Slope Stability
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Chapter 5

The Implications of Joints and Structures for Slope Stability

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5.1 INTRODUCTION

Rocks and soils typically contain many structural weaknesses which significantly reduce the shear strength of the mass below that of the intact material. Such discontinuities also have a controlling influence on the movement of groundwater through the mass. Hoek (1971) and other authors have pointed out that were it not for the presence of such discontinuities, even weak rock could stand vertically to heights of several hundred metres. Discontinuities result in markedly anisotropic engineering properties as in the case illustrated in Figure 5.1 where sliding has occurred along a planar joint through otherwise strong rock. The significance of discontinuities is sometimes less readily apparent as in the case reported by Douglas and Voight (1969) who noted a preferred orientation of microfractures through granite resulting in a directional variation in compressive strength. Where joints or structures in the rock mass are adversely orientated, site investigation for assessing slope stability must be aimed at identifying those blocks of rock that might move, taking into consideration the existing or proposed geometry of the slope. Methods of analysis must be capable of dealing with the sliding of such blocks possibly on several surfaces at once. The main difference between such 'rock' slope analytical methods and the generalized 'soil' methods discussed in chapter 2 is that for the former the geometry of failure is controlled by pre-existing planes of weakness. Generalized methods involve the search for a critical failure surface through material that is taken to be isotropic in strength. The labelling of methods as 'rock' and 'soil' is not entirely satisfactory because soil masses may contain adverse joints (particularly soils formed through weathering of rock) and will require 'rock' methods of investigation and analysis (Deere and Patton, 1971; Koo, 1982) and conversely, closely jointed rock through which irregular failure surfaces may develop must be analysed using generalized 'soil' methods (Hoek and Bray, 1981).

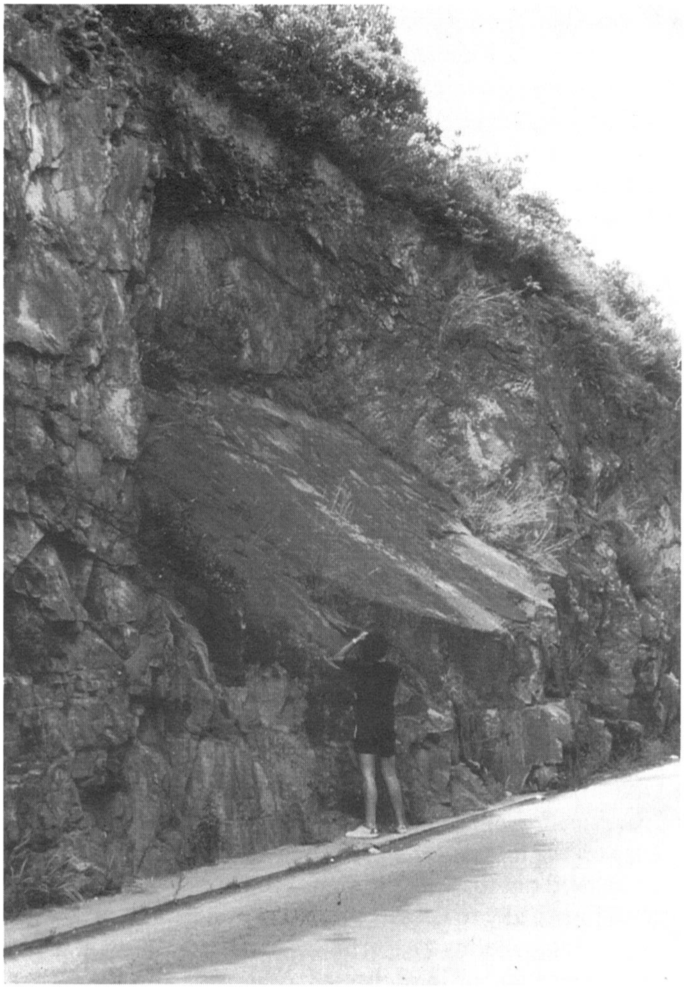


Figure 5.1 Single adverse joint controlling stability through strong rock

This chapter is arranged in five sections. The first section deals with the nature and origin of discontinuities in soil and rock masses and the second with methods for investigating and describing the geotechnically significant features of discontinuities. Shear strength of individual discontinuities and of closely jointed rock or soil masses in terms of effective stress is covered in the third section and methods for determining the hydrogeological conditions in the fourth. The final section discusses the analysis of the 'geotechnical model' determined by the methods described in the earlier sections.

5.2 ORIGIN AND CLASSIFICATION OF DISCONTINUITIES

An investigation into the extent, orientation, and distribution of discontinuities for a particular slope will only be truly effective when the geological nature of the structures is taken into account. Recognition of discontinuity type will almost certainly allow properties to be predicted and extrapolated with more confidence than could otherwise be justified. A few authors, notably Deere and Patton (1971) and especially Piteau (1970, 1973), have emphasized the importance of geology in extrapolating and assessing the importance of particular discontinuities. Piteau (1973) in presenting a well-documented case where a careful appraisal of jointing allowed patterns to be predicted in the rock mass away from the sampling point comments 'the analyst must assess whether the location at which the joint

Table 5.1 Geotechnical Classification of Discontinuities Common to all Rock and Soil Types

Discontinuity Type	Physical Characteristics	Geotechnical Aspects	Comments
Tectonic joints	Persistent fractures resulting from tectonic stresses. Joints often occur as related groups or 'sets'. Joint systems of conjugate sets may be explained in terms of regional stress fields.	Tectonic joints are classified as 'shear' or 'tensile' according to probable origin. Shear joints are often less rough than tensile joints. Joints may die out laterally resulting in impersistence and high strength.	May only be extrapolated confidently where systematic and where the geological origin is understood.
Faults	Fractures along which displacement has occurred. Any scale from millimetres to hundreds of kilometres. Often associated with zones of sheared rock.	Often low shear strength particularly where slickensided or containing gouge. May be associated with high groundwater flow or act as barriers to flow. Deep zones of weathering may occur along faults. Recent faults may be seismically active.	Mappable, especially where rocks either side can be matched. Major faults often recognized as photo lineations due to localized erosion.
Sheeting joints	Rough, often widely spaced fractures; parallel to the ground surface; formed under tension as a result of unloading.	May be persistent over tens of metres. Commonly adverse (parallel to slopes). Weathering concentrated along them in otherwise good quality rock.	Readily identified due to individuality and relationship with topography.
Lithological boundaries	Boundaries between different rock types. May be of any angle, shape, and complexity according to geological history.	Often mark distinct changes in engineering properties such as strength, permeability and degree and style of jointing. Commonly form barriers to groundwater flow.	Mappable allowing interpolation and extrapolation providing the geological history is understood.

Table 5.2 Geotechnical Classification of Discontinuities Characteristic of Particular Rock and Soil Types

Rock or Soil type	Discontinuity type	Physical characteristics	Geotechnical aspects	Comments
Sedimentary	Bedding planes/ bedding plane joints	Parallel to original deposition surface and marking a hiatus in deposition. Usually almost horizontal in unfolded rocks.	Often flat and persistent over tens or hundreds of metres. May mark changes in lithology, strength and permeability. Commonly close, tight, and with considerable cohesion. May become open due to weathering and unloading.	Geologically mappable and therefore, may be extrapolated providing structure understood. Other sedimentary features such as ripple marks and mud-cracks may aid interpretation and affect shear strength.
	Shaley cleavage	Close parallel discontinuities formed in mudstones during diagenesis and resulting in fissility.		
	Random fissures	Common in recent sediments probably due to shrinkage and minor shearing during consolidation. Not extensive but important mass feature.	Controlling influence for strength and permeability for many clays.	Best described in terms of frequency.
Igneous	Cooling joints	Systematic sets of hexagonal joints perpendicular to cooling surfaces are common in lavas and sills. Larger intrusions typified by doming joints and cross joints.	Columnar joints have regular pattern so easily dealt with. Other joints often widely spaced with variable orientation and nature.	Either entirely predictable or fairly random.
Metamorphic	Slaty cleavage	Closely spaced, parallel and persistent planar integral discontinuities in fine grained strong rock.	High cohesion where intact but readily opened due to weathering or unloading. Low roughness.	Formed by regional stresses and therefore mappable over wide areas.
	Schistosity	Crenulate or wavy foliation with parallel alignment of minerals in coarser grained rocks.	Often foliations coated with minerals such as talc, and chlorite giving a low shear strength.	Less mappable than slaty cleavage but general trends recognizable.

data are collected has been subjected to the same geological history of deformation as the location where extrapolation is to be made. If their histories are found to differ, extrapolation is not valid.' Too often the approach to the collection and processing of data is almost purely statistical with only scant regard to the geological history of the site. The emphasis in the rock mechanics literature, certainly in recent years, has been much more concerned with such aspects as statistical sampling errors and idealized joint shapes than with the true characteristics and properties of the various types of discontinuity.

For geotechnical purposes, a discontinuity may be defined as a boundary or break within the soil or rock mass which marks a change in engineering characteristics or which itself results in a marked change in the mass properties. This definition clearly includes such features as lithological boundaries, faults, bedding planes, and tectonic joints but also can be stretched to include microstructures such as a preferred orientation of microfractures. In all these cases the orientation and extent of the discontinuity or set of discontinuities could be measured, at least in theory, and would lead to anisotropic behaviour of the rock or soil mass when loaded. Tables 5.1 and 5.2 provide a simple classification of the most common types of discontinuity and list their typical characteristics and geotechnical significance. Table 5.1 deals with discontinuities which are common to all rock types while Table 5.2 lists discontinuities which are restricted to rocks of particular types. It should be noted that the terms discontinuity and joint are often used synonymously, joints being the most common type of discontinuity.

5.2.1 Tectonic Joints

Tectonic joints which are formed as the result of orogenic stress in the earth's crust are common to all rock types and may even be found in recent sediments (Burford and Dixon, 1978). They often occur in distinct 'sets', a term which is sometimes defined for rock mechanics purposes as a series of parallel joints (Herget, 1977) such as those illustrated in Figure 5.2. Structural geologists may also use the term 'set' to describe a group of joints of common origin which need not be parallel to one another (Hobbs *et al.*, 1976). The geometrical relationship between sets may sometimes be interpreted with respect to a regional stress pattern or a local geological structure such as a fault or a fold (see Price, 1966). Other joints may be non-systematic, often forming cross-joints between the systematic joints. In some cases well defined sets of joints are recognized that are apparently independent of tectonic stress fields (de Sitter, 1964; Fookes and Denness, 1969). Similarly, joints which have the appearance of tectonic joints are found in rocks which have been neither folded nor faulted. De Sitter (1964) comments that in such cases the joints themselves are indicative of a certain level of tectonic activity. Price (1959, 1966) attributes all tectonic jointing to the retention and subsequent release of strain energy in the rock.

Where the geometrical pattern of joints may be explained by reference to overall geological structure, then extrapolation and interpolation of joint data may be carried out with reasonable confidence. In many cases, however, it is very difficult to relate the joint pattern to a known cause and then a more cautious approach must be adopted. As noted in Table 5.1, joints formed as the result of shear stresses are commonly less rough than joints formed under tension and might therefore be expected to exhibit lower shear strengths.

Figure 5.2 Parallel set of tectonic joints

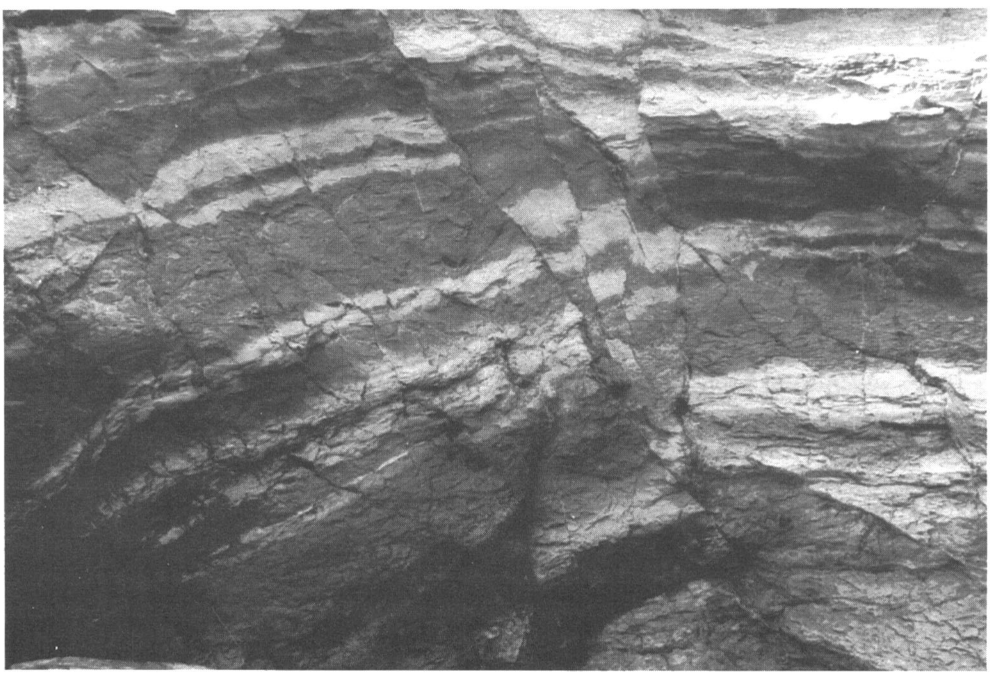


Figure 5.3 Faults through bedded sedimentary rocks

5.2.2 Faults

Faults such as those illustrated in Figure 5.3 are fractures along which displacement has taken place. They may occur singly or in groups forming shear zones. Faults are often discordant to other structures, particularly lithological boundaries and bedding, and are much rarer discontinuities than are joints. The distinction between joints and faults is strictly defined by some authors (see Fecker, 1978, for example), but in practice it is less easy to separate them. For example, discontinuities which exhibit slickensides, clearly indicating that displacement has taken place, are often regarded as joints rather than faults where they occur as sets throughout the mass rather than as isolated shear planes. Slickensides on such joints are probably indicative of minor internal movements within the mass (Skempton *et al.*, 1969). Similarly, where a zone of intense and discordant jointing is encountered, the term shear zone or fault might be employed even though there is no clear evidence of displacement along the zone.

Faults often cause geotechnical problems, not only in slopes but also in foundations and especially tunnels due to their association with shattered and sheared rocks and the fact that they often have relatively high permeabilities and may carry a lot of groundwater (see discussion by Sharp *et al.*, 1972). Weathering is often concentrated along faults and the rock may show signs of hydrothermal alteration. Earthquakes are associated with recent faulting and if it is suspected that faults may be seismically active, this may be checked by instrumentation, literature review or field observation (see Clark *et al.*, 1972; Sherard *et al.*, 1975; Ben Menahen, 1976; Donovan, 1978, for example). Observations of offset rivers, truncated spurs, and other geomorphological features may all indicate that faults are active. Faults are recognized by many features, particularly the displacement of recognizable beds and by crushed zones of rock (fault breccia and mylonite). Major faults are often preferentially eroded and trends can be recognized from air photographs.

5.2.3 Lithological Boundaries

Geological boundaries between different soils and rocks often mark sharp changes in engineering properties and are, therefore, significant for stability analysis. This is reflected by the often close correspondence between geomorphology and underlying rock type, a fact which geologists exploit to interpolate boundaries between exposures. Geological boundaries, as distinct from many other discontinuities, are readily identified in drill core and can be interpolated with reasonable confidence. The same cannot be said for weathering zones through the rock which may vary laterally in an unpredictable manner. Dykes or sills commonly form barriers to groundwater as in the case illustrated in Figure 5.4, where a perched water table developed above a dolerite dyke through decomposed granite and caused a landslide (Hencher and Martin, 1984). Where the lithological boundary marks a change from 'soil' to 'rock', then different methods of analysis will be required for the two component materials of the slope.

5.2.4 Sheeting Joints

Sheeting joints in hard rocks are typically rough and extensive, running parallel to the topography of the present-day, or that of the recent geological past. An example of topography controlled by extensive sheet jointing is given in Figure 5.5. There is some debate (Twidale,

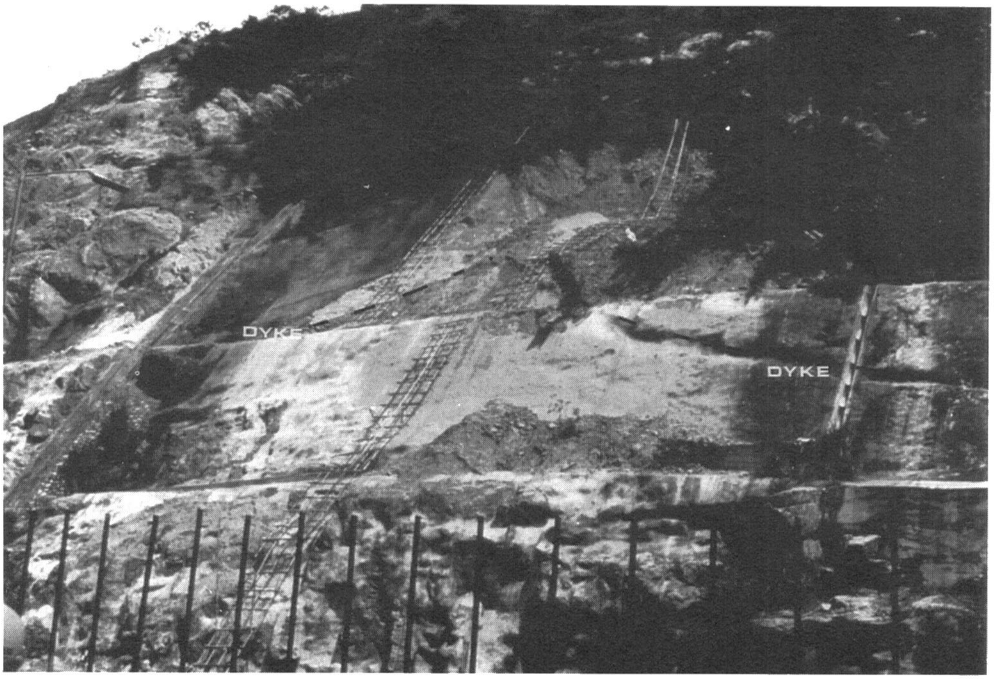


Figure 5.4 Landslide caused by perching of water above dyke

Figure 5.5 Sheeting joints through granite

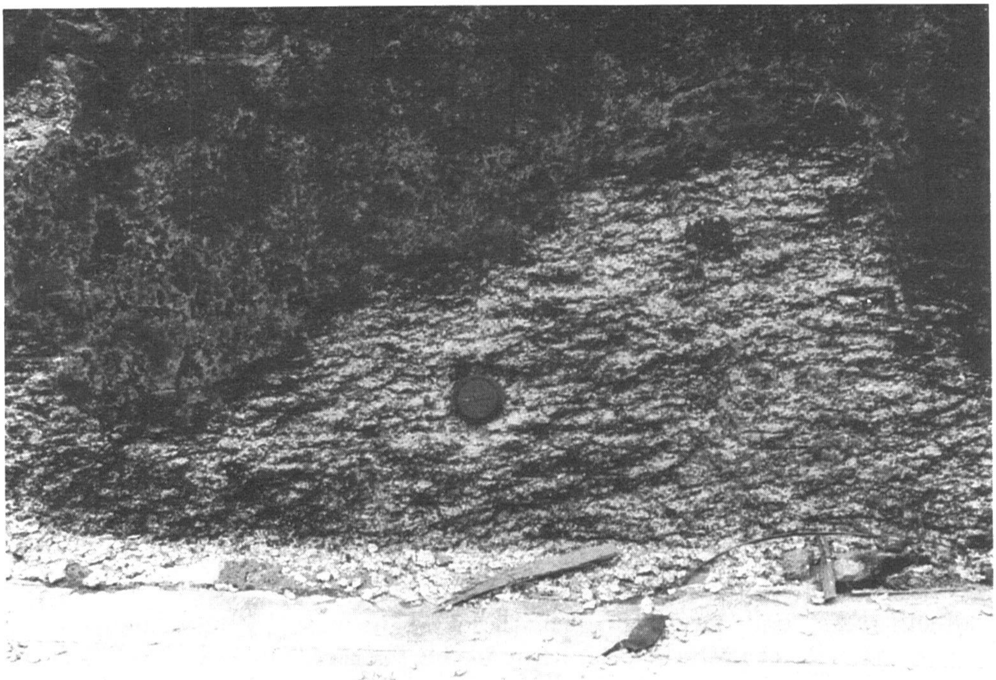


Figure 5.6 Parallel microfractures through weathered granite

1973) as to which came first, the jointing pattern or the geomorphological form. The majority of workers, however, attribute such joints to tensile stresses following unloading due to erosion or perhaps the removal of ice load. A discussion of the mechanism envisaged is given by Nichols (1980).

Sheeting joints occur in all lithologies and have been reported in quite recent sediments (Fookes, 1965; Skempton *et al.*, 1969). Characteristically they decrease in frequency with depth below ground surface and may cross geological boundaries. Rather surprisingly, they may be found in rocks that are otherwise quite highly jointed. It might have been expected in such cases that tensile loads could have been accommodated by movement along existing joints without need for further fracturing of the intact rock. Such occurrences might indicate that the 'tectonic' joints only fully developed *after* the sheeting joints as a result of weathering and stress relief.

A common feature in weathered rocks in Hong Kong is a mass fabric of microfracturing parallel to topography as illustrated in Figure 5.6, and this has similarly been attributed to unloading (Hencher and Martin, 1982). Examples are known of failures occurring along such orientated microfractures in the same way as along other discontinuities.

5.2.5 Bedding Planes and Bedding Plane Joints

Bedding planes in sediments mark either a change in sediment type or a hiatus of deposition. While imposing on the rock or soil mass a marked anisotropy, bedding planes are often closed and retain a strong cohesion. Because of unloading or weathering, however, they may open up to form bedding plane joints. In drill core it is sometimes difficult to tell whether the degree of jointing observed is representative of the rock in the ground or due to drilling disturbance. Most bedding planes are fairly flat and smooth but sedimentary features such as ripple marks or load casts associated with bedding surfaces may lend surfaces a rough texture. Planes commonly extend over wide distances although beds may pinch out due to the coalescence of different planes.

5.2.6 Fissures

Fissures are a common feature of unlithified clays and silts, are generally of limited length, and often appear essentially random in orientation although some authors have managed to relate them to specific stress fields (Skempton *et al.*, 1969; Fookes and Denness, 1969; McGown *et al.*, 1974). They are generally thought to develop during diagenesis and are best described as a mass feature of the soil. The presence of such fissures has been shown to influence the strength and permeability of soils considerably (Skempton and La Rochelle, 1965).

5.2.7 Cooling Joints

In igneous rocks, joints form as the magma cools. The most striking examples are the columnar joints formed perpendicular to the cooling surface in lavas (see Figure 5.7). The regular pattern of such jointing allows them to be readily accounted for in stability analysis.

Figure 5.7 Columnar jointing through rhyolite

Cooling joints in larger intrusions often form a doming pattern together with cross-joints (see discussion in Price, 1966; Gamon and Finn, 1984). Layering of igneous rocks due to density segregation of minerals on cooling can also be found as well as flow banding and such features will result in anisotropic engineering behaviour.

5.2.8 Metamorphic Fabrics

Metamorphic rocks formed under pressure commonly contain well-defined sets of discontinuities. Slaty cleavage, a close pattern of parallel planar discontinuities, is imposed on rocks by regional stresses and results in markedly anisotropic properties (Brown *et al.*, 1977). Phyllites and schists result from higher grades of metamorphism and typically exhibit foliation, the surfaces of which are often coated with minerals such as chlorite, talc and mica which may have low frictional properties. Such foliation is generally more wavy than slaty cleavage and this will contribute to shear strength. Metamorphic fabrics which owe their origins to regional stresses are often mappable over wide areas.

5.3 INVESTIGATION

5.3.1 Introduction

Stability analysis of slopes requires a knowledge of the distribution, geometry, and engineering properties of the discontinuities in the mass. The quantity of data collected, the methods employed, the quality of information obtained and its eventual usefulness will depend on many factors, notably the nature and seriousness of the problem, the accessibility of exposure, the time and cost justifiable for the task, and the experience and local knowledge of the investigator.

There is general agreement (ISRM, 1978; Herget, 1977) regarding the scope of description required to characterize the nature of discontinuities and the main attributes are listed below and illustrated schematically in Figure 5.8.

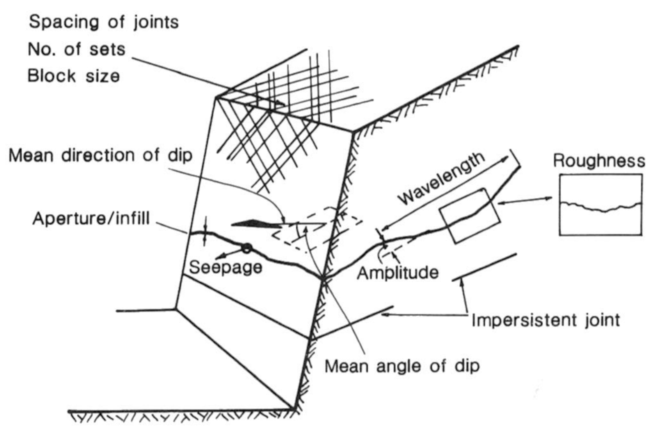


Figure 5.8 Description for the characterization of discontinuities

- Orientation (mean dip and dip direction).
- Spacing
- Persistence
- Roughness and Waviness
- Wall Strength
- Aperture
- Filling
- Seepage
- Number of sets
- Block size

Three main methods are used for obtaining data and will be discussed here, viz:

1. Photographic interpretation and measurement.
2. Mapping exposed faces.
3. Drilling.

Methods for assessing or measuring shear strength and water pressures will be discussed in later sections.

5.3.2 Photographic Methods

Photographic methods are particularly important for collecting data on the stability of natural hillsides but can also provide useful information for more detailed local studies. Trends of major joint sets and faults can often be identified from aerial photographs (Norman and Huntingdon, 1974) and angles of dip may sometimes be measured or estimated using stereographic pairs of photographs or by considering the traces of lineaments crossing variable topography. Allum (1966) points out that the significance of major faults or joints is often more readily appreciated from aerial photographs than in the field. Terrestial photogrammetry may be used for tracing joints in exposures and excavations (Moore, 1974; ISRM, 1978) and good quality photographs are commonly used as a basis for locating discontinuities and recording data in the field (see Starr *et al.*, 1981, for example). Useful though remote methods are for providing data at a relatively low cost particularly where access is difficult, detailed slope stability analysis requires a knowledge of characteristics such as roughness and degree of weathering of particular joints which can only be obtained by inspection. Furthermore, the usefulness of photographic methods is limited by the amount of exposure.

5.3.3 Mapping Exposed Surfaces

Provided that suitable exposures are available, mapping and inspection are the best ways for assessing the characteristics of discontinuities. Qualities such as roughness and persistence can only be determined by observing the lateral traces and surface features of discontinuities and therefore cannot be determined from drill core. Often the best approach to surface mapping is a combination of statistical sampling and concentration on areas of particular importance. Piteau (1973) reports a case where despite good exposure and experienced personnel, structural domains were not recognised in the field and only became apparent following the statistical processing of data.

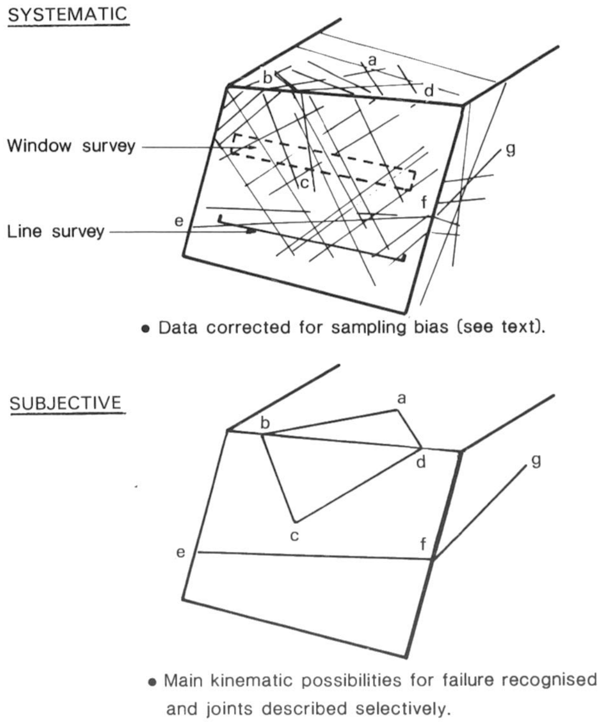


Figure 5.9 Methods for logging exposed rock faces

Systematic methods for collecting data involve logging all discontinuities intersecting selected lines or occurring in 'windows' set out on the exposed face (see Herget, 1977 and Figure 5.9). The use of proformas such as that presented in Figure 5.10 encourages objective and full description. Data may be treated statistically and corrected for sampling bias due to the orientations of the sampling lines (Terzhagi, 1965; Priest and Hudson, 1981; Hudson and Priest, 1983).

A danger of using a totally statistical, 'objective' approach is that the need to make engineering judgements in the field is under-emphasized. Data may be collected with little thought as to their implications and critical but rare data may be overshadowed in analysis by vast quantities of statistically correct but irrelevant data. Many authors caution against an over-statistical approach to collecting geological data (Whitten, 1966; Hoek and Bray, 1981), and at the other extreme a survey may be carried out in a totally subjective manner with only those discontinuities considered of importance being described (see Figure 5.9). Such an approach requires suitably experienced personnel to do the work and most importantly good, relevant exposure. Clearly the most suitable method for collecting data for a particular project will depend upon the nature of the problem, the quality of exposure, the resources available for investigation and the experience and expertise of the personnel involved.

There are certain characteristics of joints that are very difficult to measure even where the exposure is excellent. Impersistence in particular is one of the main factors contributing to slope stability and yet is extremely difficult to investigate. Joints can suddenly die out

GENERAL INFORMATION

Site | Date (Day, Month, Year) | Operator | Discontinuity data sheet No. of

NATURE AND ORIENTATION OF DISCONTINUITIES

Chainage or No.	Type	Dip	Dip direction	Persistence	Aperture	Infilling	Consistency	Roughness	Differentially weathered zone width mm	Waviness wavelength	Waviness amplitude	Water	Remarks

Type
0. Fault zone
1. Fault
2. Joint
3. Cleavage
4. Schistosity
5. Shear
6. Fissure
7. Tension crack
8. Foliation
9. Bedding

Dip, Dip direction
(Expressed in degrees)

Persistence
(Expressed in metres)

Aperture
1. Wide (>200mm)
2. Mod. wide (60-200mm)
3. Mod. narrow (20-60mm)
4. Narrow (6-20 mm)
5. Very narrow (2-6mm)
6. Ext. narrow (<2 mm)
7. Tight

Nature of infilling
1. Clean
2. Surface staining
3. Non-cohesive
4. Cohesive
5. Cemented
6. Calcite
7. Chlorite, talc
8. Others-specify

Consistency of infilling

Soil strength
1. Soft
2. Firm
3. Stiff
4. Hard

Rock strength
5. Weak
6. Mod. strong
7. Strong
8. Very strong

Roughness
1. Polished
2. Slickensided
3. Smooth
4. Rough
5. Defined ridges
6. Small slope
7. Very rough

Waviness
Express wavelength and amplitude in metres

Water
1. Dry
2. Seepage
3. Slight flow <0.1 1/sec
4. Mod. flow 0.1-1 1/sec
5. High flow >0.1 1/sec

Figure 5.10 Proforma for the collection of discontinuity data. Record characteristics of each discontinuity encountered at chainages along survey line.

Figure 5.11 Impersistent joint in volcanic rock

as in the case illustrated in Figure 5.11, or may contain rock bridges that can provide considerable cohesional strength. Persistence can be estimated realistically only by observing the general characteristics of sets of joints in exposed faces. In practice, joints are normally regarded as persistent unless proved otherwise and it is accepted that design might err on the conservative side. Some authors have proposed mathematical methods to deal with impersistence (Jennings, 1970; Einstein *et al.*, 1983) and the matter is discussed at length by ISRM (1978).

Roughness and waviness (the distinction between which is not always clear) are essentially small-scale textural surface variations and larger scale undulations of the discontinuity surface respectively, and can only be measured on actual exposed joint surfaces. Both are important for defining the attitude of joints and for their contribution to shear resistance as discussed later. If the exposure is good enough, plates of various diameters may be used to express the variation in attitude of the joint over different base lengths (Fecker and Rengers, 1971; Hoek and Bray, 1981). In some studies, if the exposure at the actual site is not good enough for roughness to be measured, then exposures of similar joints in the vicinity perhaps in quarries or at the coast may be studied and the results interpreted intelligently (Richards and Cowland, 1982). This type of survey is imperative if rational decisions based on estimate of shear strength are to be taken.

Seepages in exposures together with evidence such as lush vegetation, localized weathering, and stains on joints in exposed faces can all be important for understanding the hydrogeological conditions. A detailed knowledge of groundwater

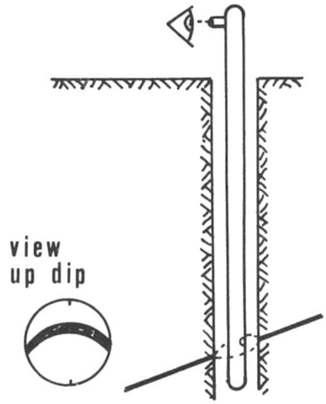
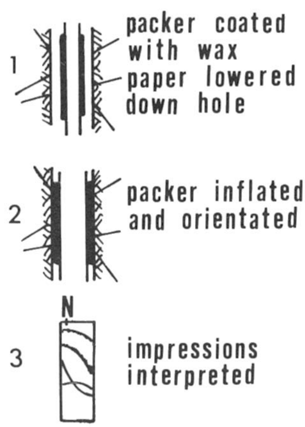
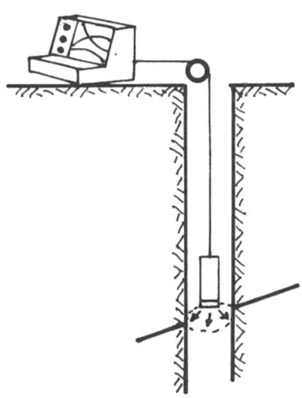
Methods		Comments
Borehole Periscope		• Simple device that works successfully to about 30 m. • Orientation, dip and depth can be measured. • Aperture/infill can be observed/photographed. • Water ingress seen if hole pumped dry.
Impression Packer		• Excellent impressions achieved. • Joint characteristics other than orientation are unknown unless reference is made to core. • Care must be taken that compass is fully set before withdrawal.
Television Camera		• Video record can be made of hole • Some systems only suitable for general viewing of quality rather than detailed measurement of joint data.

Figure 5.12 Downhole methods for measuring the orientation of discontinuities

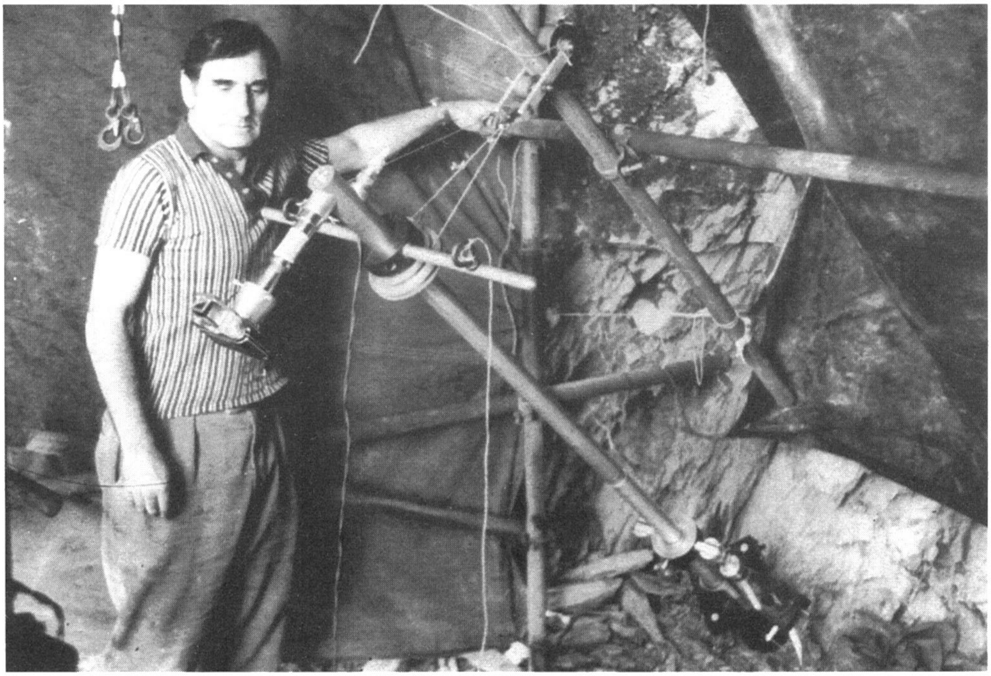


Figure 5.13 Borehole periscope in inclined hole being used to take a photograph of a discontinuity

conditions, however, can only come in the long-term from instrumentation which can cope with short-term variations in pressure as discussed later (Patton, 1983; Cowland and Richards, 1985).

5.3.4 Drilling

Discontinuities within the rock mass can be sampled only by drilling or by excavation. In rock, core drilling is generally carried out using either double- or preferably triple-tube barrels to reduce disturbance. The core obtained can be logged to record the frequency of joints and samples may be selected for testing. The sampling size is, however, generally much too small to allow a representative roughness to be assessed.

Orientation of joints in rock core is not easy to measure. As drilling proceeds core may be rotated and only by very carefully piecing together of core (with 100% recovery) or by reference to some feature of known orientation such as regional bedding can the dip direction of joints be estimated. Various methods such as scratching a side during drilling are used for orientating core and these are discussed by Hoek and Bray (1981). Several devices can be used to measure the orientation of discontinuities in the wall of the drill hole after the core has been removed and are illustrated in Figures 5.12 to 5.14. Even if the orientation could be measured accurately, the problem of how representative is the core of the whole rock mass would remain.

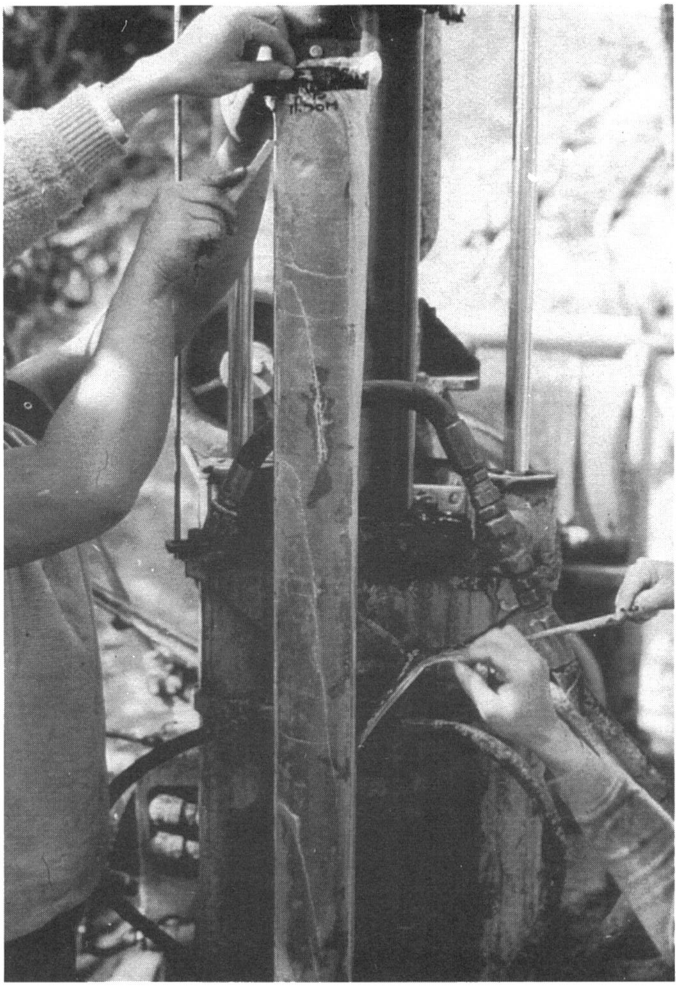


Figure 5.14 Impression packer showing traces of discontinuities. (Photograph by A. Cipullo)

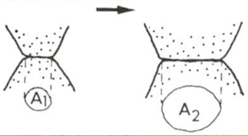
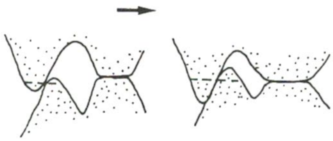
5.4 SHEAR STRENGTH

Slope stability analysis requires knowledge of the shear resistance along potential failure planes. If a failure can occur along discrete joints or joint sets then the shear strength of the individual joints must be assessed. If a more complex failure surface passing in part along joints and in part through intact material is possible then parameters are required that take into account that mode of failure.

It must be emphasized that shear resistance must always be expressed in terms of effective stress to take account of water pressures as discussed in the next section on groundwater.

Major advances have been made over the last two decades (Hoek, 1984) in our understanding of the factors controlling the shear strength of jointed masses and some acceptable methods for measuring or estimating strength have been established and are discussed below.

Table 5.3 Factors Contributing to the Shear Strength of Rock Discontinuities

Increasing normal load  	ADHESION (BASIC FRICTION) —Adhesion over true area of contact (A_1, A_2) • Proportional to normal load • Does not cause dilation • No reduction with displacement • Same for different roughnesses
Increasing normal load 	INTERLOCKING AND PLOUGHING (ADDITIONAL FRICTION COMPONENT) —Surface texture component • Proportional to normal load • Does not cause dilation • Reduces with displacement with production of debris • Increases with rougher surface texture
 	OVERRIDING • Work done due to dilation • Purely geometrical effect • Uphill movement results in increased shear strength (as measured in the horizontal)
	COHESION – Shearing of major asperities • Distinct from interlocking (see above) in that it is not proportional to normal load • Does not cause dilation • Lost after peak strength • Relative contributions of overriding and cohesion depend on stress level

5.4.1 Shear Strength of Individual Joints

The term shear strength is used to describe the total resistance against shearing developed along a surface and the main contributing factors for persistent joints are illustrated in Table 5.3. These factors can be separated mathematically although in most cases several factors will operate at the same time—for example shearing of asperities will often be accompanied by dilation. The term friction is restricted to describe the resistance proportional to normal load (a combination of factors 1 and 2 in Table 5.3). Patton (1966) demonstrated that friction and the effects of roughness on shear strength of rock joints could be separated in a practical way and Barton (1971, 1973) similarly distinguishes between these two components in his empirical equation for rock shear strength based on carefully scaled model tests:

$$\tau_p = \sigma_n \tan \left[JRC \log_{10} \left(\frac{JCS}{\sigma_n} \right) + \Phi_b \right]$$

where $\tau_p =$ peak shear strength
$\sigma_n =$ effective normal stress
$JRC =$ joint roughness coefficient
$JCS =$ joint wall compressive strength
$\Phi_b =$ "basic" friction angle

Barton discusses the measurement of JRC and JCS in the papers referenced above and in later papers (Barton and Choubey, 1977; Barton and Bandis, 1980) and claims that his equation can predict shear strength extremely closely. Recent work by Bandis (Bandis *et al.*, 1981) has demonstrated the importance of scale effects in determining JRC and JCS values. The Φ_b value is the frictional resistance for 'flat or residual surfaces' and is a convenient reference point obtained from direct shear tests on planar rough-sawn or sand blasted surfaces of rock.

The preferred method of the author for deriving realistic shear strength parameters involves direct shear testing of representative samples of natural joint surfaces and interpreting the results obtained with respect to field-scale roughness. An advantage of testing natural surfaces is that the influence of surface coating and natural surface texture may be investigated, the significance of which might not be apparent were saw-cut surfaces of rock to be used. For example, during the investigation of a rock slope failure on a shallowly dipping plane in 1982, careful shear testing demonstrated the importance of a thin coating of chlorite which, at low stresses gave an effective friction angle of only 19 degrees but at higher stresses was less significant due to increasing contribution of the underlying monzonite rock surface. These detailed observations allowed the failure mechanism to be understood.

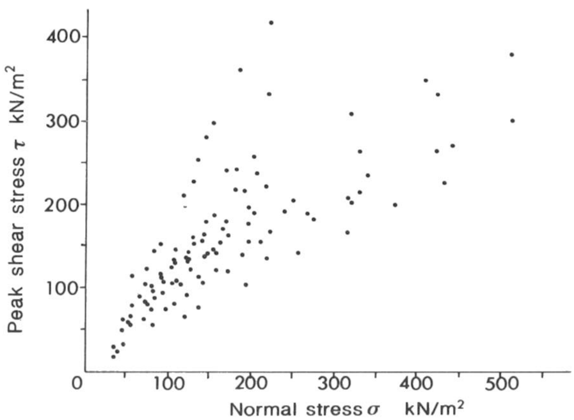


Figure 5.15 Peak shear strengths from multistage tests on rough discontinuities through granite

In carrying out shear tests on small rock samples it must be recognised that individual tests will give different results according to the roughness of each sample. As an example, the results of a series of tests on natural granite joint samples of various roughnesses are presented in Figure 5.15. In the early 1970s, the tendency was to test larger and larger samples, either *in situ* or in the laboratory with the aim of obtaining parameters that would incorporate the effect of larger scale joint roughness. In fact such tests often just give even wider scatter and at much greater expense than do small scale tests of the type reported by Ross-Brown and Walton (1975).

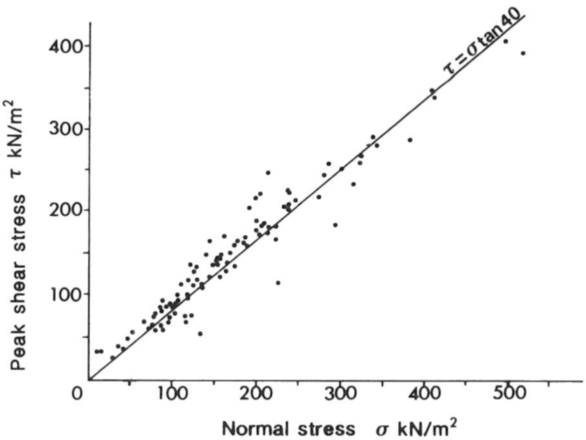


Figure 5.16 Peak shear strengths corrected for dilation and compression during tests

Figure 5.17 Planar tectonic joint through volcanic rock

Direct shear tests on small samples of rock joints can only be expected to provide the following data:

1. The basic frictional resistance for groups of like joints once the effects of sample variability have been removed.
2. Information on the mechanics of shearing of the natural joint that can aid in the selection of a roughness coefficient for the joint *in situ*.

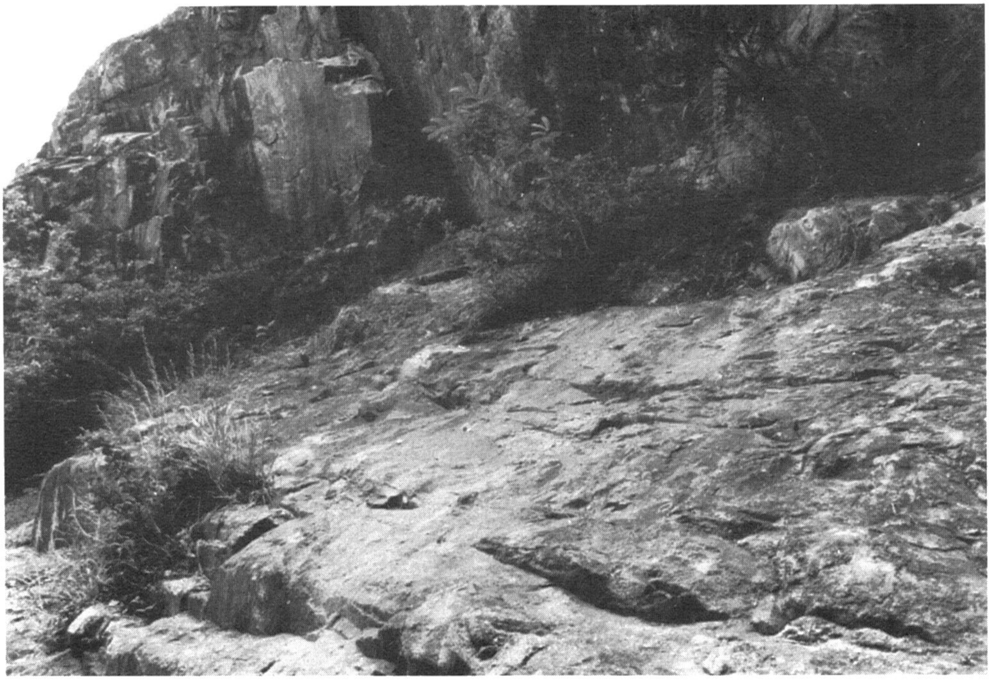


Figure 5.18 Rough sheeting joint through volcanic rock

Interpretation requires the correction of data for the dilation or compression angle at peak strength which allows friction coefficients for effectively planar, 'natural' surfaces to be derived (Hencher and Richards, 1982). Figure 5.16 shows the data of Figure 5.15 corrected for the effects of individual sample roughness and the reduction in scatter is encouraging. The basic friction angle thus obtained was about 6 degrees higher than that for saw-cut surfaces of the same rock.

Once parameters describing the basic frictional strength have been obtained then the effects of field roughness must be taken into account and this is where engineering judgement is required. Examples of a planar tectonic joint and a rough sheeting joint through the same volcanic rock are given in Figure 5.17 and 5.18. A practical method for measuring the roughness of representative joints has been proposed by Fecker and Rengers (1971) and is described well in Hoek and Bray (1981). The method is explained schematically in Figure 5.19.

Different 'orders' of roughness are measured using different diameter base plates, measurements being taken on a grid pattern over the joint surface. It is generally found that the smaller the measuring plate, the greater the deviation from the average angle of inclination of the plane. Reference to the damage caused to asperities during shear tests at the correct stress levels, will aid in deciding the allowable roughness angle that will cause dilation of the plane during sliding (Richards and Cowland, 1982). This angle, $i°$, can be added to the basic friction angle so that the shear strength of joints becomes:

$$\tau = \sigma' \tan(\Phi + i) + c$$

where σ' is the effective normal stress

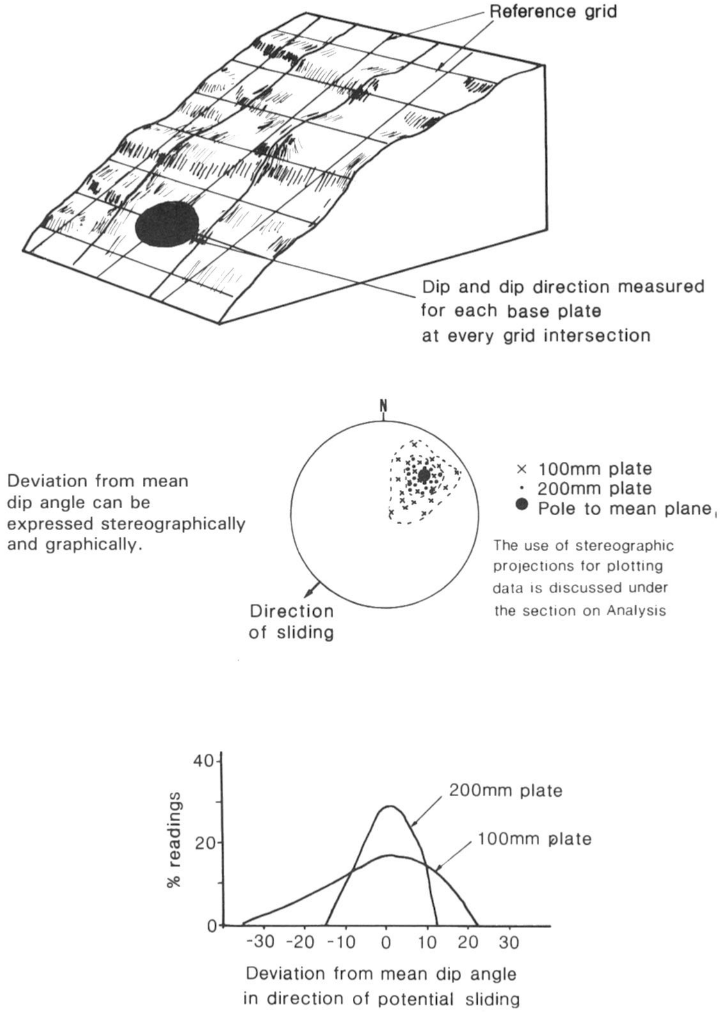


Figure 5.19 Method for quantifying the roughness of a joint

$\Phi°$ is the corrected friction angle from shear tests
$i°$ is the maximum allowable roughness angle
c is cohesion (see discussion below).

For the granite joints discussed earlier, field measurements together with observations of surface damage caused during shear tests suggested that a roughness angle of 15° could be allowed above the corrected friction angle of 40°. The effects of weathering (or infill) could be accommodated by reducing the roughness angle. The procedure for this practical and rational assessment of shear strength by measurement is illustrated in Figure 5.20.

Apparent cohesion due to shearing through of asperities is seldom measured at the low stresses typical of small rock slopes (say up to 50 metres in height) and in fact it is probably

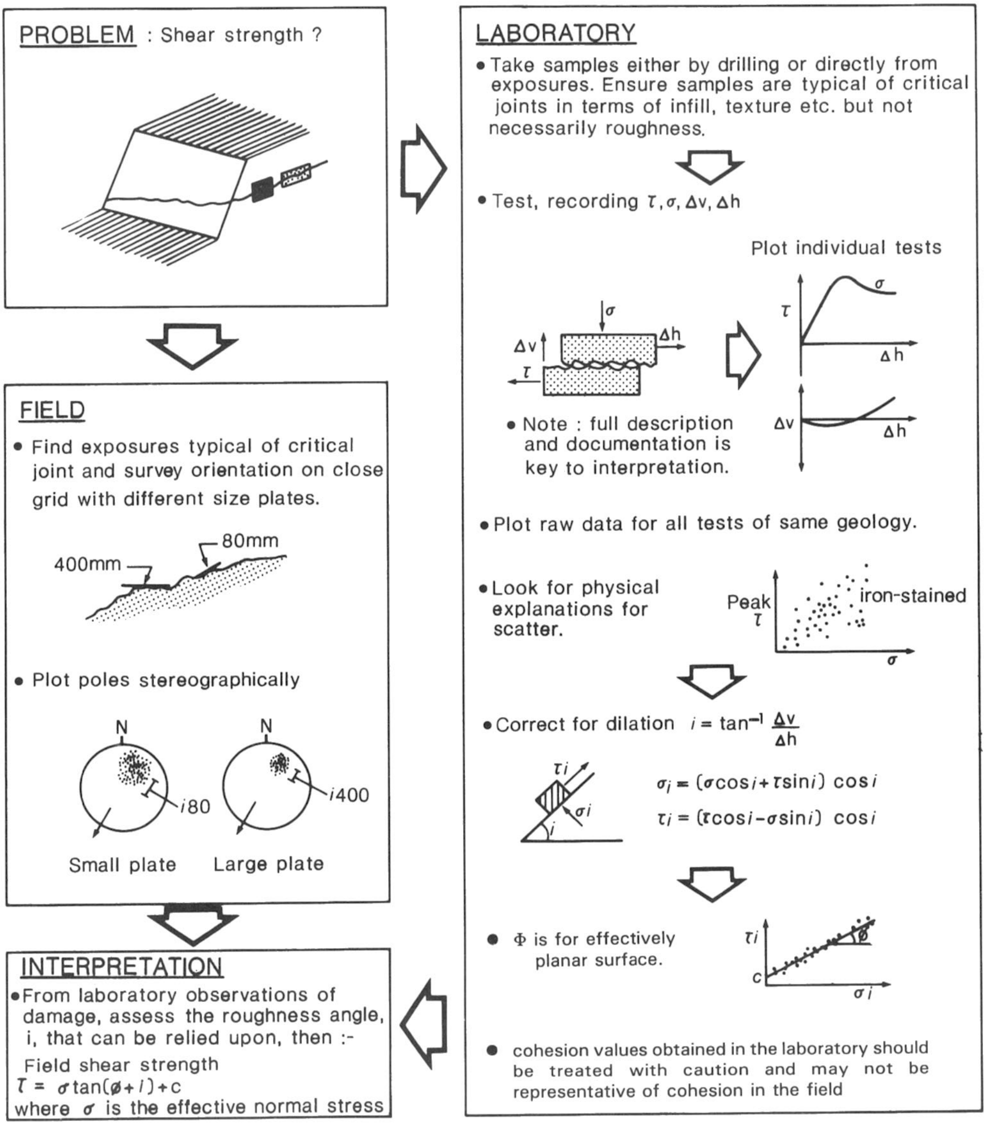


Figure 5.20 Schematic diagram illustrating method for assessing field shear strength

good practice to force test results through the origin in defining the basic friction angle unless there is an obvious cohesional factor. A true cohesion will result from impersistence of joints and several authors (see Jennings, 1970 and Einstein *et al.*, 1983 for example) have addressed the problem theoretically but at present, in the author's opinion, there is no better method available than to use engineering judgement following careful field description. Field values for cohesion might be calculated by back-analysis of failed slopes or by assuming a Factor of Safety of 1.0 for unfailed slopes but often other parameters are so poorly known that this proves impossible (Leroueil and Tavernas, 1981; Hencher *et al.*, 1984).

5.4.2 Shear Strength of Closely Jointed Rock and Soil Masses

Many slope failures occur through closely jointed soil and rock where the joints contribute to a general weakening effect of the mass rather than providing a simple joint-controlled shear surface (Skempton and La Rochelle, 1965; Koo, 1982). Representative shear parameters are very difficult to measure for such modes of failure although several authors have tested large-scale jointed models or large samples which have provided an insight into the factors involved (Hoek, 1984). Hoek and Brown (1980) have reviewed available data and have suggested several empirical strength equations for which the unknowns are rock type, quality of rock mass and compressive strength of the intact material. The authors experience of back-analysis of slope failure through such closely jointed materials is that the equations of Hoek and Brown seem of the right order and in fact there seems little alternative to their use. It is hoped that future published case histories will allow the equations to be tested further. One area for improvement might be to make some allowance for the *degree* of adversity of the jointing and Koo (1982) and Harris (1984) have noted the importance of this factor.

5.5 GROUNDWATER

Groundwater has a considerable influence on slope stability with rainfall being the most common triggering cause of landslides in many parts of the world. Brand *et al.* (1984) for example estimate that more than 90% of landslides in Hong Kong are the direct result of rainstorms (1,500 failures occurring during two rainstorms in 1982).

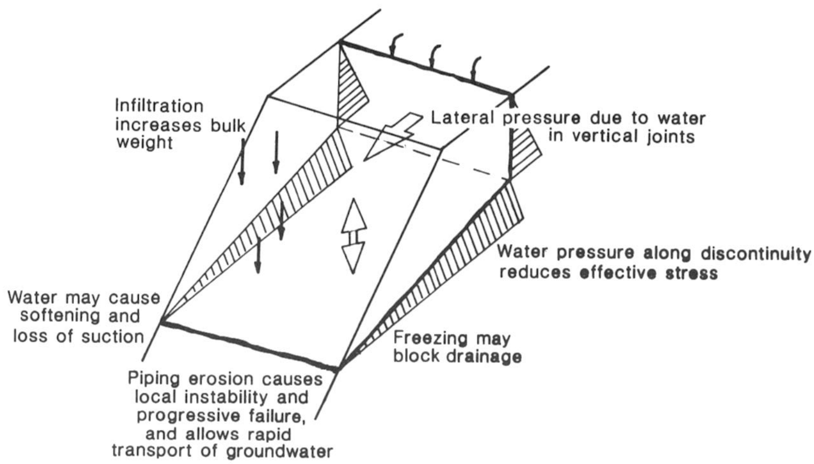


Figure 5.21 Influence of water on slope stability

The main ways in which groundwater affects the stability of slopes in soils and rock masses are illustrated schematically in Figure 5.21 and listed below:

1. Water pressures along the sliding surface reduce the effective normal stress transmitted across the surface thereby limiting the shear resistance developed.

2. Water pressures in vertical joints or fissures apply a force which may initiate sliding.
3. Water infiltration increases the unit weight of the material above the slip plane. This leads to an increased shear load without proportional increase in shear resistance due to (1) above.
4. Internal erosion due to piping.
5. Freezing of groundwater can temporarily restrict drainage and in addition may cause loosening of the mass because of the expansive force produced.
6. Water can cause softening of material as well as loss of suction; both reduce strength.

Drainage measures must be taken into account in the design of remedial or preventive works and the potential corrosion of anchoring systems or reinforcement must also be considered. The complexity of groundwater flow through discontinuous rock or soil will depend upon the geological structure. Certain features will have a controlling influence, notably the distribution of relatively permeable and impermeable lithologies, the degree, distribution and nature of jointing, the presence of faults which may allow rapid ingress

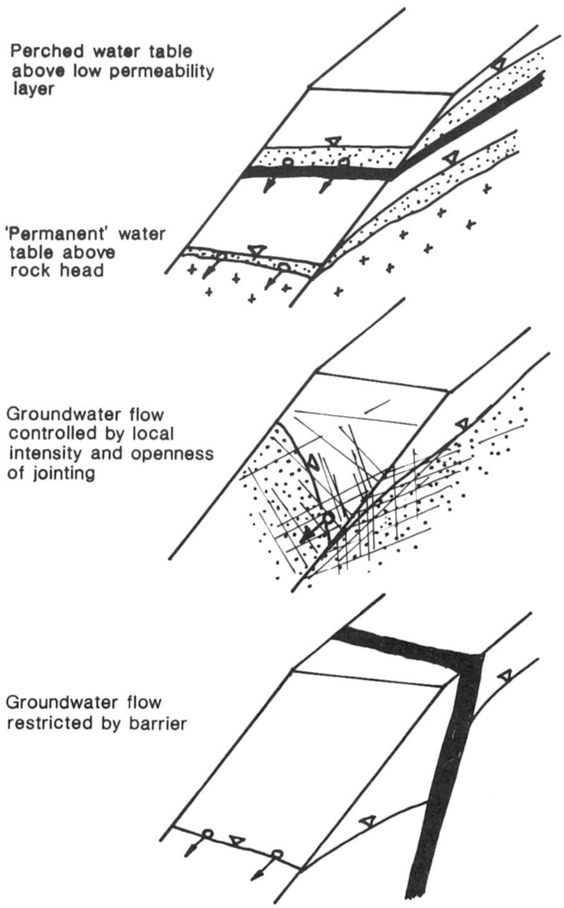


Figure 5.22 Geological controls of groundwater

and distribution of water and the position of rock head in areas of weathered materials. These are illustrated diagrammatically in Figure 5.22.

Detailed information on the hydraulic conductivity or permeability of sections of the mass can be gained by carrying out field tests or by theoretical consideration of the degree and openness of jointing (Hoek and Bray, 1981). The use of dye tracers can be helpful in determining drainage paths. Relevant data gained during site investigation will include levels where water is encountered and depths at which drilling water is lost, particularly if the loss of water can be traced to seepage in an existing face. The nature of the core itself can also be revealing. Joints which carry a lot of water may be particularly weathered and coated with deposits of oxides and clay.

From such investigations it is possible to establish the factors controlling the hydrogeological pattern but to gain an accurate assessment of water pressures to use in design it is generally necessary to install piezometers. The selection of piezometer type and location within the ground should take the following factors into account:

1. The geological and probable hydrogeological structure.
2. The conceivable modes of failure.
3. The expected rate of change of groundwater pressure.

Piezometers are sometimes placed at the base of boreholes with little reference to the site conditions and nature of the problem, the intention being to try to get a general view of the groundwater system. This is fine if the situation is not complex geologically but where, for example, there is a possibility that a perched water table may develop then it is vital that this be investigated specifically. Similarly, where a particularly adverse joint has been identified then piezometers must be installed to measure pressures on that joint alone. Patton (1983) comments that usually too few piezometers are used in slope stability studies to get a true picture and recommends the use of a 'modular' piezometer system which can measure pressures at different levels in a single drillhole. If piezometers are placed at the wrong location then misleading or insufficient data will be obtained. For example, of eleven failures investigated by the author in 1982, six had piezometers either through or close to the failure scar (Hencher *et al.*, 1984) but none of those piezometers provided data representative of conditions on the failure scar at failure (as proved by field observation or back-analysis). This was in some cases due to poor choice in location of piezometers relative to the critical geological structures, and in others due to an inadequate monitoring system.

The sophistication of the monitoring system must match the expected fluctuations in water pressure (see Chapter 3). If these are slow changing, then standpipes which are dipped on an occasional basis may be sufficient. A simple device comprising a string of plastic containers (buckets) suspended in the pipe can indicate maximum levels which occur between manual readings and is described by Brand *et al.* (1983). Conditions where transient response is expected to be important or where there is a high degree of complexity may demand that an automatic monitoring system be set up (Pope *et al.*, 1982). Cowland and Richards (1985) describe an automatic system for measuring water pressures on specific rock joints and show how such a system can reveal behaviour that would have been very difficult to predict. They demonstrate the transient nature of water pressure through a slope following intense rainfall with pulses of groundwater pressure migrating rapidly along the surfaces of large joints. This produces less severe conditions than might be suspected using results from a less sophisticated system.

Once a reasonable understanding of the groundwater pattern and the factors contributing to changes in groundwater pressure are established, modelling techniques may be used to predict the effect of changes such as increased infiltration, or excavation. Commonly used methods include flow nets (Wittke and Louis, 1966), electrical analogue methods (Hoek and Bray, 1981) and, more rarely, computer techniques (Leach and Herbert, 1982). The incorporation of 'design' water pressures in analysis is discussed in the next section.

5.6 ANALYSIS OF SLOPE STABILITY IN JOINTED ROCKS AND SOILS

5.6.1 Introduction

This section deals primarily with the analysis of slopes through which well-defined adverse discontinuities have been recognized. Two other special cases will be discussed briefly:

1. Closely jointed rock masses where soil mechanics methods of analysis are applicable.
2. Natural slopes where only very limited geotechnical information is available.

The degree of confidence in the results from any analysis will depend upon the quantity and quality of available data which will generally reflect: (a) the seriousness of the problem, i.e. risk versus consequence; (b) the resources available; (c) the difficulties of investigation.

It is sometimes not appreciated how difficult it is to obtain adequate information for the analysis of jointed rock masses using drillholes. The results of analysis based on such an investigation must, therefore, be treated with caution and in the case of the design for a new cutting, assumptions must be checked during construction.

The importance of any uncertainties in the input data can be quantified by carrying out a series of analyses in which each parameter is varied in turn within its probable range of values. Such a study is called a sensitivity analysis and is particularly useful when trying to assess the cost effectiveness of different options for preventive or remedial works (e.g. drainage or anchoring). Several authors have taken this type of analysis further and have expressed the likelihood of failure as a probability based on the statistical variation of the input parameters (McMahon, 1975; Piteau and Martin, 1977; Priest and Brown, 1983).

5.6.2 Discontinuity-controlled Failures

The analysis of discontinuity-controlled failures generally involves two interrelated stages:

1. Firstly, the available joint data must be sorted to identify those along which failures might occur. This may sometimes be done for an existing slope by eye providing the exposure is good.
2. Secondly, each joint-defined block must be analysed to determine whether the available shear resistance exceeds the forces tending to cause failure.

A combination of various techniques is used to carry out the analysis:

1. Stereographic projection is a geometrical construction which allows individual joints and other planes to be represented and their angular relationships to be measured; it

is used primarily for sorting data into a manageable form and for preliminary assessment of the potential for a block of rock to move. Detailed analysis can also be carried out using stereographic projection but that requires all data to be expressed as angles which involves some difficulty.

2. Graphical methods in which force vectors are drawn as a polygon are used to assess the stability of particular blocks. The geometry of each block must be pre-defined perhaps following a stereographic appraisal of all joint data.
3. Numerical analytical methods can be applied to simple wedges or blocks of rock but require the geometry to be pre-determined as for graphical methods.
4. Other methods include physical modelling using base friction or block models. Finite element and boundary element computer techniques are useful for calculating and illustrating stress distributions but have also been used to produce visual (and numerical) models of slopes failing.

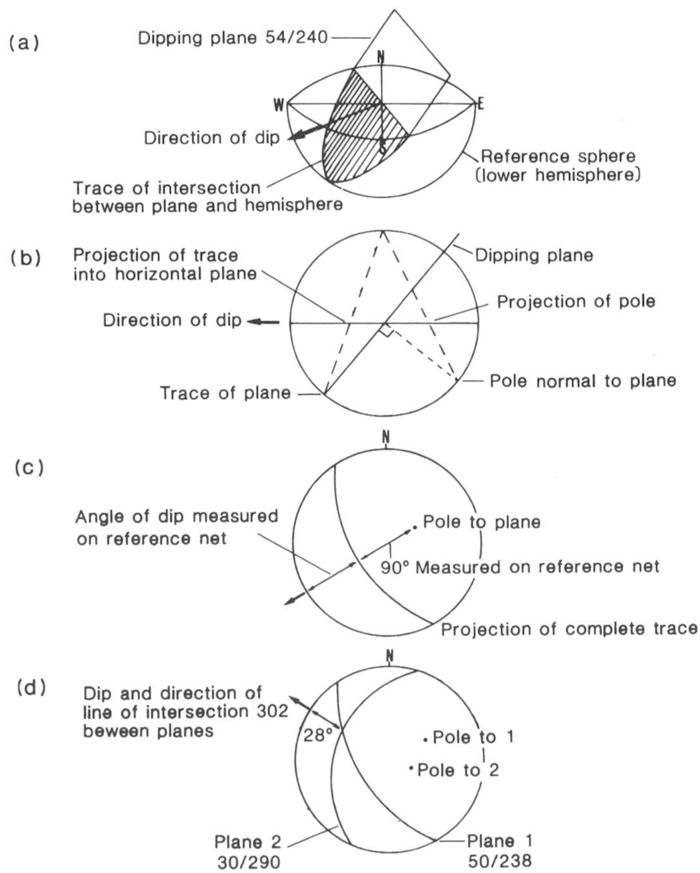


Figure 5.23 Representation of dipping planes as traces and poles

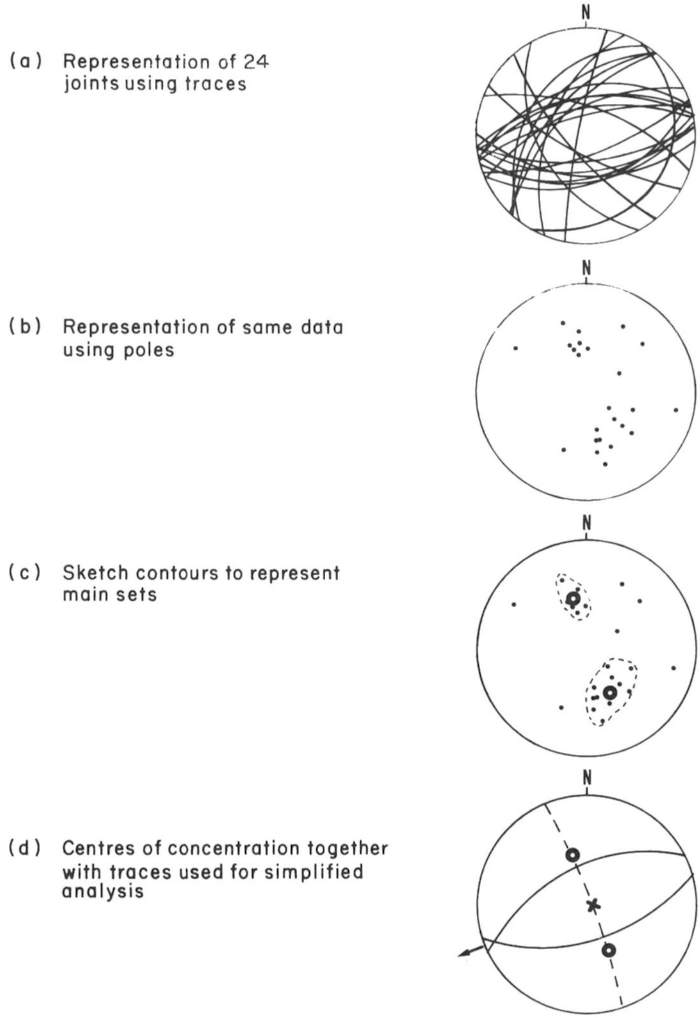


Figure 5.24 Principles for sorting large amounts of data

5.6.3 Stereographic Projection

Stereographic projection is a powerful tool for representing the geometrical relationships of inclined planes and is familiar to geologists through its use for evaluating problems in structural geology and crystallography. It has been adopted and developed by workers in rock mechanics to the stage where its use can provide direct analytical results for quite complex slope stability problems albeit with some difficulty in expressing input data with respect to cohesion, water pressures, and reinforcement (see, for example, Attewell and Farmer, 1976; Priest, 1985). Recent advances in the use of the technique for analysing the kinematics of moveable blocks are discussed by Goodman (1983).

The main use of stereographic projection in the stability analysis of jointed masses is for sorting otherwise unmanageable quantities of joint data (Pentz, 1971) and for the

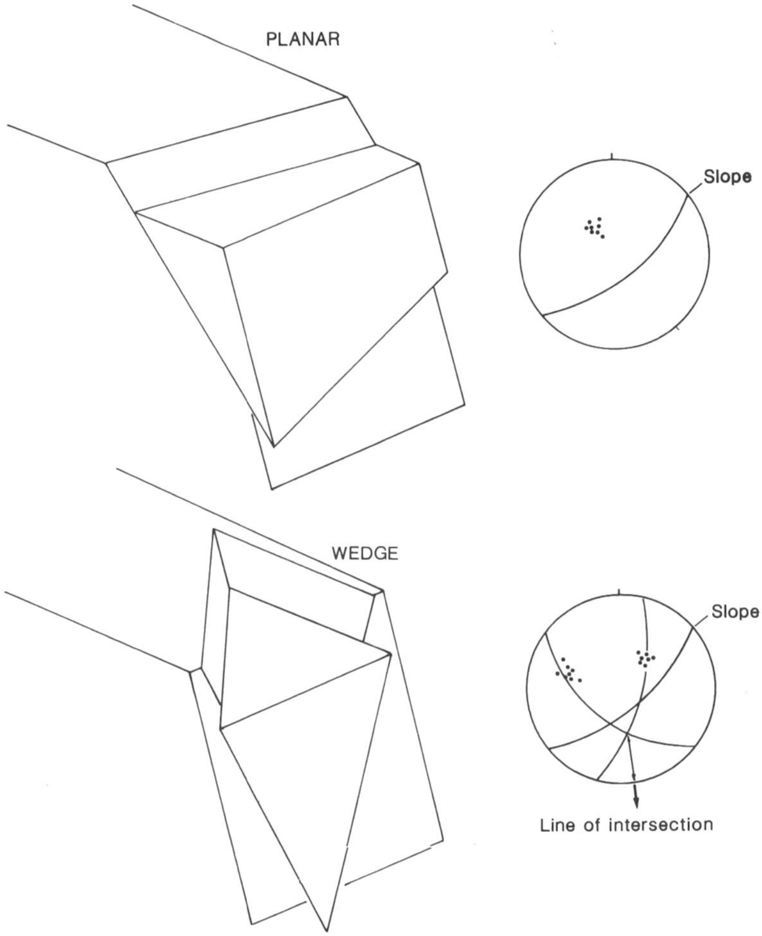
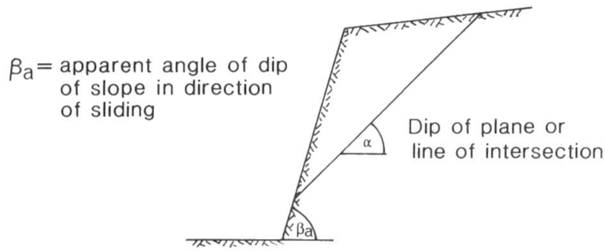


Figure 5.25 Typical modes of failure

preliminary analysis of those data with reference to other aspects of the problem that can be readily expressed geometrically (slope angle, friction angle). The stereographic projection allows identification of the main structural features and particular discontinuity combinations that require more detailed analysis.

The principles and technique for preparing a reference stereographic 'net', for plotting data and for analysis are covered by several excellent publications, notably Phillips (1971), Hoek and Bray (1981), Richards *et al.* (1978), and Priest (1985) the latter three with particular reference to slope stability, and the reader is referred to those references for a clear explanation of how to carry out this work. The principle, as illustrated in Figure 5.23(a), is to imagine all planes defined by their average dip angles and dip directions bisecting a reference sphere which is fixed in space. The planes intersect either the lower hemisphere (by convention) or more rarely the upper hemisphere. Either construction allows the same analysis. The trace marking the intersection of each plane with the hemisphere



For sliding: $\beta_a > \alpha > \Phi$

where Φ is a representative friction angle

Note: Effects of water pressure, cohesion, impersistence and wedging not taken into account.

Figure 5.26 Conditions for sliding

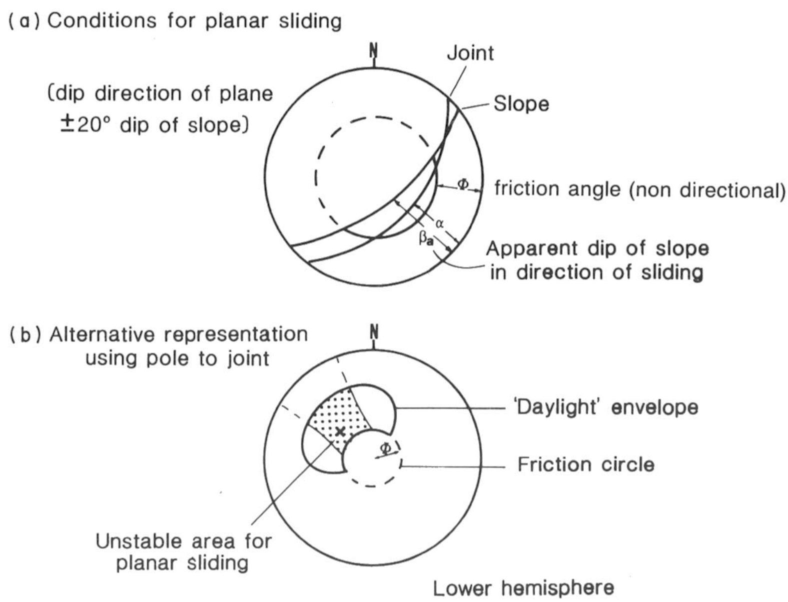


Figure 5.27 Stereographic test for planar sliding

is projected on to the horizontal section through the reference sphere as shown in Figures 5.23(b) and (c). As discussed later, it is often convenient to represent planes, not by their traces, but by a single pole normal to the plane (Figures 5.23(b) and (c)). Either construction may be used to represent the same unique information for a single plane as plotted in Figure 5.23(c). Figures 5.24(a) and (b) are plots of the same joint data presented as traces and as poles respectively. The advantage of using traces is that the points of intersection

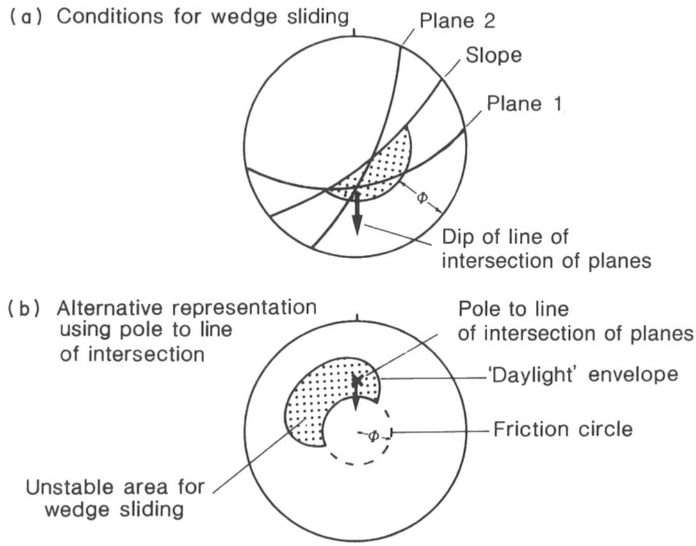


Figure 5.28 Stereographic test for wedge sliding

Table 5.4 Use of Stereographic Projections for Slope Analysis

USES FOR ROCK SLOPE STABILITY

- measuring angular relationships
- identifying sets of joints
- demonstrating relationships between discontinuities and slope face geometry thereby showing which individual discontinuities or wedges daylight and therefore might cause failure
- comparing inclinations of potential failure planes and wedge intersections with trial friction angles
- to demonstrate the degree of roughness of a single discontinuity
- can be used directly for analysis providing shear strength parameters c and Φ and water forces U and V are known (see Attewell and Farmer, 1976)

LIMITATIONS

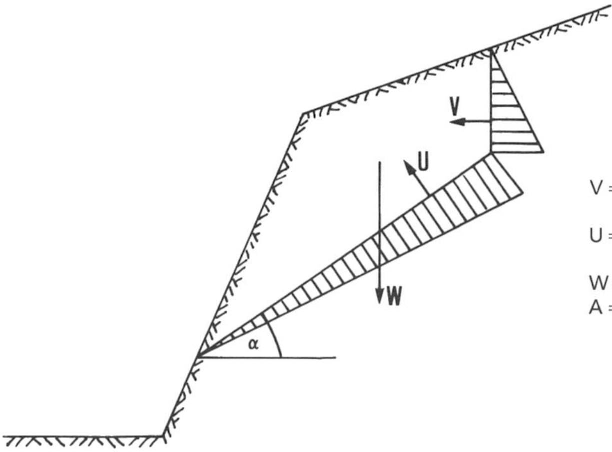
- spatial relationships not shown (e.g. two joints apparently forming wedge on the stereoplot may be separated in the slope)
- nature of individual joints not represented
- discontinuities plotted as single poles
- cohesional component of strength not readily taken into account
- effects of water not readily taken into account (see discussion by Sekula, 1982)

COMMON MISUSES

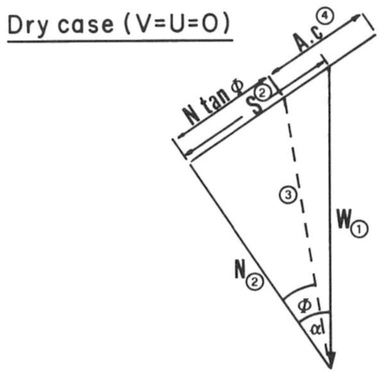
- used directly for analysis in an oversimplistic manner rather than as tool to aid understanding
- contouring to show centres for joint sets can overshadow more important, adverse joints
- contouring of too few joints

HINTS

- only contour data where absolutely necessary or to get a better understanding of the problem; do not contour a limited amount of data
- always check the nature of any 'adverse' joints removed by contouring
- once adverse joints have been identified, then analyse potential failures individually, using limit equilibrium methods to take full account of strength, water pressures, and other characteristics of specific joints of interest
- critically assess the representativeness of your data



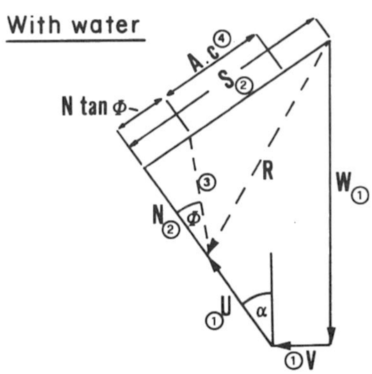
V = Horizontal force due to water in vertical joints
U = Uplift force due to water pressure along joint
W = Weight of block
A = Area of sliding surface (cohesion force = Ac)



1. Construct weight vector
2. Construct shear and normal components of weight
3. Draw line offset Φ degrees from N to construct friction force N tan Φ
4. Draw on cohesive force

$$\text{F of S} = \frac{N \tan \Phi + Ac}{S}$$

(in case illustrated > 1)



1. Draw vectors for weight and water forces as shown
2. Construct shear and normal components of resultant R
3. As above
4. As above

$$\text{F of S} = \frac{N \tan \Phi + Ac}{S}$$

(in case illustrated < 1)

Additional forces such as earthquake and anchor forces can be drawn at Stage 1 to produce a different resultant R

Figure 5.29 Force diagram for assessing stability are all clear but the overall diagram is cluttered, the data are more difficult to plot and it is less easy to identify any 'sets' than on the plot of poles. Poles can be split into groups by contouring, the centres of concentrations being marked by single poles (Figure 5.24(c)). Techniques are discussed by Hoek and Bray (1981). Centres of concentration are taken as representative of 'sets' and can then be used for further analysis. It should be noted, however, that there is a danger that important individual or rare discontinuities may be overlooked as a result of contouring and this is discussed in detail by Hencher (1985).

Planes and wedges which may result in failure are typified by particular patterns on the stereogram (Figure 5.25) and are analysed by comparing their angles of dip with their angles of sliding and determining whether or not they will daylight (appear in the slope face). Figures 5.26 to 5.28 illustrate the procedures. It should be noted that although two planes appear to form an adverse wedge on the stereogram they may not do so in the field due to geographical separation. Wedges formed from sets of joints are much more likely to occur naturally. It is always necessary to check the results of the stereographic analysis in the field and a useful though time-consuming technique to aid in this check is to mark each joint with a reference number during the initial survey and to use those numbers during analysis.

Toppling failures are associated with joints dipping steeply back into the slope and such joints can be identified using stereographic techniques. Goodman and Bray (1976) discuss the criteria for such failures. Some of the most important uses and potential misuses of stereographic techniques for the analysis of slope stability in jointed rocks and soils are listed in Table 5.4, the misuses mostly stemming from the exclusive use of contoured data for interpretation.

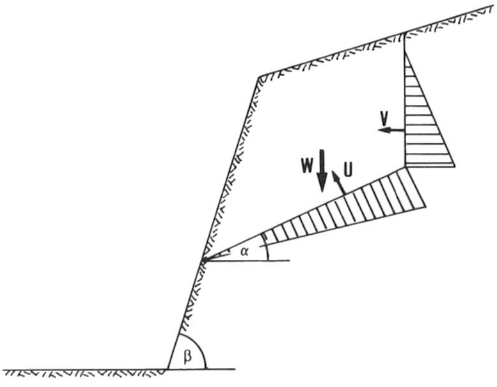
5.6.4 Graphical Methods

Provided the geometry of the slope and failure surface are well defined then stability can be assessed graphically by constructing a force diagram as illustrated in Figure 5.29. Hoek *et al.* (1973) use the method to analyse the stability of a complex wedge.

5.6.5 Numerical Methods

The simplest slope failure type to analyse is where sliding occurs along a single plane. The factor of safety is calculated by resolving all forces along the failure plane and dividing the forces resisting sliding by the forces inducing sliding, as illustrated in Figure 5.30. Stimpson (1979) presents simplified equations for calculating factors of safety for various slope conditions. It has been noted earlier that cohesion is often of great significance in such calculations yet is very difficult to determine.

The analysis of wedges of rock such as that in Figure 5.31 sliding on two surfaces is complex despite the apparent simplicity of the situation. The reader faced with the need to analyse such a wedge is referred to Hoek and Bray (1981) who explain the analytical method very carefully and provide examples of calculation. Most such calculations are carried out using either a programmed calculator or a computer. Warburton (1983) has developed a program which can produce a three-dimensional block diagram of a slope and its critical discontinuities and can analyse the stability of individual wedges.



Shear resistance, $S = cA + (W\cos\alpha - U - V\sin\alpha)\tan\Phi$

Shear Load $= W\sin\alpha + V\cos\alpha$

where
$\alpha =$ angle of dip of plane
$W =$ weight of block
$U =$ uplift force due to water
$V =$ water force in vertical discontinuity
$\Phi =$ angle of friction (including roughness)
$c =$ cohesion
$A =$ area of shear plane

$$\text{Factor of safety} = \frac{\text{Shear resistance}}{\text{Shear load}}$$

Figure 5.30 Analysis of sliding rock

Figure 5.31 Wedge due to intersection of two joint sets

Conditions for toppling failure (Figure 5.32) and its analysis are discussed in detail by Evans (1981) and Goodman and Bray (1976) and design charts have been presented by Zanbak (1983).

5.6.6 Other Methods

Physical models are useful both to illustrate geological structural relationships and actually to model loading conditions. One of the most illustrative methods is the base friction device

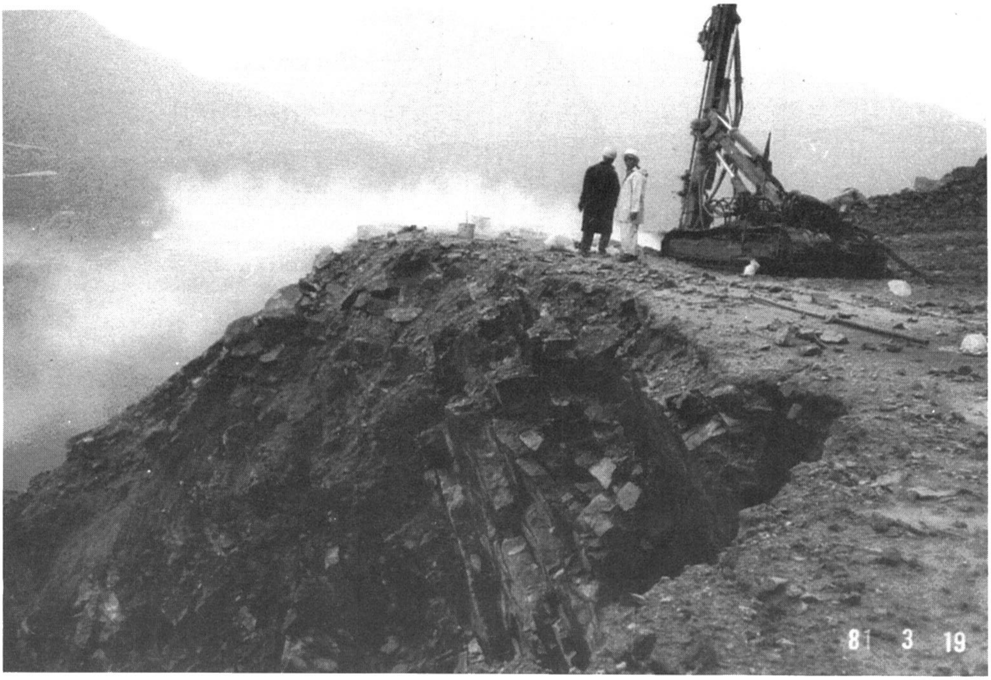


Figure 5.32 Potential toppling failure

which involves modelling slopes on their sides on a moving belt of rough sandpaper. The friction between the paper and the model provides an equivalent to gravitational loading and allows the importance of key blocks and joint orientations to be assessed (Bray and Goodman, 1981). Cundall (1971) developed a two-dimensional computer model which can similarly demonstrate the general mechanism of failure. Finite and boundary element techniques have not proved as useful for slope stability analysis as they have for underground excavations, particularly for assessing stress conditions. For rock slope analysis, stress conditions, though in reality quite complex, are generally treated in a very simplified manner.

5.6.7 Natural Slopes

Aerial photographic interpretation backed up by field checks is often the main method for investigating the stability of large hillslopes. The data obtained by such methods is unlikely to be of sufficient quality or quantity to allow detailed analysis. As a result several authors have developed broadly based classifications for assessing the risk of landslides based on such general parameters as degree of jointing, average rock strength, and slope angle (Selby, 1980 and Chapter 15 in this volume; Vecchia, 1977). Styles *et al.* (1984) have assessed the reliability of geotechnical risk maps based on such criteria by comparison with the results from 1,400 limiting equilibrium analyses along 32 km of cross-section and have shown reasonable agreement.

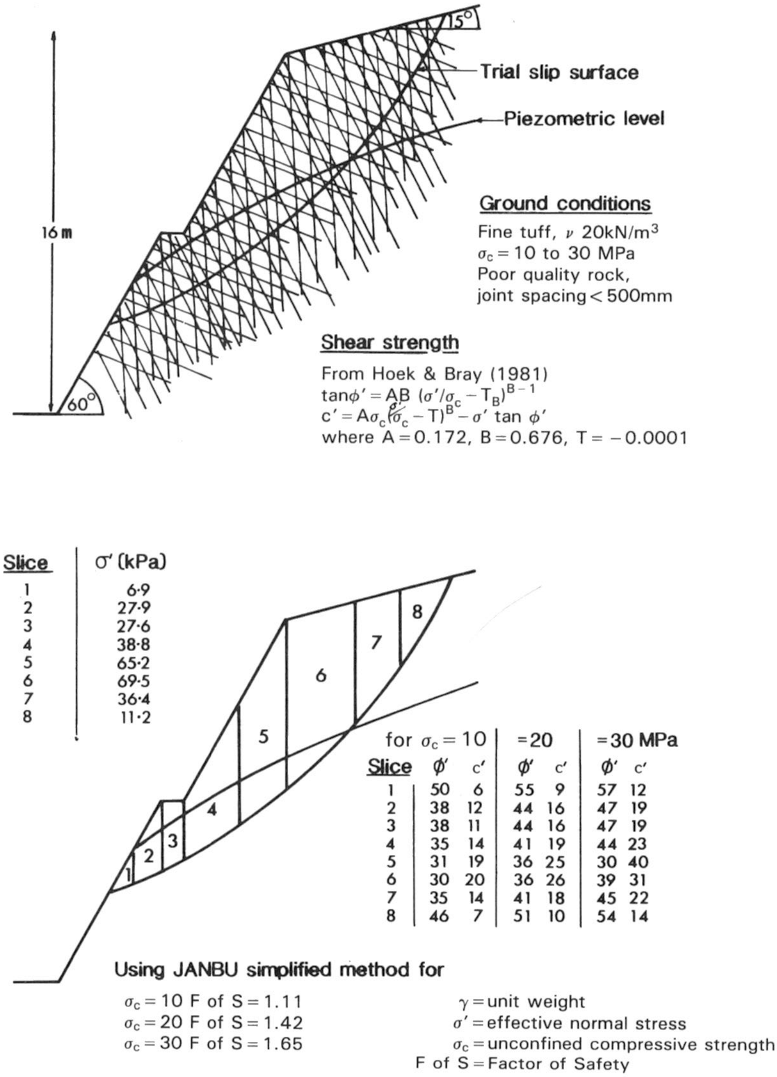


Figure 5.33 Example of analysis of closely jointed rock slope using Hoek–Brown empirical strength criteria

5.6.8 Closely Jointed Masses

Where the ground is so closely jointed that failure may occur along a generalized surface through the mass then soil mechanics methods of analysis such as those discussed in chapter 2 are the most applicable. Hoek and Bray (1981) give useful design charts against circular failure for different groundwater conditions and Hoek and Brown's (1980) empirical strength criteria can provide an estimate of the shear strength parameters to use in the analysis. Surfaces other than circular arcs can be dealt with most easily using the methods of Janbu (1973) and Sarma (1979). An example is given in Figure 5.33. Priest and

Brown (1983) give a well-documented example of the analysis of generalized failure surfaces using Janbus's method and Hoek and Brown's strength criterion. They vary the input data statistically within pre-defined limits and express the results in a probabilistic manner.

5.7 CONCLUSIONS

The presence of discontinuities in rock and soil masses considerably affects their engineering properties and generally imposes markedly anisotropic strength. Slope stability analysis in such materials is dominated by the need to identify, measure, and analyse potentially failing blocks delineated by discontinuities. The importance of geological knowledge to all site investigations lies in the geologist's ability to recognize and establish the geological structure on the basis of limited data. This allows him to predict and extrapolate the nature of materials away from the point of observation. A table of discontinuity types has been presented to aid in their recognition in the field and in assessing their probable nature.

The shear strength of persistent rock joints can be measured providing representative samples of joint surfaces are available. Impersistence and hence true cohesion of rock joints is, however, difficult to assess and further research is needed into how this property can be estimated. Groundwater pressures in jointed masses are often complex and require careful instrumentation to allow them to be measured with an acceptable degree of accuracy. This is probably one of the weakest aspects of most investigations and care should be taken to carry out preliminary analysis prior to instrumentation to ensure that pertinent information is collected. Methods of analysis for jointed rock and soil differ from those for 'homogeneous' and 'isotropic' soil in that the failure shape must be identified physically in the field rather than searched for mathematically. Stereographic analysis is particularly useful for identifying potential failure blocks from a mass of data on discontinuities. Numerical methods are now available to analyse quite complex wedge shapes allowing components of strength, weight, external loading, water pressure, and preventive or remedial works to be taken into account.

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