchalenko (1967) have described bands of strongly oriented shear material bounding the 15-mm-wide shear. The same bounding bands of oriented clay have been optically observed and measured to be 20-30 m thick by Early and Skempton (1972). Bell (1976) has described the orientation of clays and micas in a failure zone in schist.

There is also a tendency to mechanically concentrate finer materials, especially the clay-sized fraction, within failure zones (Brekke and Selmer-Olsen, 1965; Wahlstrom and Nichols, 1969; Early and Skempton, 1972). The mechanism has been described by Wahlstrom and Nichols, and is illustrated in figure 23.

Both physical and chemical changes may occur in conjunction with a failure surface or zone. Skempton (1964) has described a 2.5 cm-wide zone of softened clay on either side of a slip surface in the London Clay. In association with a failure surface in clay-rich colluvium, Early and Skempton (1972) have observed water-induced chemical changes which have oxidized the normally gray clay into several millimeter-thick

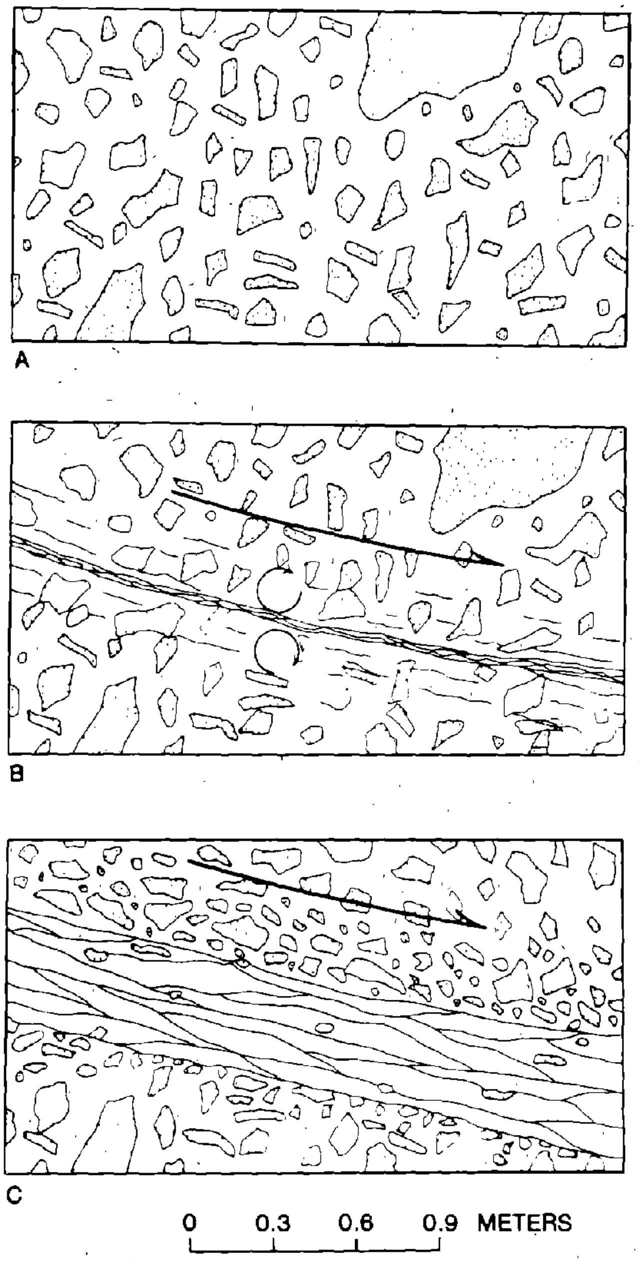


Figure 23. Stages of shear zone development. A, prior to shear; B, initiation of shear; C, late stage of shearing; arrows show direction of movement. Modified from The morphology and and chronology of a landslide near Dillon Dam, Dillon, Colorado, E. E. Wahlstrom and T. C. Nichols, Jr., Engineering Geology, Vol. 3, no. 2, 1969.

zones of yellow-brown clay. Bieniawski (1973), in his engineering classification of jointed rock masses, considered the influence of weathering of wallrock along joints. The joint alteration factor used by Barton, Lien, and Lunde (1974) in their rock-mass strength classification also is dependent on the degree of alteration along joint surfaces (see table 17).

The geometry of failure surfaces may be quite complex, even though published generalizations would indicate otherwise (Broms, 1975). The greatest complexity arises when failures occur in rock masses where they are controlled by discontinuity orientations. "S"-shaped failure surfaces in profile have been described in sedimentary rocks by Broili (1967) and Stout (1971). While it is normally assumed that a failure surface is continuous regardless of its geometry, Cooksley (1964), Dodds (1966), and Wahlstrom and Nichols (1969) have all described failure surfaces that are multiple discontinuous surfaces. The nature of the material in which failure has occurred may preclude identification of the slip surface. An example is the Parson's Landing slide on Santa Catalina Island off the California coast. The slip surface could not be located in the intimately sheared and foliated rocks of the Franciscan assemblage in which the slide had occurred (Slosson and Cilweck, 1966).

Added to the many unknown or variable factors associated with failure surface features is the problem of depth. While depths of 200 m or more have been reported (Lo and others, 1972), most will be much less than this figure. Henkel and Skempton (1955) have concluded that shallow, nonrotational slides in clay shales have wide occurrence. Prior to obtaining more definitive data from drilling or geophysics, the depths to unexposed surfaces can be estimated using the failure mode and size of the failed mass.

The development of a failure surface affects a slope in at least two ways. One is the introduction of a typically continuous surface along which failure took place. The second is the strength reduction from peak strength before modification of first- and second-order surface irregularities to residual strength. A further reduction in strength occurs with the presence of the crushed zones that are commonly formed in rock-mass failure. The geotechnical literature abounds with such evidence in all materials, as indicated by the following selected references: Terzaghi (1936), Skempton (1964), Fookes and Wilson (1966), Patton (1966a), Deere and others (1967), Jennings and Robertson (1969), Skempton, Schuster, and Petley (1969), Piteau (1970, 1971), Rengers (1970), Goodman (1972), Gray, Ferguson, and Hamel (1974), and Cording and others (1975).

Joint separation and associated filling materials

As discussed in the preceding section, the separation between joint surfaces and the presence of filling material may have a profound influence on the strength of a jointed rock mass. They are so closely associated with surface roughness that together they constitute the character of a joint (Brekke and Howard, 1972). However, they exert sufficient influence on rock-mass strength to be considered separately. It should be noted that occasionally joint separation is referred to as joint tightness in the literature (Hall and others, 1974).

Joints may occur in which the surfaces are separated with no filling, partial filling, or complete filling of the space. The filling material may be clay, sand, or coarse fragmental material with or without the finer fractions, either deposited or the result of faulting. The filling material may also be a mineral precipitate which effectively heals the joint with material that may be as strong or stronger than the wallrock (Tulinov and Molokov, 1971). Weathering or alteration of the wallrock surfaces also can be a source of filling material. Some clays and sands may wash or squeeze out under changing natural conditions or during construction, altering the joint strength (Tulinov and Molokov; Brekke and Howard, 1972).

The separation of joint walls may result from the tensile stresses that created the joints, solution widening of joints, and shearing movements which can separate the surfaces through the generation of gouge or filling material, as well as separate joint surfaces along wavy surfaces, as described in the preceding section. Separation has entered into the rock-mass strength classifications of Bieniawski (1973) and Barton, Lien, and Lunde (1974).

Open or unfilled joints within a rock mass commonly are a product of differential movement and dilatant displacement along an irregular surface, as discussed in the preceding section. Though highly variable in amount of opening (Goodman, 1974), they provide access by water to the rock mass with resulting deposition of material in the joints and weathering of wallrock. Goodman (1976) has written that the strength along such a surface as compared with that along a mated joint surface is as different as gravel is from rock.

Joints filled with clay, silt, and sand will have reduced shear strengths. Clay fillings can result in extremely low shear strength (Goodman, 1969). Cording and others (1975) have reported friction angles of 8°-15° where shearing has occurred and joint filling consisted of clay. This may be compared with angles of 22°-35° for sheared unfilled joints. They stated that the residual shear strength of joint filling material is a function of plasticity and grain size; the finer the material and higher the plasticity index (PI), the lower the residual angle of friction. Patton and Deere (1971) have reported clay filling material with a PI of 61 percent and a liquid limit (LL) of 103 percent. Montmorillonite clay contributes to such high values. Moisture content is the major factor in determining the strength of the filling material (Brekke and Selmer-Olsen, 1965).

A rating of joint character by Brekke and Howard (1972) is based on the presence or absence of filling material. It ranges from healed joints and clean joints with no filling or coating to clay and sand fillings subject to swelling and/or washing. They cautioned that the characteristics of the filling or gouge material are not uniform along a joint, nor would they be entirely a function of wallrock composition, because the material may be added from other sources.

The thickness of a filled joint will influence the shear strength along the joint. If the filled separation is less than the size of the asperities, the strength is influenced by the strength of the asperity, size, and degree of roughness, or roughness amplitude (Piteau, 1971; Tulinov and Molokov, 1971; Goodman, 1976). If the filling is so thick that there is no rock contact, the friction properties of the joint are those of the filling material alone, and failure will follow soil mechanics criteria (Brekke and Selmer-Olsen, 1965; Piteau, 1970, 1971, 1972; Rengers, 1970; Goodman, 1976). A comparison of these conditions and associated strengths is shown in figure 24, from Hoek and Bray (1974).

Progressive failure and jointing

The presence of fissures and joints in a rock mass may lead to progressive failure of the mass. Such failure is the progressive extension of a rupture or failure surface (joint or fissure) until the rupture has propagated to an exterior surface of the rock (Terzaghi, 1962). Very large tensile stresses are concentrated at the ends of fractures, so that once formed they are self propagating (Price, 1959; Skempton, 1964). Skempton, Bjerrum (1967), Fleming, Spencer, and Banks (1970), and Hooper (1970) are among those investigators who have noted the common occurrence of progressive failure in stiff fissured clays and clay shales.

Terzaghi (1962), Broili (1966), and Deere and others (1967) have addressed the occurrence of progressive failure in hard jointed rock masses. Terzaghi and Broili have emphasized the role that intersecting sets of joints play in the propagation of failure. Broili and Deere and others, have pointed out the influence that stress relief has on initiating and continuing jointing by the associated increase in tensile stress. This influence has special significance where construction rapidly exposes rock with high residual stresses.

Joint mapping problems

The measurement of joint spacing and frequency on exposed rock surfaces is subject to observational error. This error or bias is introduced by the orientation of the jointing relative to the exposed surface and the size of the exposure (Terzaghi, 1965; Deere and others, 1967; Steffen and Jennings, 1974; Attewell and Farmer, 1976; Call and others, 1976; Kohlbeck and Scheidegger, 1977). Regardless of the size of an exposure, a universal problem is that the joints that intersect the surface nearly perpendicularly are counted more frequently than

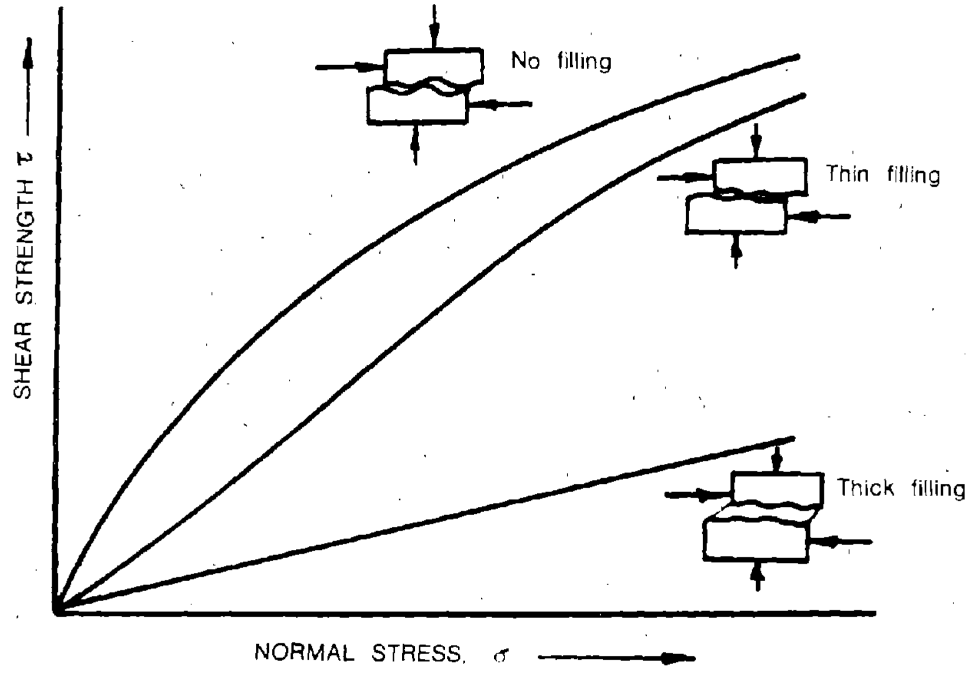


Figure 24. Relationship between shear strength and normal stress for discontinuities with different thicknesses of gouge infilling. From E. Hoek and J. Bray, Rock Slope Engineering, Institution of Mining and Metallurgy, London, 1974.

those that range to the parallel limiting case. Kohlbeck and Scheidegger have written that the natural tendency is to place greater importance on the joints that appear most frequently.

The problem is not limited to exposures, as drill-core orientation can create the same problem (Franklin and others, 1971). Hall, Newmark, and Hendron (1974) have reported RQD values of 63 percent for joint measurements taken across a joint system and 85 percent for those parallel to the system from exposures in mine walls for the same rock mass. The average value of 73 percent compared well with the 75 percent obtained from drill cores oriented perpendicular to the predominant joint orientation. Obviously, any rock-mass strength classification that employs RQD or similar spacing and frequency data will be influenced by this bias. Stability analyses of rock blocks bounded in part by less easily observed joints also would be affected.

McMahon (1968a), Robertson (1971), Attewell and Farmer (1976), and Kohlbeck and Scheidegger (1977) have proposed systems for recording and analyzing joints of different orientations relative to a surface. Call, Savely, and Nicholas (1976) suggested the numbers of readings necessary for statistically adequate samples of joints for various surface exposures. These investigators also added a caution about extrapolating surface orientations to greater depths because of variations in structural trends. This would also apply to stress-relief jointing which is responsive to surface topography and depth.

All of the described characteristics of jointing influence the selection and use of indirect methods of exploration for them. Methods employed must be sensitive to variations in composition of the materials involved as well as all of the physical variables described associated with orientation, spacing, continuity, surface characteristics, and filling materials. In addition, there are scale differences that limit the recording of data over a wide range of conditions. An example would be the relative ease of recognition of and determination of the orientation of a joint surface as compared to the mapping of surface roughness on the same surface.

Bedding

General

Bedding or stratification of rocks is primarily a feature of sedimentary rocks, although multiple lava flows and pyroclastics such as ash and welded tuff may occur as layered units. For the purpose of this discussion, only bedding related to a sedimentary environment and origin will be considered.

Thickness and continuity

Depending on the depositional environment, sedimentary rocks may be very uniform in thickness over large areas or they may change in thickness over short distances, making complex, lenslike units (Krumbein and Sloss, 1963; Pettijohn, 1975). The bedding surfaces separating the beds would be planar to curving.

With the wide range of thickness observed in sedimentary rocks, it is only natural to expect a number of classification systems with associated names for the various thicknesses. A summary of five such classifications is shown in table 20 (Berkman and Ryall, 1976).

Table 20. Bedding thickness terminology.
Berkman and Ryall, 1976. Reproduced
with permission from the Australasian Institute of
Mining and Metallurgy
[Leaders (---), no data in that publication]

Bedding Term	Splitting Term	Average Thickness of Beds/Splits (mm) Anon. (1971)	Anon. (1960)	Kelley (1956)	McKee & Weir (1953)	Payne (1942)
Laminated	Fissile	<10	---	---	---	<2
Shaly	---	---	---	---	---	2-10
Very thinly bedded	Flaggy	10-30	---	---	10- 50	10-100
Thinly bedded	Flaggy	30-100	<900	---	50-600	---
Medium bedded	Slabby	100-300	---	---	600	---
Thickly bedded	Blocky	300-1000	>900	---	600-1200	---
Very thickly bedded	Massive	>1000	---	>1800	>1200	>100

Surface features

Bedding surfaces exhibit a variety of forms, all of which affect the shear strength of the surface in terms of roughness. These forms are commonly referred to as sedimentary structures, and they may be of primary or secondary origin with respect to origin at the time of deposition or at some later time (Pettijohn, 1975).

The primary structures on bedding surfaces that most affect roughness are ripple marks and a variety of features grouped under the term "sole marks." Ripple marks are the most common of these. They are caused by wave and stream action and by wind, and may be symmetrical or asymmetrical in cross section (Pettijohn, 1975). In plan view, ripple marks may be quite linear where wave or current movement has been oscillatory or unidirectional in motion. Where there have been cross currents, the ripple marks form intersecting ridges and depressions rather than troughs. The resulting features are called interference ripple marks (Pettijohn, 1975). In either case, the ripple mark molds and the overlying casts interlock along the surface.

Sole marks are found on the undersides of some sandstone beds and, less commonly, limestones that overlie shale. On the underside of a sandstone bed there are raised structures which have resulted from the filling of depressions in the mud surfaces on which the sand was deposited (Pettijohn, 1975). The origin of such features may be water currents, deformation at the bedding interface from the overlying sediment load, and organisms. The current-caused features range from elongate, moundlike forms with one bulbous end opposite a flaring or merging end (flute casts) to linear, rounded to sharp-edged features (groove casts). The former are caused by current scour of the mud and the latter by shells, sand grains, and similar objects moved across the mud by the current. The moundlike forms are the larger of the two extremes, ranging up to several centimeters in height and to more than a meter in length.

Deformation from loading at the mud-sand interface can result in downward bulbous protrusions into the underlying soft mud, called loadcasts. They lack the current orientation and symmetry of the preceding features, and are not casts of current-formed depressions.

Surface markings may also be found on the bedding surfaces within a sandstone unit rather than in contact with shale. These commonly result from wave action in the beach or littoral zone environment. Other complex bedding features may be formed by contemporaneous slumping of sediments on a gentle slope during accumulation of material.

Although there are a number of secondary structures, as in the case of primary structures, only a few may be of importance concerning bedding-plane roughness. Of these, only stylolites occur commonly enough to be considered here. Stylolites are a jagged or interlocking interface between two layers of limestone or dolomite. Insoluble material such as clay often is found at the interface, leading to the conclusion that stylolites originate by solution along a bedding surface.

Organisms may cause irregularities on bedding surfaces. The most common sources of roughness are fossil shells, their casts, molds, and tracks, and trails of unattached bottom-dwelling invertebrates.

In all the preceding structures, the resultant roughness may influence slope stability by increasing frictional resistance to sliding. For those structures that involve shale, failure might occur in the shale, bypassing the restraining influence of the roughness. Photographs of the structures described and others may be seen in publications by Pettijohn and Potter (1964) and Pettijohn (1975). The latter reference is recommended reading for details of the structures.

Geotechnical aspects

Although references to failure along bedding surfaces are common in the geotechnical literature, descriptions of the surface roughness are not. Sherrell (1971) has described bedding planes in mudstone and siltstone as being essentially planar, smooth, and frequently slickensided. By comparison, he described sandstone surfaces as far less smooth, especially where loadcasts projected into the underlying mudstone. Skempton, Schuster, and Petley (1969) referred to bedding surfaces in the London Clay as somewhat rough with a bumpy texture. Deere (1964) considered the naturally occurring roughness on bedding surfaces in his degree of roughness rating of slick, smooth, and rough.

Slickensides on bedding surfaces of sedimentary rock units that have been folded are common, especially in the more competent (or least deformable) rocks. They have been factors in failures reported by Sherrell (1971) and Cruden and Krahn (1973). The slickensides have been formed by differential movement along bedding surfaces during folding.

Slope failures along bedding surfaces in interbedded shale, sandstone, and related mudstone, claystone, and siltstone are common, as reported by Smith and Cedergren (1963), Taylor (1970), Sherrell (1971), Eigenbrod and Morgenstern (1972), Hamel (1972), M. Barton (1973), and Voight (1974). Seams of coal and bentonite may have added to the failure reported by Eigenbrod and Morgenstern. Failures along bentonite layers have been described from the Pierre Shale by Erskine (1973).

Limestone beds are the sites of failure either at the contact with overlying sandstones and shales (Nonveiler and Suklje, 1955), interbedded clays (Henkel, 1961), or along bedding within the limestone (Muller, 1964b; Broili, 1967; Cruden and Krahn, 1973; Cruden, 1976).

Less commonly reported are failures along bedding surfaces in quartzite (Rawlings, 1968; Cruden, 1976). In all cases of failures along bedding surfaces noted, the bedding surface has been inclined in the direction of failure and noted as daylighting in the slope in many of them.

Although glacial deposits are not classically thought of as having bedding surfaces, Wilson and Johnson (1964) have reported movement along horizontal bedding planes in highly overconsolidated glacial clays upon relief of lateral pressure. More to be expected are failures between glacial till and underlying shale bedrock, reported by Christensen and Lohnes (1973), and between till and overlying soils, described by Bishop and Stevens (1964) and Conlon, Tanner, and Coldwell (1971).

Foliation

General

As discussed in earlier sections on kinds of discontinuities and problems in terminology, foliation is common and restricted to metamorphic rocks. It also is referred to as schistosity, gneissosity, and slaty cleavage, all of which are varieties of foliation. Foliation here will refer to planar orientation of platy and elongate minerals either through reorientation of minerals or through recrystallization of metamorphic minerals under heat and/or stress.

In contrast to the large range of thickness possible between bedding in sedimentary rocks, foliation where present in metamorphic rocks is normally closely spaced (Taylor, 1966). Also in contrast to bedding surfaces, foliated surfaces may undulate with shorter radii of curvature than the lenslike bedding in some sediments. The surfaces are typically smoother than found along bedding surfaces because of platy mineral orientation.

The orientation of minerals within a foliated rock mass has considerable influence on strength of the rock. Deklotz, Brown, and Stemler (1966) have reported shear strength differences in schistose gneiss of as much as 50 percent, depending upon orientation of principal stress relative to foliation, the foliation plane being the weakest failure surface.

Geotechnical aspects

Slope failures in which foliation in schist was the controlling or contributing discontinuity factor have been reported by Benson (1946), Hadley (1964, 1974), Piteau, Mylrea, and Blown (1978), and Bell (1976). All but Hadley documented the presence of weak, altered foliation surfaces which added to the loss in strength already present from foliation.

The less well-defined foliation in gneiss is, in turn, less susceptible to failure. Where involved in failure, the failures appear to have occurred in the schistose portions of gneiss and schist combinations, a common occurrence (Londe, 1973; Piteau and others, 1978). Gneiss and schist are also subject to shearing and crushing along foliation surfaces from tectonic stresses (Fowler, 1976). These crushed or mylonite zones constituted the discontinuities along which failure occurred at Malpasset (Londe, 1973).

The combination of complex foliated surfaces and earlier tectonic failures may make the identification of a failure surface in the foliated metamorphic rocks a difficult to impossible task. Slosson and Cilweck (1966) and Robinson and Lee (1972) have encountered this problem in exceptionally complex rock masses.

Faults

General

Faults are discontinuities along which there has been relative displacement of the bounding materials. The direction of relative movement defines the kind of fault, such as normal, reverse, thrust, and strike-slip, from the geological standpoint (Spencer, 1977). Geotechnically, recognition of faulting is of greater importance than classification. This is because faults create continuous discontinuities in rock masses that may extend for miles.

Recognition of faulting in the field is achieved most easily by observing displacement where marker beds are present. Where they occur in homogeneous crystalline rocks, their presence is marked by slickensided surfaces, continuity of a greater degree than is normal for joints, and often the presence of gouge or mylonite. Slickensides and gouge are also common to sedimentary and metamorphic rocks that contain marker units. The smoothed surfaces typical of slickensides cause a marked reduction in shear strength along the discontinuity.

Faults may be expected where there has been regional crustal deformation. Mountainous areas are the most obvious sites. However, deformation much less in magnitude may cause faults, as in the large basin and dome (and arch) structures of the upper Mississippi and Ohio Rivers and Great Lakes area of the United States. Faults may be numerous in an area, with patterns and associations related to the deformation of the area. Where closely spaced, they may join to form a zone of interlacing small faults or a crushed zone of gouge, breccia, or mylonite (Spencer, 1977). The zone of crushing is sometimes referred to as a shear zone. In either case, the mass of rock involved is sizable. The influence on rock-mass strength will be great unless the zone has been recemented or "healed." The deleterious influence will be even greater if alteration to clay has occurred and moisture content is high. Robinson and Lee (1972) have described the problem of identifying the slip surface of a landslide that occurred in a shear zone with its heterogeneous mixture of sheared bedrock.

Geotechnical aspects

In rock-slope stability analyses, faults are treated as boundaries, as are other discontinuities (Taylor, 1966). Large-scale slope failures have occurred in rock from such continuous features, especially where they dip steeply into an excavation (Deere and others, 1967).

Minor faulting may offset other discontinuities such as bedding surfaces, and actually may increase the rock-mass strength if potential block movement were to follow the bedding (Rawlings, 1968). The orientation of irregularities on a fault surface relative to the direction of potential movement also may have a keying effect on the mass strength (Taylor, 1966; Patton and Deere, 1971).

Even though the orientation of joint sets may be known for an area, faulting creates additional joints close to faults. They may be quite different in orientation, frequency, and continuity from those that are of regional origin (Piteau, 1970).

Failures of major structures or of associated slopes have occurred during and following construction, and have resulted from faults and shear zones, as at Malpasset Dam in France, Rapel Dam in Chile, and Libby Dam in the United States (Londe, 1973; Hamel, 1974). Cruden (1976) has described a major slide in the Canadian Rockies that followed a fault.

Although no usually classed as rocks geotechnically, the overconsolidated clay shales are geologic units. They are caught up in crustal deformation with the more typical rock units with which they are interbedded. The association of slope failures with faults in such materials has been described by Fookes and Wilson (1966) and Esu and Calabrisi (1969).

The influence of material, moisture content, slope, and vegetation on slope stability

General

Many factors contribute to the stability of natural and manmade slopes in addition to discontinuities. Because of inherent stregth of intact (discontinuity-free) rock, the stability of a rock mass has been shown to be influenced more by the presence of mass-strength reducing discontinuities than a soil mass (Terzaghi, 1962; John, 1970; Piteau, 1970, 1971; Cruden, 1975). Conversely, failures of soil masses usually are not dependent upon the presence of discontinuities because of the characteristically low shear strength of intact soil masses in comparison with rock (Patton and Deere, 1971; Piteau, 1971).

The stability of soil masses primarily is dependent on material type and moisture content as both determine or influence the shear strength of a given soil. To a lesser degree, material type and moisture also influence the stability of rock slopes.

The amount of steepness of slopes greatly influences the stability of both soil and rock masses. Slope height and length interact with slope steepness to introduce instability. In addition, slope aspect or the direction in which a slope faces, affects stability. Thus the shape and orientation of natural and manmade slopes must be considered as factors that contribute to the stability of soil and rock slopes.

Vegetation, though not as important a factor as material type, moisture content, and slope, may influence the stability of some slopes. The change in vegetative cover by natural causes as in fires or by manmade causes as in forest clearcutting affects moisture content of soils and may be a contributing factor in slope failure from increased moisture. The presence of vegetation may also contribute positively to soil strength as well as being a part of the load or surcharge on a slope. In addition to the preceding factors, such things as weathering of soils and rocks, sudden surcharge changes from rockfalls and debris flows, and previous slide history may influence slope stability (Fleming and others, 1977). Obviously, all or some of the many contributing factors may interact at a given time and place to upset slope equilibrium.

Material

General

In this section emphasis will be placed on those aspects of materials which are the products of composition, texture, mode of origin, and occurrence in nature. Such geologic factors may contribute much to slope design decisions (Palladino and Peck, 1972). Geotechnical parameters of soils such as shear strength, coefficient of internal friction, cohesion, and Atterberg limits will not be discussed. Averages and ranges of values for various types of soils may be obtained from texts in soil mechanics and engineering geology. These data also abound in the geotechnical literature for given cases.

In addition, discontinuities in rocks such as bedding surfaces, joints, and foliation which so greatly affect rock mass strength have been described in detail earlier. With the exception of the effects of water on the stability of rock slopes the strength-reducing influence of discontinuities must be superimposed on the other strength-related rock characteristics that are considered in this section for a complete evaluation of stability.

Composition and texture

Among soils, the presence of clays is the consistent common denominator among those slopes that are susceptible to slope failure. Watari (1967) reported that in 4 years of observation, fresh, unweathered shale had decomposed into clay and failed. Yatsu (1967, p. 396) noted that earthflows in Japan, Canada, and Scandinavia have a "remarkable relationship to swelling clay minerals." The clay shales, which often are classed and treated geotechnically as overconsolidated soils or clays, are the rocks most commonly associated with material-related failures. In North America the Upper Cretaceous Claggett Formation and the Bearpaw and Pierre Shales are classic examples of failure-prone clay shale (Banks, 1972). Of 37 landslides in the Edmonton, Alberta, area, studied by Thomson and Yacyshyn (1977), all of those that occurred in rock (18) were in Cretaceous clay shale. In their evaluation of landslide potential in the United States, Krohn and Slosson (1976) classed Mesozoic and Cenozoic sedimentary rocks which contain large amounts of clay as landslide-prone rocks.

The percentage of clay-sized material present in clay soils and clay shales has been reported by several investigators (Taylor, 1970; Early and Skempton, 1972; Palladino and Peck, 1972) with a lower limit of 24 percent and a maximum of 95 percent. As little as 10-15 percent clay may be a factor in slope failure if the clays are swelling clays (Quigley and others, 1971). Studies of long-term stability of overconsolidated clay shales of Carboniferous age by Skempton (1964) have shown a reduction in shear strength with increased clay content.

For failure-susceptible slopes in both soils and the clay-rich sedimentary rocks, the swelling clays montmorillonite, mixed-layer illite-montmorillonite, smectite, and swelling varieties of vermiculite are characteristically present in most cases (Leighton, 1966; Wahlstrom and Nichols, 1969; Anderson and Schuster, 1970; Taylor, 1970; Kerr and others, 1971; Quigley and others, 1971; Banks, 1972; Hamel and Flint, 1972; Prior and Ho, 1972; Erskine, 1973). The presence of swelling clay has been found to accelerate soil softening with subsequent failure (Quigley and others, 1971). In addition, the percentage of montmorillonite present in a soil or clay shale is inversely proportional to the residual shear strength (Attewell and Farmer, 1976).

The physical properties of montmorillonite, in turn, are greatly influenced by the kind and amount of exchangeable cations in the mineral lattice. Increased sodium increases montmorillonite plasticity and susceptibility to failure while increased calcium reduces plasticity and has been used to stabilize slopes in montmorillonite-rich soils (Krynine and Judd, 1957; Matsuo, 1957; Attewell and Farmer, 1976). Erskine (1973) reported that the montmorillonite in the Pierre Shale is responsible for its susceptibility to failure. Those members which contained less montmorillonite consistently had fewer failures and held steeper slopes. The claystone units of the failure-prone Orinda Formation in California are composed of from 24 to 95 percent montmorillonite (Taylor, 1970).

Montmorillonite has been associated with the development of rotational and complex slides in the Barbados by Prior and Ho (1972). The complex slide with the greatest distortion and breakdown of original soil structure was composed almost entirely of montmorillonite. Mixtures of the swelling and nonswelling clays resulted in rotational slides while exclusively kaolinitic (nonswelling) soils were associated with translational slides. This change to translational sliding is a function of the reduction in plasticity and water content with increased nonswelling clay content.

As noted above, slides occur in soils which may have clays which are exclusively nonswelling. Early and Skempton (1972) reported that the predominant clay mineral in failed colluvium derived from Carboniferous mudstones was kaolinite. Many investigators have reported slides in soils which contain a wide range of nonswelling and swelling clay combinations including illite, kaolinite, and chlorite in addition to the swelling clays noted earlier (Wahlstrom and Nichols, 1969; Paeth and others, 1971; Hamel and Flint, 1972; Palladino and Peck, 1972; Prior and Ho, 1972). Where mixtures occur, the more stable slopes are those that have proportionally higher amounts of nonswelling clays (Paeth and others). In their work in the western Cascade Mountains of Oregon, Paeth and others observed that the stability of soils did not appear to correlate with the amount of clay present but rather with the kinds of clays present.

Clay content in clay-rich soils and in clay shales also is a factor in the development of fissures. Fissures may develop from volume reduction accompanying desiccation and also from volume increase upon removal of overburden stress in the case of overconsolidated clays and clay shales (Attewell and Farmer, 1976). Examples of fissure development from overconsolidation appear commonly in the geotechnical literature.

Overconsolidated clay-rich soils typically are associated with overburden stresses exerted by ice with subsequent stress reduction upon melting of the ice. These range from water-laid clays (Wilson and Johnson, 1964; Palladino and Peck, 1972; Haug and others, 1977) to clay-rich glacial tills or boulder clays (Skempton, 1964; Hooper, 1970; McGown and others, 1974). While Skempton found only a few overconsolidated boulder clays with fissuring, McGown, Saldivar-Sali, and Radwan found fissures to be common in the tills they examined. A comparison of till fabric and fissure orientations indicated an ice stress origin for the fissures rather than from other causes such as slumping.

Overconsolidated clay shales ranging in age from Paleozoic to Cenozoic have been reported by Henkel and Skempton (1955), Hardy, Brooker, and Curtis (1962), Skempton (1964), and Fleming, Spencer, and Banks (1970). Both stratigraphic loads and ice loads have contributed to the overconsolidated state of the sediments. Anderson and Schuster (1970) reported the occurrence of fissuring in clay interbeds between basalt flows in the Pacific Northwest. The clays were formed by alteration of pyroclastic deposits and were overconsolidated by the overlying volcanics.

Failures of sensitive soils have been attributed in part to high pore pressures in sand partings within the sensitive mixtures of clay and silt (Pryer, 1957; Hutchinson, 1961; Conlon, 1966). Although highly sensitive (or quick) clays typically have a moisture content greater than their liquid limits, a combination of texture and bonding contributes greatly to their sensitivity. When deposited in a marine environment as was the case for many clays in eastern Canada and Norway, the individual clay particles will flocculate as their deficient ionic charges are satisfied by cations in the marine water. The resultant sediment has a very open, random texture and a high water content. Leaching of the marine-related cations by fresh water removes the bonding while still retaining interclay water in excess of the liquid limit. Collapse of the structure then can occur upon disturbance or remolding of the soil or by excessive pore pressures. The normal concentration of salt in the marine pore water is 35 parts per thousand. Increases in sensitivity occur in the 5-10 parts per thousand range and extremely quick clays may have the salinity reduced by leaching to less than 1 parts per thousand (Bjerrum and others, 1969). The classic paper by Bjerrum (1955) and a summary of sensitive clays by Attewell and Farmer (1976) should be consulted for more information about this unique combined compositional and textural influence on slope stability.

While clays contribute to the low shear strength of many failure-susceptible materials, noncohesive soils also have low shear strengths. They normally are not as widely reported as troublesome materials because their obvious lack of cohesiveness and low shear strength precludes the development of steep natural slopes and the construction of oversteep slopes. In addition, their good internal drainage reduces the influence of moisture on stability. Yee and Harr (1977) reported some unusual cohesionless soils from the Oregon Coast Ranges. They consist of soil grains composed of aggregations of smaller particles held together by clay, organics, and iron and aluminum oxides. Although cohesionless, the soils had unusually high internal friction angles which improved stability.

Regardless of their residual or transported origins, soils ultimately are products of the mechanical and chemical weathering of rocks. Soil parent materials therefore control soil composition and gradations with some modification by the transport and depositional environments for transported soils. Figure 25 shows the relation of soil and rock failures to parent material for a study in North Carolina by Leith (1965) of over 400 slopes. Predictive capability of soil physical properties is enhanced by field knowledge of soil origin and parent materials. Bishop and Stevens (1964) concluded that the solution of future landslide problems in southeastern Alaska will require knowledge of parent materials so that soil composition and properties can be predicted.

Few rocks are unstable for reasons other than the presence of discontinuities and clay. Some metamorphic rocks have low shear strengths from compositional differences rather than from just foliation. Bell (1976) found that failure may be localized by soft, weak rocks as talc schist. Dominance of mica in some schists and gneisses also may cause preferential failure zones. As these zones are so intimately related to foliations both in orientation and occurrence, separation of mineralogy from discontinuities would be academic.

Zones of different permeability within a rock mass for reasons other than discontinuities may control ground-water flow and pressures with resultant instability. Such variations may be present in the sand- and coarser-sized detrital sedimentary rocks because of differing degrees of cementation or from gradation differences caused by fluctuations in the depositional environment. Eisbacher (1971) noted such stability controls in graywackes and conglomerates in British Columbia.

Interbedded materials

In their comprehensive paper on the causes and effects of landslides, Radbruch-Hall and Varnes (1976) illustrated very well the occurrence of landslides in layered earth materials of differing composition. To be sure, a number of the cases have involved failures along discontinuities within or between the rock types. However, there is sufficient documentation of the failure of a weak unit which inherently has low shear strength to cause one to be suspect of such layered or interbedded situations. Radbruch-Hall and Varnes stated that such weak materials are especially prone to sliding when interbedded or overlain by more resistant rock. In other instances, increased pore pressures in a unit may influence failure more than the presence of weak material such as clay.

The most common reference to failure of interbedded rocks involves clay-rich rocks such as shale in combination with the other sedimentary rocks such as siltstone, sandstone, conglomerate, limestone, and dolomite. Failures typically occur in the clay-rich units and have been reported by many investigators in North America, Europe, and Asia (Nonveiller and Suklje, 1955; Henkel, 1961; Monroe, 1964; Henkel and Yudhbir, 1966; Broili, 1967; Wahlstrom and Nichols, 1969; Taylor, 1970; Eisbacher, 1971; Stout, 1971; Hamel, 1972; Voight, 1974).

Landslides in layered igneous rocks from causes other than discontinuities characteristically occur in extrusive materials. More specifically, failure takes place in clays formed by alteration of pyroclastics such as tuff. The clays are found interbedded with lavas such as andesite and basalt. The Tertiary volcanics of the Pacific Northwest are the most common examples of such failures in North America (Staples, 1957; Anderson and Schuster, 1970).

Slope failures in alternations of metamorphic rock types seldom occur for reasons other than discontinuities and related mineral orientation. However, in some metasediments argillites may be present in a rock sequence and may be subject to failure. Rawlings (1968) reported failures in argillite in interbedded massive quartzite and thin argillite.

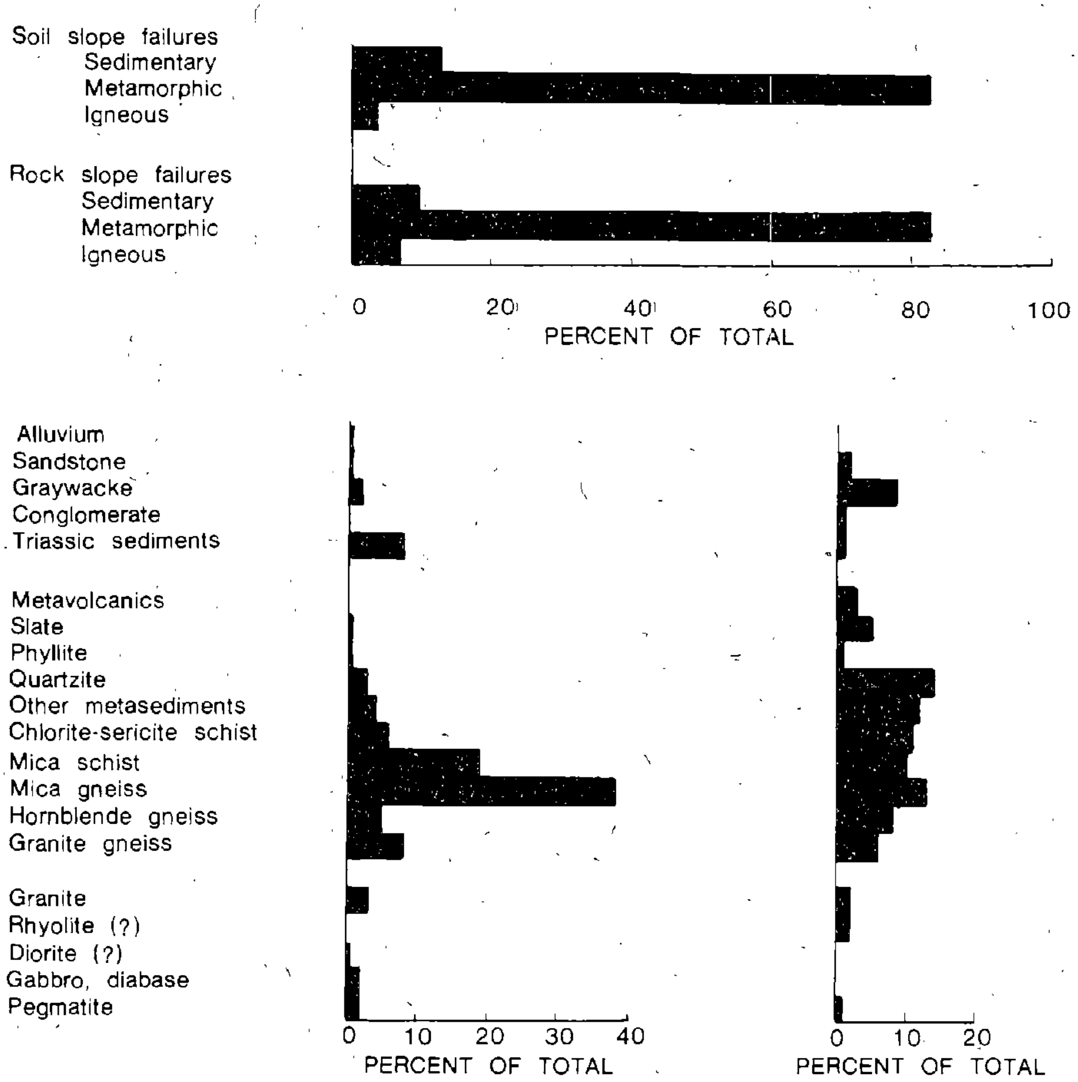


Figure 25. Slope-failure frequency for material types. A, Soil and rock slope failures by rock class; B, Soil slope failures by parent rock material; C, Rock slope failures. From C. J. Leith, The influence of geological factors on the stability of highway slopes, Trans. SME/AIME, Vol. 232, 1965, p. 151.

Residual soils

Soils typically are more susceptible to failure than rocks with the exception of closely jointed rock masses or those with discontinuity orientations and surfaces that are conducive to failure. Of the soils, residual soils developed on parent rock often have unique characteristics that set them apart from transported soils even though compositions may be similar. These characteristics include nonuniform decomposition into soil, topographic control of thickness of soil, and oriented zones or surfaces of differing strength (Deere and Patton, 1971). Preweathering discontinuities are controlling factors in both the nonuniform decomposition of some residual soils and the presence of relict structures or zones. Intensity of weathering is affected by climatic factors such as temperature, amount and intensity of rainfall, frost penetration and freeze-thaw action, chemical composition of ground water, slope, and presence of discontinuities which permit entry of water (Fleming and others, 1970; Scully, 1973).

Landslides commonly occur in weathered claystones and shales (Chandler, 1969; James, 1971; Scully, 1973; Prior and Graham, 1974; Briggs and others, 1975). Chandler reported several physical changes that occur in such rocks when weathered. The ratio of plasticity index (PI) to liquid limit (LL) changes from 10/35 for unweathered material to 35/65 for weathered material. In addition, bulk density decreases with weathering, as does shear strength. Failure of slopes has been observed to occur at the contact of weathered clay and unweathered shale by James and Prior and Graham.

De Fries (1974) listed the controls on the shear strength of residual soils developed on schists and phyllites. They are (1) degree of decomposition, (2) mineral composition of parent rock, (3) orientation of relict weakness planes, (4) magnitude of parent material deformation, and (5) others, such as degree of saturation and presence of cementing agents. Other than that of magnitude of deformation, they also are the controlling factors for all residual soils. Hundreds of slides in residual soils on phyllites have been reported in northern Italy by Engelen (1967).

The nonuniform degree and depth of weathering is characteristically related to the presence and orientation of previous discontinuities (relict features). This influence of discontinuities on weathering is especially well shown in weathered intrusive igneous rocks such as granite which have relatively widely spaced joints that control weathering. Weathering inward from the joints results in great irregularity and the development of unweathered cores of the blocks (Deere and Patton, 1971). The more closely spaced the discontinuities, the more thorough the weathering, as is the case in the foliated metamorphic rocks (Leith, 1965; Patton and Deere, 1971). Complex structure within the metamorphic rocks may result in highly irregular depths to fresh rock (Deere and Patton). Figure 26 illustrates weathering in igneous and metamorphic rocks.

Relict structures are present even in completely decomposed rock material and exert a weakening influence on the soil mass (Leith, 1965; St. John and others, 1969; Deere and Patton, 1971). Leith reported that such structures influence both the geometry and mechanism of slides. Deere and Patton identified relict joints and faults in partially weathered rock as potential sliding surfaces especially where high pore pressures are present.

St. John, Sowers, and Weaver (1969) described the presence of seams up to 2.5 cm thick of weak clay and complex organic compounds in residual soils developed on igneous and metamorphic rocks. These seams have been interpreted as fillings of old joints and have been responsible for numerous landslides in residual soils worldwide. Similar seams reportedly have been found in soils developed from sandstones and tuffs, rarely in limestone residual soils, and never in soils from shales.

Colluvial soils

Colluvial soils are those soils that form at the foot of slopes chiefly by gravity movement downslope of rock weathering products (Gary and others, 1972). The slide-prone colluvial soils reported in the geotechnical literature characteristically have been derived from fine-grained, detrital sedimentary rocks such as shale and siltstone (D'Appolonia and others, 1967; Early and Skempton, 1972; Hamel and Flint, 1972; Gray and others, 1974; Miller and Wiethe, 1975; Royster, 1975). Gradations typically range from clay to boulder size where shales, claystones, and siltstones are interbedded (Miller and Wiethe; Royster). The various expansive and nonexpansive clay minerals are present and have reportedly constituted as high as 70 percent of the colluvium where the parent material was a mudstone (Early and Skempton).

Failures of colluvial soil slopes occur in several ways. Slides may occur along buried soil profiles within the colluvium that developed as the colluvial slope formed (Deere and Patton, 1971). Creep and/or sliding during slope development will have reduced the shear strength to residual or near-residual strength along slip surfaces. Disruption of equilibrium by natural or manmade causes can trigger failure along these surfaces (Gray and others, 1974).

The most commonly reported cause of failure of colluvial slopes is the contact between the colluvium and the underlying material. Failure may occur where colluvium overlies impermeable residual soils (Royster, 1973; Miller and Wiethe, 1975) and shale (Royster, 1975). Jones and Larsen (1970) described failure at the contact of colluvium and underlying low-strength residual sand. With the development of colluvium on a slope, Royster (1975) pointed out the obvious influence

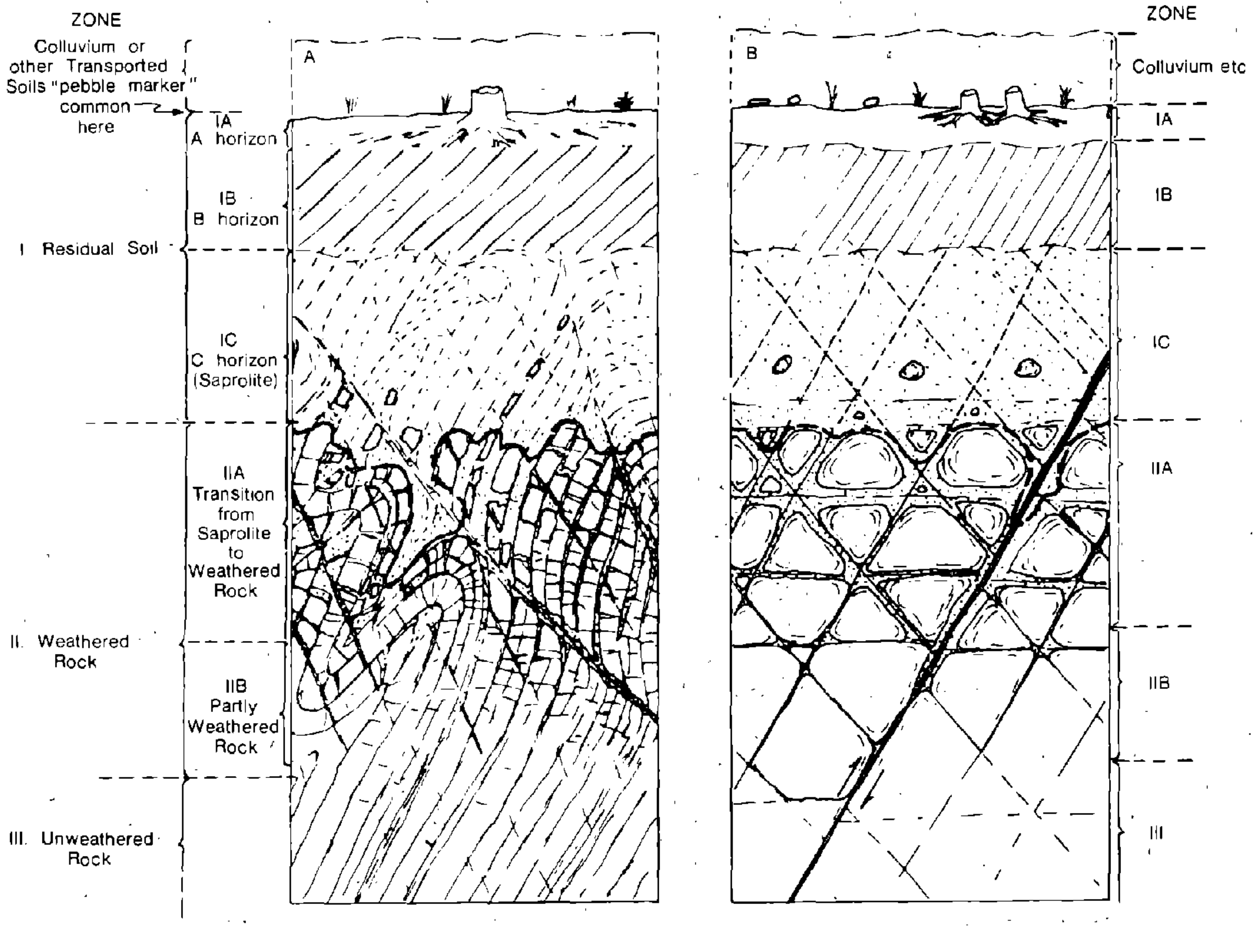


Figure 26. Typical weathering profile for A, metamorphic and B, intrusive igneous rocks. From Slope stability in residual soils, D. U. Deere and F. D. Patton, p. 90, Proc. Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, San Juan, Puerto Rico, 1971.

of the slope angle on stability. He has stated that in rugged topography the steepness of the contact between colluvium and underlying material, shale in this case, is the controlling factor in slope stability. As will be noted in a later section, moisture combines with differences in permeability, as at the base of a colluvial soil, to decrease stability.

Glacial soils

Of the many kinds of soils that may be the result of glaciation, the ones most often referred to in terms of slope stability are the glacial lake (lacustrine) clays, as noted in an earlier section. The cases described have all been in areas of continental or piedmont glaciation (Wilson and Johnson, 1964; Quigley and others, 1971; Palladino and Peck, 1972; McGuffey, 1973; Tubbs, 1974; Haug and others, 1977; Thomson and Yacyshyn, 1977). Failures in other stratified soils of glacial origin have also been reported, such as in interbedded clays, silts, and sands in glacial outwash (Shepard and others, 1962), preglacial lake deltas (McGuffey, 1973), and in deposits of loess (Lutton, 1971). In the cases described by Tubbs in the Seattle, Wash., area, 28 of 40 slides studied occurred in permeable sands overlying impermeable glacial clays. Of 14 failures in glacial deposits studied by Thomson and Yacyshyn, 9 occurred in glacial lake clays.

Failures also occur in glacial till, which is an unstratified glacial soil that is characteristically well graded. In New York State, McGuffey (1973) noted that failures are common in cut slopes in all kinds of glacial till deposits but that they potentially present the greatest problems in moraines and colluvium developed on moraine slopes. Slides have been reported in both clay-rich, impermeable tills (Skempton, 1964; McGown and others, 1974) and in permeable tills (Beaty, 1972a). Of 14 slides examined in Alberta which were in glacial deposits, three were in till of probable impermeable texture (Thomson and Yacyshyn, 1977). The remainder were all in glaciolacustrine and glaciofluvial deposits.

Altered rocks

Alteration or chemical decomposition of rocks by hydrothermal or heated, chemically active solutions results in radical reductions in strength, often the result of formation of weak clays. Areas of volcanic activity are especially conducive to such alteration regardless of the rock types present. Of the 140 slides that have occurred in the H. J. Andrews Experimental Forest in Oregon since 1950, 138 have been in altered volcanic materials (Swanson and Dyrness, 1975). Watari (1967) and Radbruch-Hall and Varnes (1976) reported on problems involving hydrothermally altered volcanic rocks in Japan. The large Downie slide in British Columbia is in altered mica schist and granitic gneiss (Piteau and others, 1978).

Material contact influence on failure

Contacts between materials other than typical discontinuities often form slip surfaces at time of slope failure. In addition, thin discrete zones of weak material also may be the sites for failure.

Shallow slides have occurred within the soil profile coincident with soil changes in the A and B horizons (Deere and Patton, 1971; Prior and Ho, 1972; Blong, 1974). Development of soil horizons must be pronounced for failure to occur. The residual soils that develop in the tropics are favorable sites for such development.

Contacts between different soils also can coincide with failure surfaces. Glacial clays and compact, clay-rich tills both provide failure surfaces for overlying soils (Swanston, 1969; Tubbs, 1974). Jones and Larsen (1970) have described a failure at the contact of a silty clay colluvium with underlying residual micaceous quartz sand.

Surficial materials in contact with a bedrock surface provide the necessary material contrast for many slope failures. This is especially true where the contact between the overlying material and the underlying bedrock is sharp. The literature would indicate that slides along a soil-shale contact are most common (Benson, 1946; Christensen and Lohnes, 1973; Royster, 1973; Murphy and Rubin, 1974; Miller and Wiethe, 1975; Royster, 1975). Jimenez (1972) reported slides at the interface between talus and underlying undifferentiated sedimentary rocks.

While not a soil-bedrock contact, the presence of thin seams of clay within other rock types with greater shear strength cannot be ignored as a source of many slope failures. There is a close relationship between failures along these seams and the orientation and continuity aspects of discontinuity-controlled failures. Clay seam-controlled slides within a sedimentary rock sequence have been reported by Staples (1957), Hutchinson (1961), Kerr, Stroud, and Drew (1971), Eigenbrod and Morgenstern (1972), Hamel (1972), M. Barton (1973), Pasek (1974), and Wu, Thayer, and Lin (1975). The unique clay seams formed by decomposition of ash layers between basalts within the Columbia River Basalt Group have been described by Anderson and Schuster (1970).

Water

General

Given near-equilibrium conditions in soil and rock slopes, the amount of water in soil pore spaces or in rock discontinuities is an almost universal factor in whether a slope remains stable or fails. The disturbance in equilibrium may result from loss in shear strength from increased pore pressures in permeable soils, the softening or lubricating of impermeable clays, or development of uplift pressures along joints (Terzaghi, 1936, 1950; Leighton, 1966; Bruce, 1968; Patton and Deere, 1971; Krohn and Slosson, 1976). The increased bulk density or surcharge of a soil mass from an increase in moisture content in turn increases the driving force in a potential slide mass, disturbing equilibrium (Leighton; Easton, 1973; Krohn and Slosson). Seepage forces acting in the direction of potential failure may also upset the equilibrium of both soil and rock slopes (Leighton; Deere and others, 1967; Swanston, 1974; Attewell and Farmer, 1976).

Bishop and Stevens (1964) have written that given susceptibility to sliding, moisture content is the universal and most important single causative factor in slope failure. Beaty (1956) concluded that although there are multiple causes for sliding in the Coast Ranges of California, excess water is the primary immediate cause. Similarly, Scully (1973) observed that water controls most of the slides that occur in the Pierre Shale. Inadequate control of surface and subsurface water has been given by Brawner (1959) as the major cause of slope instability in highway construction in British Columbia.

Rainfall and snowmelt

Rainfall is the most commonly cited source of water which in turn influences slope stability in soil and rock. Natural slopes tend to equilibrate with respect to the average rainfall in an area. As a result, above average to high-intensity rainfall has been identified by many investigators as the primary causative factor in all types of instability, ranging from rockslides to debris flows in all kinds of earth materials worldwide (Beaty, 1956; Staples, 1957; Bishop and Stevens, 1964; Dyrness, 1967; Crozier, 1969a; Lambe and Whitman, 1969; Swanston, 1969; Eden and Mitchell, 1970; Erskine, 1973; Blong, 1974; Gray and others, 1974; Tubbs, 1974; Voight, 1974; Briggs and others, 1975; Nilsen and Turner, 1975; Vandre, 1975; Fowler, 1976). Peck (1967) noted the significance of accumulated rainfall over a period of time.

Extended periods of rainfall, as is the case in areas subject to seasonal rainfall, may trigger slope failures. Most commonly, however, the initiating factor in sliding in such areas will be the occurrence of a high-intensity storm following a period of rainfall. Such relationships have been documented by Leighton (1966), Onodera and others (1974), Miller and Wiethe (1975), Swanson and Dyrness (1975), Nilsen, Taylor, and Dean (1976), and Radbruch-Hall and Varnes (1976).

Critical levels of rainfall needed for slope failure have been observed by some investigators. Radbruch-Hall and Varnes (1976) reported that slope failures in Japan increase abruptly when seasonal rainfall exceeds 150-200 mm and intensity exceeds 20-30 mm/h. This relationship was recognized earlier by Onodera and others (1974), who noted that rainfall intensities of 100 mm/h are not rare in Japan. Radbruch-Hall and Varnes also reported that failures in the residual soils in Hong Kong exceeded 50 per day when daily seasonal rainfall exceeds 100 mm after a previous 15-day cumulative rainfall of greater than 350 mm. They also have determined that in California the critical rainfall cumulative level is 250 mm. If periods of drying occur during the rainy season, greater than 250 mm will be needed to obtain failure. Nilsen, Taylor, and Dean (1976) have reported seasonal and high-intensity storm combinations that have caused large increases in slide activity in the San Francisco Bay area of California. Representative values are a 229-mm storm after a cumulative 787-mm seasonal amount and a 178-mm storm after a cumulative 330-mm seasonal rainfall.

The incidence of landslides with rainfall duration and intensity has been investigated by Nilsen and Turner (1975) in the same area. An incidence of seven slides was noted after rainfall of 105 mm over an 8-day period was followed by a 196-mm storm with an intensity of 24.4 mm/d. By comparison in the next month, 43 slides occurred after rainfall of 301 mm over a 17-day period was followed by a 259-mm storm with an intensity of 15 mm/d. Nilsen, Taylor, and Dean (1976) have shown the relationship between accumulated rainfall and landslide activity (fig. 27).

With reference to landslide activity or frequency during extended periods of rainfall, Nilsen and Turner (1975) stated that, for those landslides they studied in California, most have occurred where abundant sliding has been present in the past. In a later report Nilsen, Taylor, and Dean (1976) observed that a substantial number of the slides not associated with ancient slides may have resulted from man's disruption of previously stable conditions. In general, however, they felt that the key to predicting future landslide activity is knowing the distribution of older slides.

In addition to initiation of slides, rainfall amounts also are positively correlated with movements in previously defined slides. Peck (1967) stated that while daily rainfall is too erratic to be correlated with slide movement, accumulated rainfall over several days usually is significant in affecting movement rates. This has been shown to be true by many investigators (Benson, 1946; Nonveiller and Suklje, 1955; Brawner, 1959; Merriam, 1960; Henkel and Yudhbir, 1966; Peck, 1967; Easton, 1973; Vandre, 1975). Easton noted movement response in the Portuguese Bend landslide in California within a few hours after as little as 5-10 mm of rainfall. Also, movement rates as high as 13.5 mm/d during periods of rainfall declined to 8 mm/d within a month after cessation of the rainy season as drainage occurred within the slide. Peck has shown the relation between movement rate and accumulated rainfall (fig. 28).

Increase in moisture content in slide-susceptible materials also may result from melting snow. Eden and Mitchell (1970) and Eden, Fletcher, and Mitchell (1971) reported failures in sensitive soils in Canada following melting of snow. Combinations of rain and snowmelt contributed to failures of all types described by Dyrness (1967),

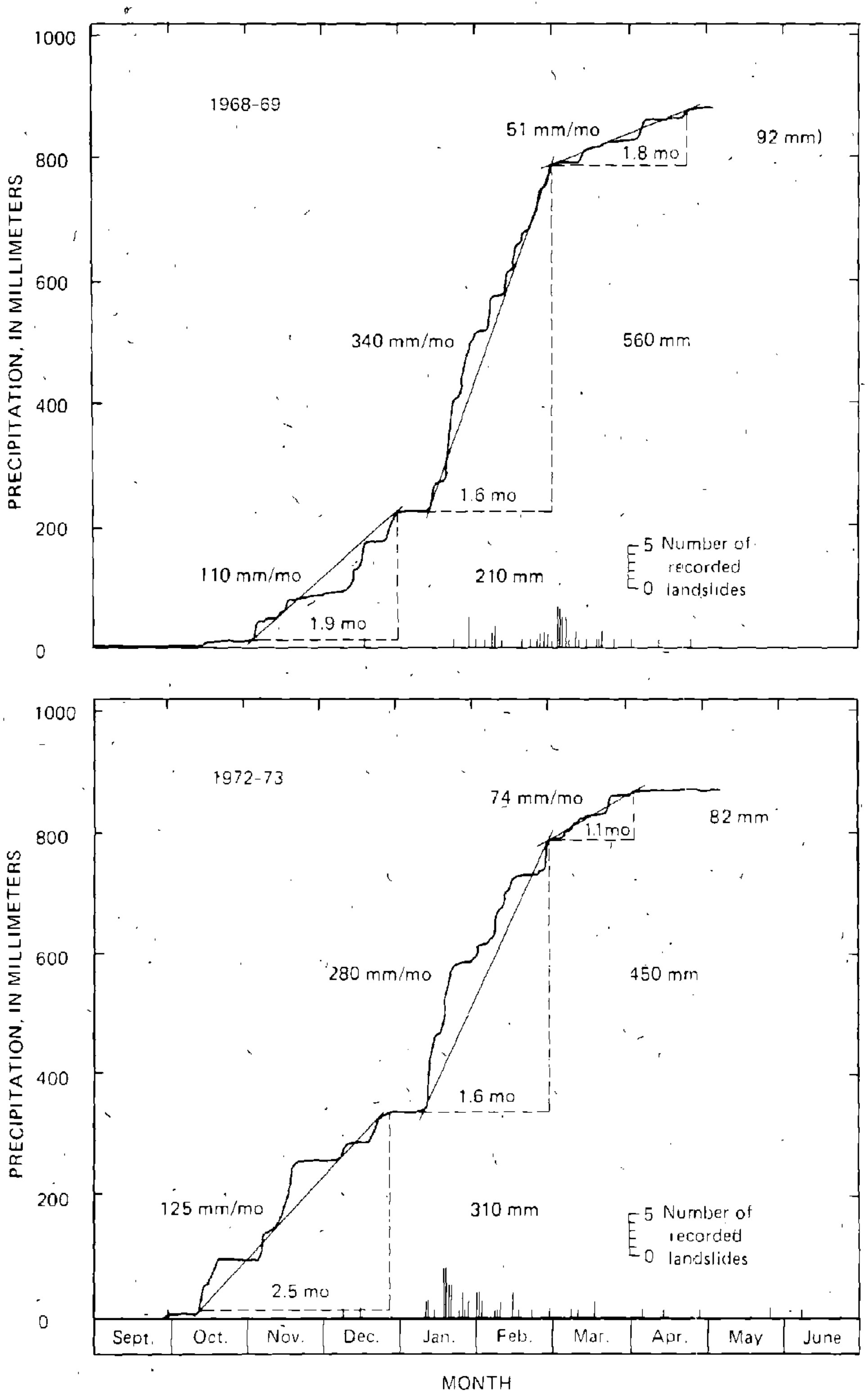


Figure 27. Comparison of landslide activity and rainfall accumulation. Modified from Nilsen, Taylor, and Dean, 1976.

Christensen and Lohnes (1973), Gray, Ferguson, and Hamel (1974), Swanson and Dyrness (1975), and Day and Megahan (1976). Old slides may be reactivated by moisture increases from rapid melting of snowpack (Fleming and others, 1977).

Moisture content

Moisture contents in amounts less than saturation have an influence on the shear strengths of cohesive soils (Attewell and Farmer, 1976). The Atterberg limits (plastic limit and liquid limit) serve to illustrate this. In both limits moisture content is the measured parameter at the boundaries between the semi-solid and plastic and plastic and liquid physical states. Reduction of shear strength in cohesive soil with increased moisture content has been documented in the field by many investigators (Meyerhoff, 1957; Pryer, 1957; Nasmith, 1964; Swanston, 1969; Eden and Mitchell, 1970; Beaty, 1972b; Skopek and others, 1972; De Fries, 1974; Day and Megahan, 1976; Haug and others, 1977). Swanston estimated as much as a 60-percent loss in shear strength with above-normal moisture content in glacial soils in southern Alaska. Nasmith reported a greater than 50-percent reduction in the unconfined compressive strength of a clay till with an 8-percent increase in moisture content. De Fries noted as much as a 50-percent reduction in friction between sheetlike minerals in soils developed on schist and phyllite as saturation is reached.

Meyerhoff (1957) reported moisture contents from 6 to 27 percent greater than the liquid limit for sensitive soils at slide sites in Norway, Canada, and Sweden. The soils were predominantly clays and silty clays with some silts.

Moisture content is also higher in cohesive soils at a slide surface. Skempton (1964) observed a 5-percent greater moisture content in softened clay along a slip surface in the London Clay than in the adjoining clay. Banks (1972) found twice the normal moisture content along slip surfaces in the Cucaracha Shale in the Canal Zone. Similarly, Skopek, Rybar, and Dobr (1972) reported moisture contents as much as 27 percent higher along the remolded clay slide surface than in the claystones above and below the surface.

Although a noncohesive soil, loess may retain moisture more readily than coarser-grained soils. In his investigations, Lutton (1971) found that the shear strength of loess is influenced by moisture content. There is a characteristic deterioration of strength as loess approaches saturation. This was judged to be the primary cause of slumps in near-vertical cuts in loess.

Pore pressure

Saturated soils may be subjected to positive pore pressures of amounts dependent in part on the height of the water table or piezometric surface above the level in question. Compaction of soils which cannot drain by either natural or manmade causes also results in increased pore pressure (Duncan and Seed, 1966). The result of increased pore pressure is a reduction in effective normal stress equal to the increased pressure. It results in a reduction in the effective normal stress on the soil mass which, contributes to failures at lower stresses.

The reduction in strength at failure with increase in pore pressure has been observed in low-permeability materials such as clay, silt, and clay shales by Kjellman (1955), Terzaghi (1955), Erskine (1973), and Easton (1973). Reductions were noted in sufficient magnitude to cause Terzaghi to refer to a "near fatal" rise in pore pressure which resulted in failure of a slope. Others who have related slope failures in soils and clay shales specifically to pore pressure are Brawner (1959), Conlon (1966), Beene (1967), Dyrness (1967), Bruce (1968), Wu, Thayer, and Lin (1975), and La Rochelle, Lefebvre, and Bilodeau (1977). Brawner reported that of 18 slides studied, 16 were primarily caused by excessive pore pressure. Vandre (1975) calculated that keeping the piezometric surface below a potential failure surface raised the factor of safety from 1.05 to 1.50.

As would be expected, rainfall and snowmelt increase pore pressures in soils. Eden, Fletcher, and Mitchell (1971) observed increases in both piezometric surface and river levels with melting of record snow accumulation in Ontario. Following this the river level had dropped, but the piezometric surface remained high, with resultant high pore pressures and seepage forces in the clay, silt, and sand units. This indicates slow drainage, which also implies a delay in the buildup of pore pressures at the time of heavy rainfall or rapid snowmelt. Such a delay has been determined by Skopek, Rybar, and Dobr (1972). They found that an increase in pore pressure after excessive rainfall or thawing was invariably delayed from 6 to 8 days. Harper (1976) reported response intervals ranging from 1 to 22 days for different rock types. Similar drainage delays may result in reduced shear strength and failure in low-permeability soils when excavation is more rapid than drainage. Christensen and Lohnes (1973) have reported such a failure in glacial till.

Where sand and clay layers are interbedded, the high pore pressures that may be present in the sand layers are of great importance in maintaining the stability of the soil mass. Kjellman (1955) stated that the stability of the mass is least when the pore pressure in the sand is the greatest. In addition, the shear strength of the adjacent clay is lowest near the sand-clay interface. In such cases the failure surface will form either in the sand layer or in the clay close to the contact. Conlon (1966) also concluded that high pore pressures in interbedded sands were a contributing factor to a failure in sensitive clays and interbedded sands in Quebec.

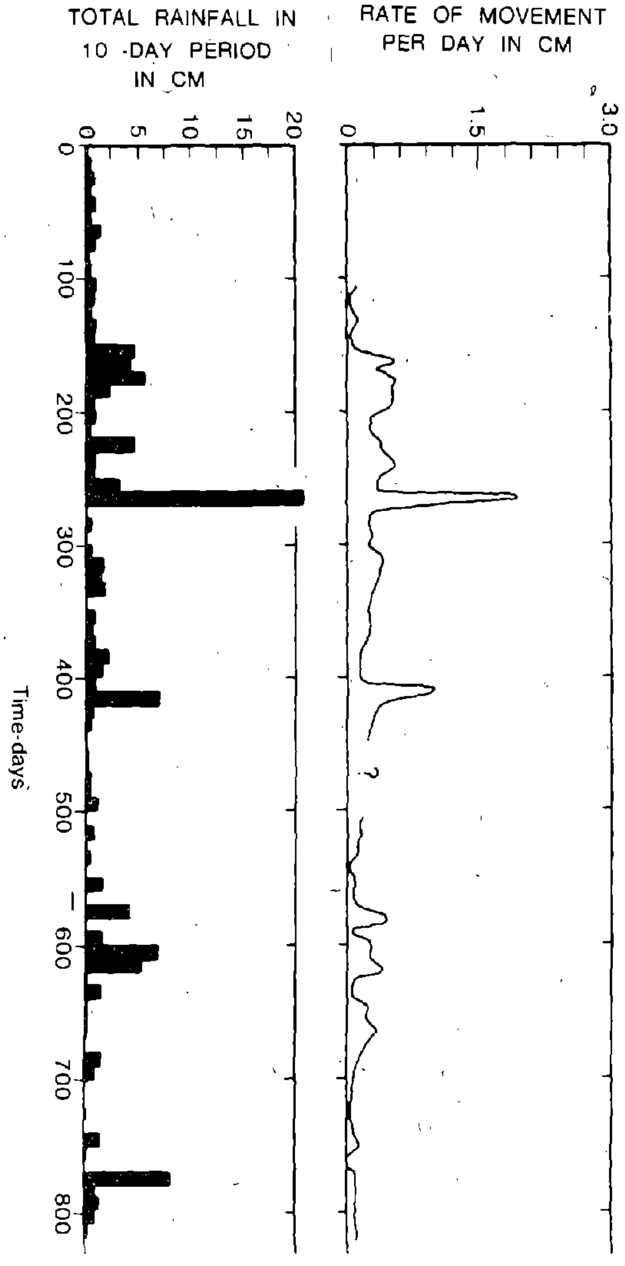


Figure 28. Comparison of rate of horizontal movement and accumulated rainfall, Waiomao slide, Honolulu. Queried where data missing. Modified from Stability of natural slopes, R. B. Peck, p. 414, Jour. Soil Mechanics and Foundation Engineers Div., ASCE, 1967.

Failure of a low-permeability soil with high pore-pressure has been found to result in a decrease in pore pressure (Skopek and others, 1972; Vandre, 1975). This drop in pressure accompanies the reduction in cohesion to zero along the failure surface (Vandre). Vandre stated that the reduction in pore pressure (or increase in effective stress) is expected to be permanent as a result of sliding. However, any stability benefits derived from this are offset by the reduction in shear strength to residual strength along the slip surface.

Influence of water on rock stability

Water pressure in rock joints plays a part similar to pore pressure in soils. Pressure is exerted on the surrounding rock when the joints are saturated and under a head. The result is a reduction in normal stress on the surface and a reduction, therefore, in the strength at failure along the surface (Hoek and Bray, 1974). Hoek and Bray stated that even though the unit water pressure may be small, the total force on the surface can be very large because of the surface area. As a consequence, control of water pressure regardless of unit amount can be critical in maintenance of slope stability. The reduction in strength at failure from water or uplift pressure along discontinuities has been recognized by many investigators (Wittke and Louis, 1966; Beene, 1967; Deere and others, 1967; Lane, 1967; 1967; Brawner and Gilchrist, 1970; Patton and Deere, 1970; Piteau, 1970; Hamel, 1972; Sharp and others, 1972; Hoek and Bray, 1974; Briggs and others 1975; Cruden, 1976).

In addition to the buoyant force exerted by water in discontinuities, instability is accentuated when the hydraulic gradient is in the same direction as the discontinuities. This is the result of driving forces in the direction of waterflow similar to seepage forces in soils (Wittke and Louis, 1966; Deere and others, 1967; Brawner and Gilchrist, 1970). The driving forces will not be uniform because of variations in permeability (tightness and openness) along the discontinuity. In any event, there will be a reduction in the factor of safety.

Water pressures in joints are influenced by the geometry of the joint sets and differences in joint permeability (Wittke and Louis, 1966; Deere and others, 1967; Patton and Deere, 1970; Scully, 1973; Voight, 1974). Deere and others noted that where a rock is uniformly and heavily jointed, steady-state seepage is developed and driving forces increase in the direction of seepage, as in soils. Local changes in driving force will occur where jointing is irregular and more widely spaced (Deere and others; Patton and Deere). The various joint patterns and nature of void spaces permit large fluctuations in ground-water levels. Patton and Deere have commented on the increase in rock permeability from blasting and stress relief that accompany excavation of a rock slope. This serves to retard the development of high water pressures near the new slope surface.

The presence of water in joints usually reduces the frictional resistance along the joints for reasons other than uplift pressures. Coulson (1972) and N. Barton (1973, 1976) reported that water reduces the surface energy and strength of mineral crystals exposed on the joint surface. For most smooth surfaces, there is little influence to a slight increase in shear strength when wet (N. Barton, 1973, 1976). Coulson found that massively structured minerals may act as an antilubricant. Quartz-rich rocks appear to be neutral while layer-lattice structure minerals such as chlorite may result in a significant reduction in shear strength when wet (Coulson; N. Barton, 1973, 1976).

Available data indicate that the shear strength of rough joint surfaces is reduced when the surfaces are wet. This is caused in part by the asperities failing more readily when wet (Barton, 1976). The peak strength of rough joints will be influenced more by water than would smooth joints. Also, water-filled depressions along a discontinuity, if trapped, can exert an uplift pressure equal to the normal force around the depression.

When a rough surface is sheared, water plays an additional role. The powdered shear debris will develop slickensides more readily when saturated, with resulting reduction in shear strength (Barton, 1976). Barton stated that when a gouge zone has developed the presence of water will further reduce the strength by softening the gouge. The same is true when joint-filling material is saturated, with accompanying reduction in cohesion and friction (Piteau, 1970)

Scully (1973) observed that joints in the Pierre Shale may or may not contribute to the problems generated by the presence of water. Where the shale is high in montmorillonite, the least amount of water enters the system as the joints seal shut. The largest slides occur in areas where the shale has low-swelling clays. Scully also found that water in the joints caused softening of the shale, a factor independent of either water pressure or roughness.

Interface water

As noted earlier under the section on pore pressure, confined water pressures exert uplift pressures at the confining boundary (Vandre, 1975). When impermeable slip surfaces have developed, uplift pressures may be observed. Sharp, Maini, and Harper (1972) noted that adverse distribution of pore pressure along a failure surface in rock may result from the presence of an impermeable fault.

It would appear from the geotechnical literature that in the case of an impermeable barrier the most common problems with slope stability occur when downward movement of water is impeded. Prolonged seepage of water along a slide-susceptible boundary, whether it be glacial till, shale bedrock, or a slip surface, will result in strength reduction and ultimate failure (Crozier, 1969a; Denness and Cratchley, 1972; Easton, 1973; Royster, 1973, 1975; Scully, 1973; Tubbs, 1974; Miller and Wiethe, 1975; Denness and Riddolls, 1976). The reservoir principle proposed by Denness (1972) is based on the accumulation of water in permeable materials above an impermeable unit such as a clay shale. The overlying permeable bed acts as a reservoir which continually causes ground water to flow along the contact to exposures where drainage occurs. Where the contact dips toward a slope, instability is to be expected because of the loss in strength along the softened clay.

Water entry

Aside from infiltration through permeable surface soils, the entry of water to the subsurface is normally by naturally occurring discontinuities. Joints, bedding surfaces, faults, and slide surfaces which intersect the surface are common points for water entry (Bruce, 1968; Hamel, 1972; Scully, 1973; Fowler, 1976). In overconsolidated clays and clay shales, fissures provide an additional entry (Terzaghi, 1936, 1950; Bruce, 1968; Duncan and Dunlop, 1969). Tension cracks in landslides permit entry of water to the slip surface during periods of rainfall and snowmelt (Henkel and Yudhbir, 1966; Easton, 1973; Fleming and others, 1977).

Man's activities also may facilitate movement of surface water into the underlying materials. Beaty (1972b) described sliding in Alberta that resulted from water which infiltrated permeable glacial soils from an unlined irrigation ditch. Irrigation also has been influential in the failure and continued movement of a large failure in Mancos Shale in southwestern Colorado (Varnes, 1949). La Rochelle, Lefebvre, and Bilodeau (1977) noted that infiltration of snowmelt into sensitive soils in Quebec was localized and accentuated by clearing of a parking area prior to failure. Disruption of surface and subsurface water flow by forest road construction has increased the potential for instability in the western Cascade Mountains of Oregon (Swanson and Dyrness, 1975).

While movement of the large Portuguese Bend landslide in California can be related to seasonal rainfall, Merriam (1960) wrote that abnormal precipitation probably was not a major factor in movement. He concluded that an estimated 121 000 liters per day water contributed by septic tanks and cesspools in the slide area doubled annual precipitation when added to moisture from lawn watering. Reactivation of the old slide began 5-6 years after home building in the area, which would indicate a contributive factor other than just seasonal rainfall. Leighton (1966) stated that in southern California water entering the earth from irrigation, swimming pools, and cesspools can be the equivalent of as much as 38 cm of rainfall, equal to the total annual rainfall.

Drainage

Drainage of slope materials is the time-honored procedure for improving the stability of soil and rock slopes. The intent is to reduce the pore pressures exerted at critical slide-susceptible zones in the slope. When natural drainage is inadequate, construction of interceptor drains upslope, toe drains at the bottom of a slope, and drilling of horizontal drain holes into the slope all have proved successful in preventing as well as stabilizing slope failures (Pryer, 1957; Jones and Larsen, 1970; Patton and Deere, 1971; Sharp and others, 1972; Vandre, 1975; La Rochelle and others, 1977).

Although natural drainage may normally keep pore pressures at a minimum, seasonal changes may affect drainage efficiency. Piteau (1970), Sharp, Maini, and Harper (1972), Hamel (1972), Christensen and Lohnes (1973), and Gray, Ferguson, and Hamel (1974) all reported retarded to blocked drainage by freezing of drain water as contributing to excess pore pressures and slope failures.

Slope

General

The steepness (degree or percent) of slope and the direction (aspect) in which a slope faces are important factors in the stability of soil and rock slopes. Krohn and Slosson (1976) stated that, for a given material, a steeper slope is more likely to be unstable than a less steep slope. The geotechnical literature confirms this conclusion. Beaty (1956), for example, observed that landslide susceptibility was directly proportional to steepness.

In a statistical study of 78 slopes in gneiss in the Colorado Front Range, McMahon (1968b) found a relationship between slope steepness and the steepness of the dip of joints. Slopes with steeply dipping joints were steeper than those with less steeply dipping joints. In addition, high slopes were statistically less steep than lower slopes in the same rock type. The values of the correlation coefficients, however, were only 0.349 and -0.398, respectively.

Moisture content, whether from rainfall or poor drainage, cannot be separated from slope steepness as the two interact inversely (Beaty, 1956, 1972a; Blong, 1974). There are also morphological factors that enter into stability such as the greater susceptibility of steeper slopes to oversteepening by stream undercutting or excavation by man (Krohn and Slosson, 1976; Nilsen, Taylor, and Brabb, 1976; Nilsen, Taylor, and Dean, 1976). The interaction of vegetation with slope and moisture will be examined later.

There appears to be little uniformity of opinion concerning the relation between slope steepness and the steepness of a slip surface. Blong (1974) concluded that there is a strong positive correlation. He noted that in contrast to his findings, Skempton (1953) and Crozier (1969b) recognized either no relation or an inverse relationship. Blong found no relation between landslide type and hillslope morphology while

Crozier observed landslide type to change from flow to translational to rotational with increased slope angle. Strahler (1956) devised the isosinal map, which shows the influence of slope on shearing stress on a slope.

Slope steepness

The threshold angle of slope below which slides do not occur is controlled by the shear strength of the slope material (Deere and others, 1967) and local environmental factors such as moisture content (Blong, 1976). In general terms, Blong calculated a threshold angle in a study area in New Zealand to be greater than or equal to 17°. In the California Coast Ranges, Beaty (1956) observed slides on relatively gentle slopes from 3° to 25°. In western Pennsylvania, Briggs, Pomeroy, and Davies (1975) reported that landsliding was most common on slopes in excess of 25 percent (14°). The average angle for slides in the Clearwater National Forest, Idaho, is 67 percent (34°) (Day and Megahan, 1976). In the San Francisco Bay area of California, Nilsen, Taylor, and Brabb, (1976) and Nilsen, Taylor, and Dean (1976) noted that the majority of slides take place on slopes greater than 15 percent (8.5°). In southern California, Rice and others (1969) placed a 38° threshold on shallow soil failures. In Alaska, Wu (1976) found that the failures concentrated on slopes greater than or equal to 50°. Similarly, Hadley (1964) observed that the rock at the site of the Hebgen slide in Montana maintained a slope of nearly 40°. In Japan, Onodera, Yoshinaka, and Kazama (1974) found that most failed slopes occurred in a slope range of 35°-45°.

From the wide range of slope threshold values, it is apparent that multiple factors control the angle. These are primarily material type, discontinuities, moisture content, and vegetation. The relationship between slope and moisture is shown indirectly in figure 29. In this figure from Dyrness (1967) for the H. J. Andrews Experimental Forest in the Pacific Northwest, frequency of landslide events is plotted for slope angle, slope aspect, and elevation. For this area the south and southwest aspects are the driest, showing that the slope failures which are most common on slopes in excess of 40 percent (22°) also occur more frequently on the more moist slopes where rock weathering and soil thickness are greatest. Dyrness attributed the elevation control to the presence of snow cover above 914 m (3,000 ft), which kept the high rainfall influence at lower levels.

Clay shale slopes characteristically have low threshold angles of stability. Fleming, Spencer, and Banks (1970) reported the following threshold ranges: Bearpaw Shale, 4.9°-7.1°; Claggett Formation, 5.0°-8.3°; Pierre Shale, 4.1°-7.4°; Fort Union Formation, 3.3°-24.1°; and Colorado Group, 7.0°-9.8°. A minimum threshold angle of 8 has been obtained for the London Clay (Hutchinson, 1967) and 9° for the Lias Clay (Chandler, 1970) in the United Kingdom.

In sandstones and interbedded sandstones and shales, slightly higher threshold angles have been reported. In the Appalachian Plateau slides have occurred on slopes ranging from 7°-14° (Gray and others, 1974). In an area in Oregon, Burroughs, Chalfant, and Townsend (1976) found that 71 percent of the failures in similar materials occur on slopes greater than 60°. Blong (1974) reported a lower limit of 20° on sandstones in the Pennines of Europe.

Slope aspect

Slope aspect has been examined by a few investigators as a factor in landslide occurrence. Beaty (1956) reviewed the literature on the subject and found that views ranged from greatest susceptibility on north- and east-facing slopes from snowmelt to south-facing slopes because of lack of vegetation to no influence at all. Where there is slope aspect control over the amount of moisture available, whether as a control over tree growth or depth of weathering, stability may be influenced by such control. The interaction of aspect, moisture, and landslide frequency is illustrated in figure 29 as described in the preceding section.

Of 112 slides studied by Beaty (1956) in an area east of San Francisco Bay in California, 70 percent occurred on slopes with aspects in "northerly" and "easterly" directions. The aspect influence was greatest during slide activity from cyclonic or regional-type storms rather than localized cloudbursts. Beaty found "little" correlation between aspect and vegetative cover. However, none of the slopes examined were timbered.

In later work in southern Alberta, Beaty (1972a, b) reported that of 124 slides studied in the Bearpaw Shale, 87 percent occurred on north, northeast, east and southeast slopes. If the southeast-facing slopes were removed, 75 percent of the total remained in the north to east directions. The distribution of slides resembles the reverse of the wind rose for the area.

Beaty (1972a) attributed the slide frequency to higher moisture occurring in the north to east aspect areas, from greater accumulated drifted snow, from prevailing westerly winds, and from less insolation. Beaty referred to both factors as microclimatic factors. He noted that the far from random sliding in the Bearpaw Shale must be accounted for in part by these microclimatic factors in addition to the standard engineering properties of the material or stream undercutting of slopes.

Chandler (1970) observed east-west-oriented valleys in Lias Clay with asymmetrical slopes. The gentler south-facing slopes were assumed to have formed from more numerous freeze-thaw cycles. The steeper north-facing slopes have more slides, higher moisture, and fewer freeze-thaw cycles.

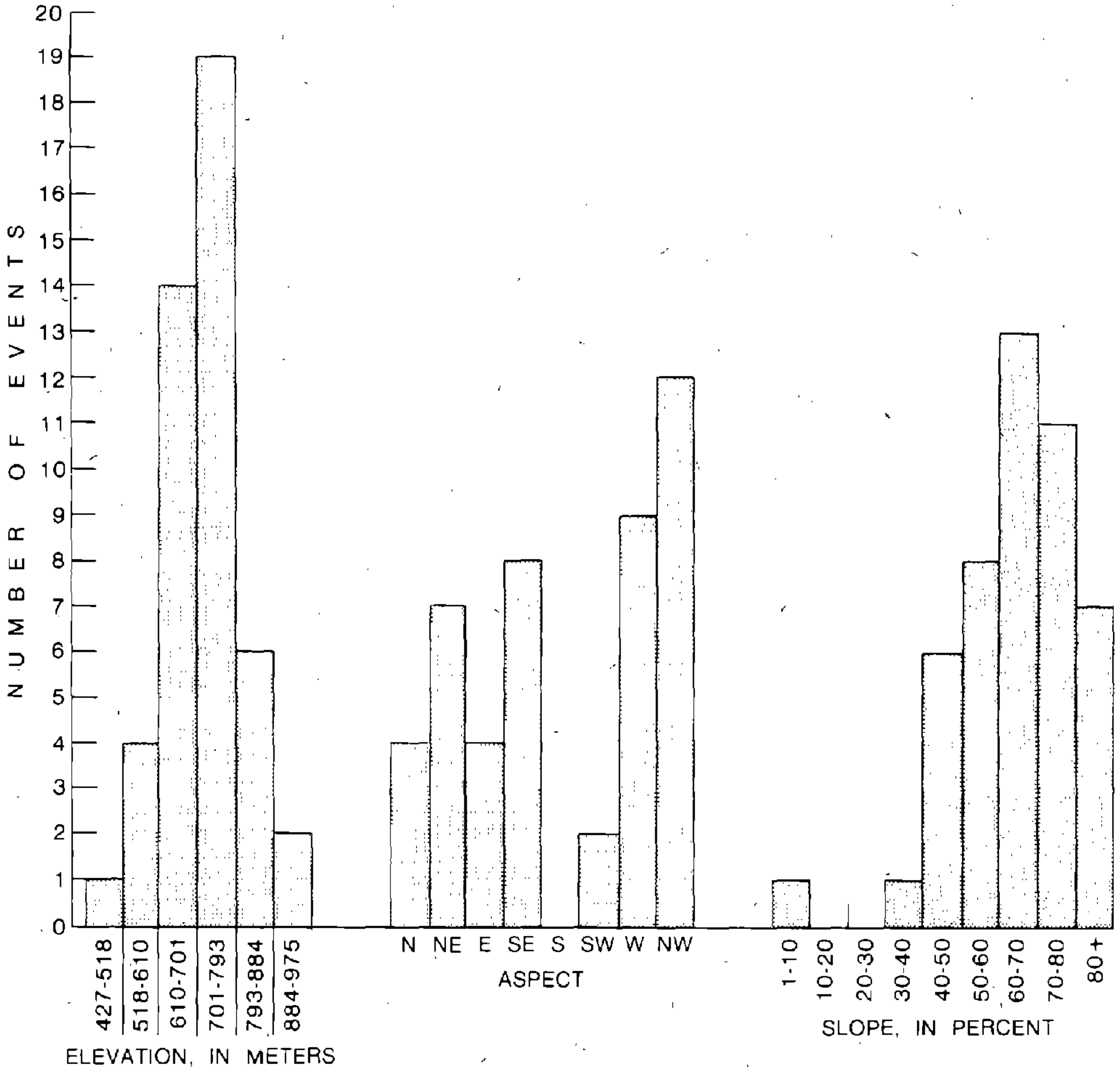


Figure 29. Slope failure frequency vs. elevation, aspect, and slope. Mass soil movements in the H. J. Andrews Experimental Forest. Modified from Dyrness, 1967.

Denness and Cratchley (1972) observed similar asymmetrical valleys in selected clay slopes in southern England to those examined by Chandler (1970). However, they disputed the conclusion that climatic conditions were influential in the development of asymmetry. They stated that asymmetry was caused by regional dip of the clay beds which was similar to that in Chandler's area.

Slope shape

The influence of slope shape on instability has been reported by Waltz (1971) for two counties east of San Francisco Bay in California. All slides studied were on natural slopes in soil and none exceeded 2 m in thickness. None resulted from undercutting of the slope by either natural or manmade causes. Waltz found that the most stable slopes were those where the slope was convex upward both across the slope and downslope. The better stability was attributed to improved runoff of surfacactors in addition to the standard engineering properties of the material or stream undercutting of slopes.

Chandler (1970) observed east-west-oriented valleys in Lias Clay with asymmetrical slopes. The gentler south-facing slopes were assumed to have formed from more numerous freeze-thaw cycles. The steeper north-facing slopes have more slides, higher moisture, and fewer freeze-thaw cycles.

Vegetation

General

The amount and kind of vegetation on a slope are factors to be considered when investigating the causes of slope instability. An inverse relationship between frequency of slides and the size and density of vegetation has been recognized by Rice, Corbett, and Bailey (1969). Gray (1970) summarized the ways in which vegetation affects the balance of forces on a slope as follows: (1) mechanical reinforcement of slopes from root system development; (2) slope surcharge; (3) wind stresses on trees; (4) root wedging to form cracks and fissures; and (5) modification of soil moisture distribution and pore pressure amounts.

On unlogged slopes in coastal Alaska, Swanston (1969) concluded that tree blowdowns, the dynamic loading of soil by wind forces, and the sudden addition of surcharge by rockfalls in addition to a rapid increase in moisture were the principal causes of landslides. The effect on slope stability from the loss of vegetative cover by logging or overgrazing has been examined by several investigators (Flaccus, 1958; Bishop and Stevens, 1964; Crozier, 1969a; Gray, 1970; Brown and Shen, 1975). Gray performed an extensive search of the literature, including several relatively inaccessible sources, and concluded that vegetation plays an important soil protective role.

Influence of root systems

Burroughs, Chalfant, and Townsend (1976) stated that living tree roots anchor shallow soils to steep slopes especially in areas where seasonal storms cause ground-water levels to rise quickly. Wu (1976) measured root strength of various root densities for use in stability calculations. Bishop and Stevens (1964) attributed landsliding in southeastern Alaska to the loss of mechanical support from root systems resulting from timber cutting and overgrazing. Root systems of trees and other vegetation were considered by Swanston (1974) to be the dominant factor in the shear strength of extremely steep slopes in Western United States. In general, root systems serve as cohesive binders and where they penetrate the soil zone to the substrate they provide an effective stabilizing influence (Swanston).

The deterioration of root systems after logging has been reported as an important factor in decreasing stability. Bishop and Stevens (1964) noted an obvious reduction in shear strength of soil after timber harvest and root deterioration. Brown and Shen (1975) concluded that root decay had a long-term influence on slope stability by decreasing the tenacity of the less cohesive soils and decreasing slope stability. Gray (1970) judged root decay after removal of tree cover to be the most serious factor of those vegetative factors that affect the balance of forces on a slope. The gradual decay would cause a gradual decrease in stability. Tests reported by Swanston (1974) indicate a marked reduction in shear strength of roots sampled in a clearcut area 3-5 years after cutting. This time lag corresponds to the lag between logging and massive debris avalanching in southeast Alaska.

At an earlier date, Flaccus (1958) disagreed with the prevailing concept that root decay after deforestation was a major factor in increased slope instability. He wrote that it was likely that new root systems from renewed vegetation would counteract the loss in strength over the period of root deterioration. The more recent work cited tends to refute his conclusions.

Slope surcharge from trees

The influence of the added weight on a slope from trees has been viewed differently by various investigators. Flaccus (1958) claimed that mature forests increased susceptibility to slope failure from the added weight to the soil mass which would increase the driving force. Bishop and Stevens (1964) calculated an increase of 2,394 N/m^2 (50 psf) from the weight of tree cover, which is the equivalent of adding 15 cm of soil. They concluded, however, that this increased load would be balanced by an increase in shear strength from the root system. In this regard, Wu (1976) used an average tree load of 3,591 N/m^2 (75 psf) for stability calculations. Gray (1970) calculated that for a given tree load the increase in shear strength along a potential slip surface from the greater normal stress would be larger than the downslope or driving component.

Wind loads on trees

For years, wind loads on trees have been referred to in the literature as contributing to slope failures. In 1958, Flaccus considered wind on trees to be one of several possible triggering mechanisms for landslides. As noted earlier, Swanston (1969) included dynamic loading of soil masses by wind stress on trees as a factor in causing slope failure. Gray (1970) concluded after an extensive literature search that the effect of downslope forces created by the wind on trees on slope stability had never been evaluated. Using experimental data from other investigators on wind drag on model forests in wind tunnels, Wu (1976) calculated the stress exerted by a 80.5-km/h (50 mph) wind. It amounted to only 95.8 N/m^2 (2 psf), a value not likely to exert a strong influence on stability.

Influence of vegetation on soil moisture

Early work relating landslide susceptibility and vegetation-controlled moisture content does not clearly separate the influence of vegetation on moisture-caused instability and root-caused stability. For instance, Gray (1970) has reported Japanese investigations that showed a higher incidence of landslides in shrubland and grassland than in forested areas. In 1956, Flaccus wrote that mature forests increased infiltration and reduced runoff in addition to adding weight and reducing shear strength. More recently Crozier (1969a) indicated that clearance of brush subjected soils to greater ranges of moisture content. In none of this work was plant transpiration or transfer of soil water to the atmosphere discussed.

Gray (1970) reported that trees deplete soil moisture and create negative pore pressures by transpiration. In addition, trees intercept rainfall in the tree cover and in ground litter, further reducing infiltration rates. As a result, Gray concluded that a forested slope will not reach critical saturation as rapidly or have as high pore pressures from high-intensity storms as denuded slopes. Studies of others reported by Gray have shown that low vegetation is nearly as effective in reducing moisture content as old growth timber.

Effect of forest clearcutting on stability

The effect of clearcutting of forest land on slope stability has been the subject of considerable research. Bishop and Stevens (1964) reported that the number of slides and area affected by slides in southeastern Alaska increased as much as fourfold over a 10-year period after logging (fig. 30). In the same area, Wu (1976) reported much greater landslide frequency after clearcutting.

After an extensive search of the literature, Gray (1970) concluded that there is a definite cause-and-effect relationship between forest clearcutting and slope instability. He found that forests play an important soil protective role and that clearcutting can promote not only soil erosion but also deep-seated failures. This is accomplished by altering the soil moisture content and through deterioration of root systems. The influence of increased soil moisture from clearcutting is most critical for the first year after cutting, as the return of vegetation to an area is quite effective in reducing moisture through transpiration.

A similar cause and effect relationship was noted by Swanson and Dyrness (1975) who documented the impact of forest clearcutting on slope instability in the H. J. Andrews Experimental Forest in Oregon over a 25-year period. The number of slides in clearcut areas was 2.8 times greater than in forested areas having the same slope materials. An increase in slide activity was noted between the time of root system decomposition and establishment of new root systems by returning vegetation. Most slides occurred in the first 12 years after cutting indicating the time required for the return of stabilizing vegetation.

Brown and Shen (1975) analyzed the various factors involving vegetative control of stability such as surcharge, wind load, and root strength. They have separated the impact of clearcutting into short-term and long-term effects. In the short term, clearcutting increases stability through reduction in surcharge from trees and elimination of wind effects. In the long term, decay of root systems decreases stability and the increase in water content from the drop in evapotranspiration decreases stability.

Overgrazing and fires may accomplish the same end result as clearcutting, as there is moisture control from grass and brush similar to that of trees (Gray, 1970). Crozier (1969a) reported that large-scale and rapid clearance of brush from slopes in New Zealand over the past 120 years has made the soil more subject to changes in moisture content and breakdown of soil structure, with resulting slope failures. Excessive livestock grazing was given by Bishop and Stevens (1964) as one cause of loss of root support and subsequent landsliding.

Two investigations are notable by indicating that deforestation has little or no effect on stability. Gray (1970) and Brown and Shen (1975) reported on work by Ellison and Coaldrake in Australia in which creep rates were higher for tree-covered slopes than those covered by sod. Gray noted that the shallow-rooted rain forest in the area could account for the observed results. As noted earlier, Flaccus (1958) claimed that forests increase instability.

Slope aspect influence on vegetation

As noted by many in areas of limited or seasonal rainfall or snowfall, tree growth is greater on north-facing rather than south-facing slopes. Beaty (1956) found this to be true in the California Coast Ranges. He cautioned after examining the work of others that more is involved in stability than just vegetation. An excess in moisture on north-facing slopes is one example.

Greatest frequency of sliding in southeastern Alaska was found on south-facing slopes by Bishop and Stevens (1964). Tree cover was less on these slopes, with a resulting reduction in mechanical support from root systems. In southern California, Rice, Corbett, and Bailey (1969) observed that brush-covered north-facing slopes were more stable than those with southerly aspects.

Multiple factors involved in landsliding

General

The interaction of material type, discontinuities, slope, moisture content, weathering, vegetation, and other factors is a pervasive theme in much of the geotechnical literature. In the preceding sections the interdependence of the factors has been noted as the specific factors were discussed. Some investigators have listed those factors that have combined to cause instability.

Selected factor combinations

Varnes (1958) listed factors that contribute to high stress (or driving force) on slopes. These are erosion, slope, surcharge, lateral pressure such as from water or clay swelling, human activity, and transitory forces such as from earthquakes and blasting. He also has provided a list of factors that contribute to low shear strength. Those related to an initial state are composition, texture, and gross structure. Secondary factors are from weathering and other physicochemical reactions such as physical disintegration, hydration of clays, base exchange of clays, pore-water pressure, and changes in structure as in fissuring and remolding.

In surveying the causes of landslides during highway construction in British Columbia, Brawner (1959) noted several factors. They were characteristics of the stratigraphy, structure of the underlying soil and rock, topography, surface and subsurface water, climate, and vegetation.

In the Pacific Northwest, Dyrness (1967) compiled a combination of material type, moisture, previous failure, fracturing and weathering of bedrock, and slope steepness and aspect. Also in 1967, Engelen listed slope steepness, lack of trees, and heavy precipitation for several days as the key factors in sliding in northern Italy.

Krinitzsky and Kolb (1969) examined the geological factors that influence the stability of clay shale slopes. Their list, which includes more than geological factors, is quite comprehensive. Included in it are mineralogy of clays, degree of lithification, presence or absence of failure-susceptible layers, presence of resistant layers, toe erosion, relaxation from an overconsolidated state, depth of weathering, topography, precipitation, access for water to subsurface, presence of permeable layers, high pore pressures, softening by ground water, dip of beds, intensity of jointing, and rock fabric.

In a shale section with thin limestone beds in north-central Illinois, Du Montelle, Hester, and Cole (1971) listed percentage of expansive clay, percentage of weathered to unweathered shale, slope, and climate as the principal factors.

While not a study of slope stability per se, the work by Brekke and Howard (1972) on stability of tunnels is of value because of the many factors that also act on slopes. Their list included method of excavation, orientation of opening and related orientation of discontinuities, width of fault zones, frequency, character, and orientation of adjacent joint sets, rock competence, presence and kind of gouge material, in situ state of stress, and water regime.

Lastly, the review of landslide causes made by Radbruch-Hall and Varnes (1976) is of interest because it brings together factors from many areas. A summary of the factors includes lithology, climate, topography--including slope angle, vegetation, altitude, structure, tectonic or structural history, and the works of man. The reader is referred to this excellent summary for more details and the sources of information used.

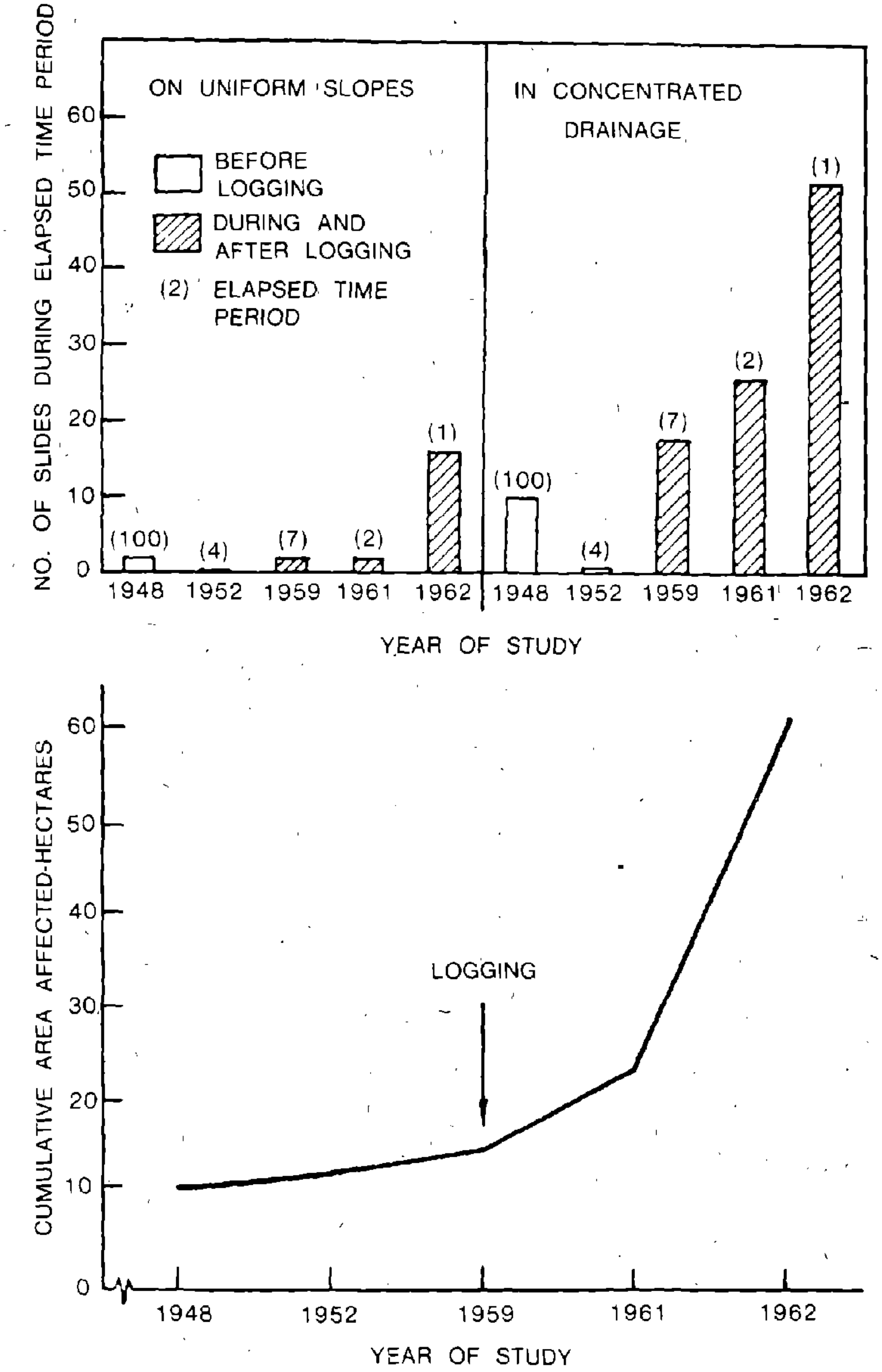


Figure 30. Frequency and area of slides before and after logging in Maybeso Creek Valley, Alaska. Modified from Gray, 1970. Reproduced with permission from the Association of Engineering Geologists.

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.[*]

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements which affect the quality of the human environment. The ultimate goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge of materials properties and technology to fully utilize available naturally occurring materials, to develop extender or substitute materials for materials in short supply, and to devise procedures for converting industrial and other wastes into useful highway products. These activities are all directed toward the common goals of lowering the cost of highway construction and extending the period of maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural designs, fabrication processes, and construction techniques, to provide safe, efficient highways at reasonable cost.

6. Prototype Development and Implementation of Research

This category is concerned with developing and transferring research and technology into practice, or, as it has been commonly identified, "technology transfer."

7. Improved Technology for Highway Maintenance

Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.

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[*] The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.