Máster Universitario
en Túneles
y Obras Subterráneas

ÁREA: A
MÓDULO: MECÁNICA DE ROCAS

TEXTO COMPLEMENTARIO A CONFERENCIA SOBRE JUNTAS

Ponente: Dra. Mercedes Ferrer
IGME
Día: 08/02/07
Hora: 16:00 a 18:00

INTERNATIONAL SOCIETY FOR ROCK MECHANICS
COMMISSION ON
STANDARDIZATION OF LABORATORY AND FIELD TESTS

Suggested Methods for the Quantitative Description of Discontinuities in Rock Masses

Prepared by

on (Coordinator), (*Norway*); W. E. Bamford, C. M. Barton and B. MacMahon (*Australia*); M. A. Kanji (*Brazil*); K. Babcock, J. M.). Cruden, J. A. Franklin, G. Herget, G. Macleod and D. R. Piteau (*Canada*); D. Cawsey, W. Dearman, M. DeFreitas, J. M. Edmond, ookes, and T. R. Harper (*U.K.*); K. Saari and M. Tammirinne (*Finland*); K. John (*Germany*); G. Manfredini (*Italy*); A. Bello (*Mexico*); ;ers (*Netherlands*); H. Barkey, R. Lien, T. Løken, F. Løset, and A. Palmstrøm (*Norway*); N. F. Grossman and R. Oliveira (*Portugal*); :Beer and J. E. Jennings (*South Africa*); M. Bergman and I. Hansagi (*Sweden*); D. C. Banks, T. L. Brekke, D. U. Deere, H. H. Einstein, liller, H. J. Pincus, D. Ross-Brown and J. Warriner (*U.S.A.*); J. Obradovic (*Yugoslavia*).

INTRODUCTION

The majority of rock masses, in particular those within a few hundred metres from the surface, behave as discontinua, with the discontinuities largely determining the mechanical behaviour. It is therefore essential that both the structure of a rock mass and the nature of its discontinuities are carefully described in addition to the lithological description of the rock type. Those parameters that can be used in some type of stability analysis should be quantified whenever possible.

For example, in the case of rock slope stability certain quantitative descriptions can be used directly in a preliminary limit equilibrium analysis. The orientation, location, persistence, joint water pressure and shear strength of critical discontinuities will be direct data for use in analysis. For purposes of preliminary investigation the last two parameters can probably be estimated with acceptable accuracy from a careful description of the nature of the discontinuities. Features such as roughness, wall strength, degree of weathering, type of infilling material, and signs of water seepage will therefore be important indirect data for this engineering problem.

For the case of tunnel stability and estimation of support requirements, all the descriptions will tend to be indirect data since a direct analysis of stability has yet to be developed. However, a careful description of the structure of a rock mass and the nature of its discontinuities can be of inestimable value for extrapolating experience of support performance to new rock mass environments. Descriptions should be sufficiently detailed that they can form the basis for a functional classification of the rock mass.

In time, as descriptions of rock masses and discontinuities become more complete and unified, it may be possible to design engineering structures in rock with a minimum of expensive *in situ* testing. In any case careful field description will enhance the value of *in situ* tests that are performed, since the interpretation and extrapolation of results will be made more reliable.

GLOSSARY

A selection of terms commonly used in these "Recommended Methods" are defined here. Contributors to the Working Party were divided in their recommendations for the best general term to represent all "breaks" in rock masses. However, a clear majority preferred *discontinuity* rather than *fracture*, as the collective term for all joints, bedding planes, contacts and faults.

Joint

A break of geological origin in the continuity of a body of rock along which there has been no visible displacement. A group of parallel joints is called a set and joint sets intersect to form a joint system. Joints can be open, filled or healed. Joints frequently form parallel to bedding planes, foliation and cleavage and may be termed bedding joints, foliation joints and cleavage joints accordingly.

Fault

A fracture or fracture zone along which there has been recognisable displacement, from a few centimetres to a few kilometres in scale. The walls are often striated and polished (slickensided) resulting from the shear displacement. Frequently rock on both sides of a fault is shattered and altered or weathered, resulting in fillings such as breccia and gouge. Fault widths may vary from millimetres to hundreds of metres.

Discontinuity

The general term for any mechanical discontinuity in a rock mass having zero or low tensile strength. It is the collective term for most types of joints, weak bedding planes, weak schistocity planes, weakness zones and faults. The ten parameters selected to describe discontinuities and rock masses are defined below:

1. *Orientation*—Attitude of discontinuity in space. Described by the *dip direction* (azimuth) and *dip* of the line of steepest declination in the plane of the discontinuity. Example: *dip direction/dip* (015°/35°).

2. *Spacing*—Perpendicular distance between adjacent discontinuities. Normally refers to the mean or modal spacing of a set of joints.

3. *Persistence*—Discontinuity trace length as observed in an exposure. May give a crude measure of the areal extent or penetration length of a discontinuity. Termination in solid rock or against other discontinuities reduces the persistence.

4. *Roughness*—Inherent surface roughness and waviness relative to the mean plane of a discontinuity. Both roughness and waviness contribute to the shear strength. Large scale waviness may also alter the dip locally.

5. *Wall Strength*—Equivalent compression strength of the adjacent rock walls of a discontinuity. May be lower than rock block strength due to weathering or alteration of the walls. An important component of shear strength if rock walls are in contact.

6. *Aperture*—Perpendicular distance between adjacent rock walls of a discontinuity, in which the intervening pace is air or water filled.

7. *Filling*—Material that separates the adjacent rock walls of a discontinuity and that is *usually* weaker than he parent rock. Typical filling materials are sand, silt, clay, breccia, gouge, mylonite. Also includes thin mineral coatings and healed discontinuities, e.g. quartz and calcite veins.

8. *Seepage*—Water flow and free moisture visible in individual discontinuities or in the rock mass as a whole.

9. *Number of Sets*—The number of joint sets comprising the intersecting joint system. The rock mass may be further divided by individual discontinuities.

10. *Block Size*—Rock block dimensions resulting from the mutual orientation of intersecting joint sets, and resulting from the spacing of the individual sets. Individual discontinuities may further influence the block size and shape.

SAMPLING PHILOSOPHY

Geological engineering investigations are generally carried out in several stages, to provide information of appropriate detail to the current state of the project:

(i) feasibility
(ii) detailed planning
(iii) construction/operation.

The degree of detail required for each stage will vary considerably from project to project.

There are two basic levels at which a rock mass survey may be carried out depending upon the amount of detail that is required. In a *subjective* (biased) survey only those discontinuities which appear to be important are described. In an *objective* (random) survey all discontinuities intersecting a fixed line or area of rock exposure are described.

A prerequisite for both types of survey is the study of any available geological maps followed by a geological reconnaisance of rock types, major geological structures, faults, dykes and lithological contacts. A study of air photographs will often be invaluable for planning this reconnaisance. At this preliminary stage efforts should be made to recognise *domains* where systematic features such as joints possess similar orientation or spacing. The fabric of the rock mass is statistically homogeneous in a domain.

The objective approach to sampling suffers from the major disadvantage that it is time consuming. Some form of automatic data processing may be required to analyse all the data. However, if structural domains cannot readily be delineated there may be no alternative. The subjective approach is best applied where structural domains are clearly recognised. This will save time and effort and will usually reveal all the discontinuity systems found in any subsequent line or area survey.

Rock masses and their component discontinuities can be described by the principal methods:

(a) outcrop description
(b) drillcore and drillhole description
(c) terrestrial photogrammetry.

1. ORIENTATION

(A) *Compass and Clinometer Method*

Scope

(*a*) The orientation of a discontinuity in space is described by the *dip* of the line of steepest declination measured from horizontal, and by the *dip direction* measured clockwise from true north. Example: *dip direction/dip* (025°/45°).

(*b*) The orientation of discontinuities relative to an engineering structure largely controls the possibility of unstable conditions or excessive deformations developing. The importance of orientation increases when other conditions for deformation are present, such as low shear strength and a sufficient number of discontinuities or joint sets for slip to occur.

(*c*) The mutual orientation of discontinuities will determine the shape of the individual blocks, beds or mosaics comprising the rock mass.

Equipment

(*a*) Compass and clinometer. Compasses which need to be levelled by means of a spherical bubble, before taking a dip reading with the lid parallel to the dip, have the advantage that the maximum declination (dip) is measured directly. Other types of clinometer need to be moved across the discontinuity wall until the maximum value is registered.

(*b*) When the rock is strongly magnetic a clino-rule and 50 m tape, or a direct reading azimuth protractor can be used.

(*c*) When estimating the dip of inaccessible joints i

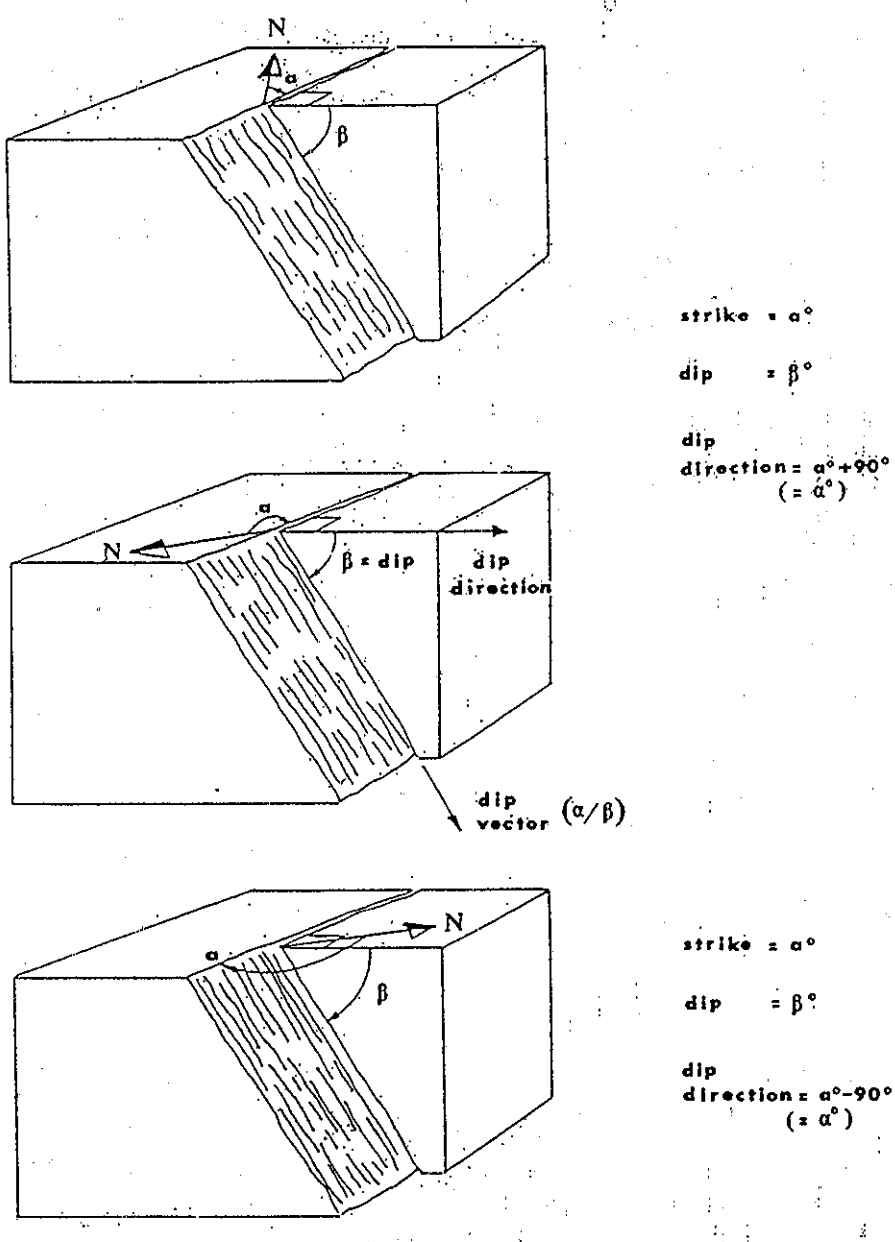


Fig. 1. Diagrams indicating the strike, dip and dip direction of three differently orientated planes.

is convenient to use a clinometer with an inclinable sighting device, and incorporating a reflected image of a horizontal bubble.

Procedure

(*a*) The maximum declination (dip) of the mean plane of the discontinuity is measured with the clinometer and should be expressed in degrees as a two digit number, e.g. 05° or 55° (00°–90°).

(*b*) The azimuth of the dip (dip direction) is measured in degrees counted clockwise from true north and expressed as a three digit number, e.g. 010° or 105° (000°–360°).

(*c*) The dip direction and dip should be recorded in that order, with the three digit and two digit numbers separated by a line, e.g. 010°/05°. The pair of numbers represents the dip vector. See Fig. 1.

Notes

(*a*) Magnetic deflections caused by iron pipes or rails, or anomalies due to ore bodies will sometimes cause compass readings to be unreliable. In such cases a 50 m long tape should be stretched parallel to the rock face or tunnel wall and orientated by means of plans and ground surveys. Dip direction can then be measured relative to this tape using a clino-rule, placing one leg parallel to the tape. The data should be corrected to true north before analysis of the field measurements is undertaken. Alternatively a direct reading azimuth protractor can be employed in place of the clino-rule and tape.

(*b*) The dip of discontinuities considered critical for stability should be measured using a down-dip base length *exceeding* the wave length of surface undulations. The local inclination of non-planar features rela-

tive to *mean dip* will be an important component in the shear strength of the surface in question. The estimated direction of potential movement may not coincide with the down-dip direction.

(*c*) It is desirable to measure a sufficient number of orientations to define the various joint sets of given domains. Opinions concerning the required number vary from about 80 to 300. A reasonable compromise would seem to be 150. It is clear that the number to be recommended will vary with the area to be mapped, with the randomness of the orientations, and with the detail required in subsequent analyses. If orientations are consistent, careful sampling will reduce the amount of orientation data considerably.

(*d*) Several countries on the European continent have for many years utilized survey equipment and compasses with horizontal scales divided into 400 parts (e.g. 0–400ᵍ). This has obvious advantages when measuring to decimal point accuracy.

The vertical circle of many clinometers is also expressed in quadrants of 100ᵍ instead of 90. The particular system utilized should be clearly stated when orientation data is reported. For the purpose of soil and rock mechanics stability analyses it is most convenient to have dip measurements measured in, or converted to, the older 0–90° system. (Conversion factor: 9/10).

(*e*) The accuracy of compass and clinometer orientation measurements will depend on several factors of which the following are probably most important; accessibility of the plane of interest, areal extent of the exposed plane, degree of planarity and smoothness, occasional magnetic anomalies, human errors. Human errors can be reduced by using a clinometer to locate the direction of maximum dip, before taking the compass reading. It is probably sufficient for rock mechanics purposes to read *dip direction* to the nearest 5°, and *dip* to the nearest even number of degrees. However, if poles are to be plotted it may in the end be more convenient to read to the nearest degree to reduce the occurrence of coincidental plotted points.

(*f*) The mean orientation of major discontinuities can be obtained by the *three point method*. The coordinates of three points lying in the plane of the discontinuity are all that is required. In the case of surface outcrops the coordinates may be determined by accurate location on a contoured relief map. The orientation of major features may also be estimated from three boreholes that intersect the plane. However less persistent features may not be intersected by all the holes.

(*g*) The orientation of minor discontinuities can be estimated from a single borehole, provided that the core can be orientated or that the borehole walls can be viewed. Core can sometimes be orientated using structural features such as bedding or foliation if these natural markers have consistent orientation. Several artificial orientation devices operated from the core barrel are also available, e.g. the Craelius core orientator. Alternatively, the orientation of minor discontinuities can be estimated by down-the-hole viewing techniques such as borehole television cameras, photographic cameras and borehole periscopes. Besides orientation these methods also provide invaluable information concerning spacing, the thickness of the discontinuity fillings and the level of seepage paths. (See 11. Drill Core for details).

(*h*) A special core recovery method known as the integral sampling method [1] is recommended for obtaining orientation data in heavily fractured rock masses. The method essentially consists of recovering a core sample which has previously been reinforced with a grouted bar whose azimuth is known from positioning rods. The reinforced bar is coaxially overcored with a larger diameter coring crown.

Presentation of results

(*a*) *Strike and dip symbols*. The simplest methods of data presentation are the strike and dip symbols drawn in the correct location on the geological map of the area. For example:

45° represents a discontinuity with a dip of 45° and strike as shown by the orientation of the line. The dip direction is indicated by the down-dip symbol.

represents a horizontal discontinuity.

represents a vertical discontinuity with a strike as shown by the orientation of the line.

Space limitations on the geological map obviously limit the number of planes which can be represented in the above manner. Nevertheless, for giving a general impression of the principal discontinuity orientations they can be quite useful.

Further detail can be obtained by using different symbols to represent the various types of discontinuities. For example, the following symbols are often used to represent joints, bedding and foliation:

joints bedding foliation

A clear key to symbol terminology should always be given.

The outcrop of major discontinuities should be drawn directly on geological maps. For example thick continuous lines (——) can be used for major, persistent discontinuities that are visible, and thick broken lines (– – – –) for major discontinuities whose persistence is implied, but which are locally covered.

(*b*) *Block diagrams*. At an early stage in the assessment and communication of raw field data it is helpful to present orientation measurements qualitatively using some visual technique. Perspective drawings such as that shown in Fig. 2(a) help to give an overall view of the relationship between the engineering structure and the rock mass structure. (If available, a stress ellipsoid giving the measured principal stress vectors might also be presented on such a diagram, to aid in the evaluation of the optimum orientation of the structure.)

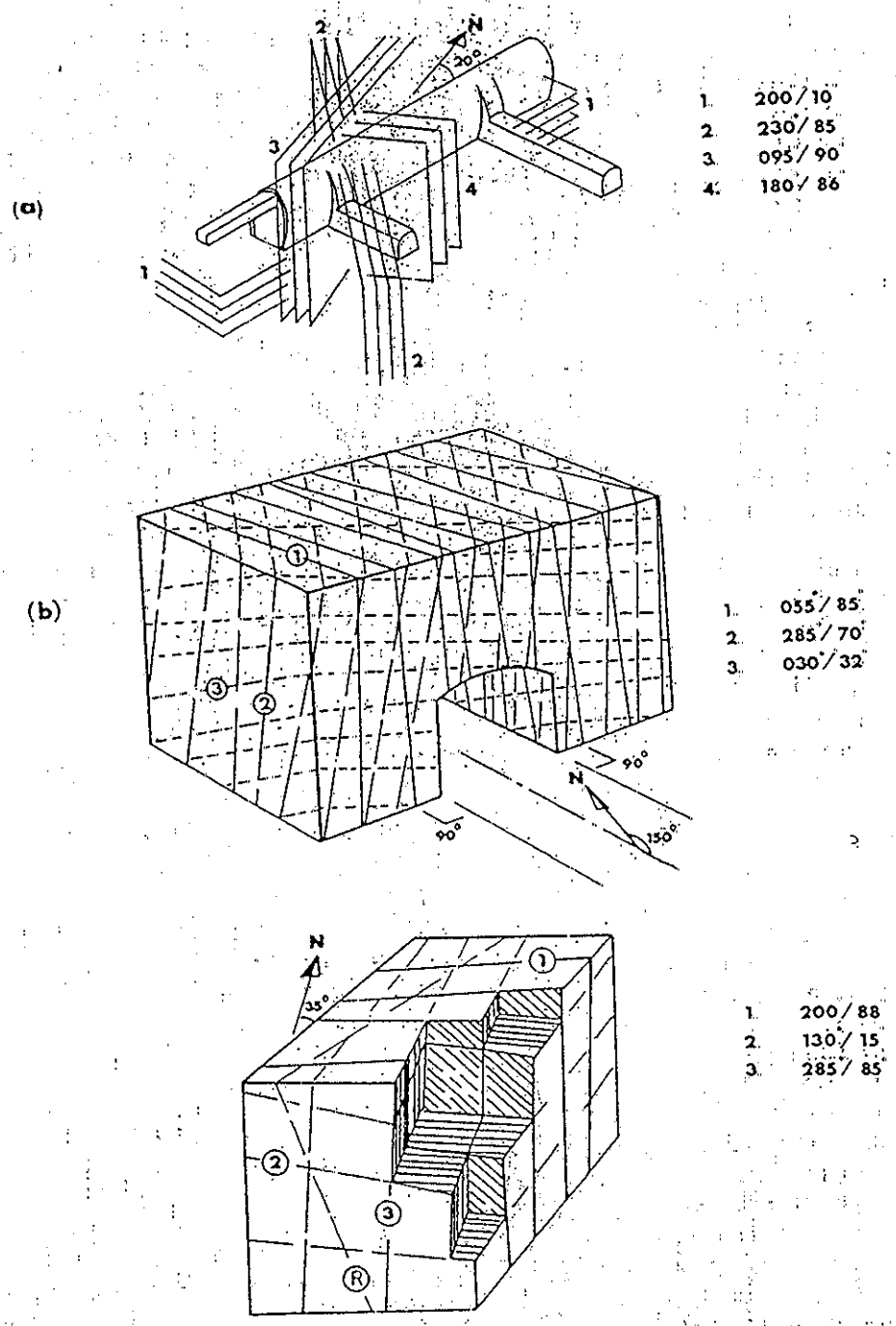


Fig. 2 Perspective views and block diagrams provide a qualitative picture of jointing and its relationship to engineering structures

On a more detailed scale *block diagrams* can be used, such as that illustrated in Fig. 2(b). Many types of structure can be represented in this idealized manner, for example tunnel portals, cross-sections through tunnels or large rock caverns, rock slopes, dam abutments etc. (Depending upon the scale the discontinuity spacing and persistence may be represented in addition to the orientation.)

(Block diagrams showing "excavated" corners as in Fig. 2(c) give a visual impression of the rock structure. They are also a useful substitute for photographs where foliage or soil cover partly obscure the exposure.

In the examples shown in Fig. 2 it is helpful to number the joint sets, show the orientation relative to true N. and list the dip direction and dip at the side of the diagram. (This is also helpful when presenting photographs of rock mass structures.)

(c) *Joint rosettes.* A common method of plotting and presenting a large number of orientation measurements in a more quantitative manner than the above is by means of joint rosettes.

In this instance measurements are represented on a simplified compass rose, marked from 0-360° (or 0-400^g) with radial lines at 10° (or 10^g) intervals. Observations are grouped in the nearest 10° sectors.

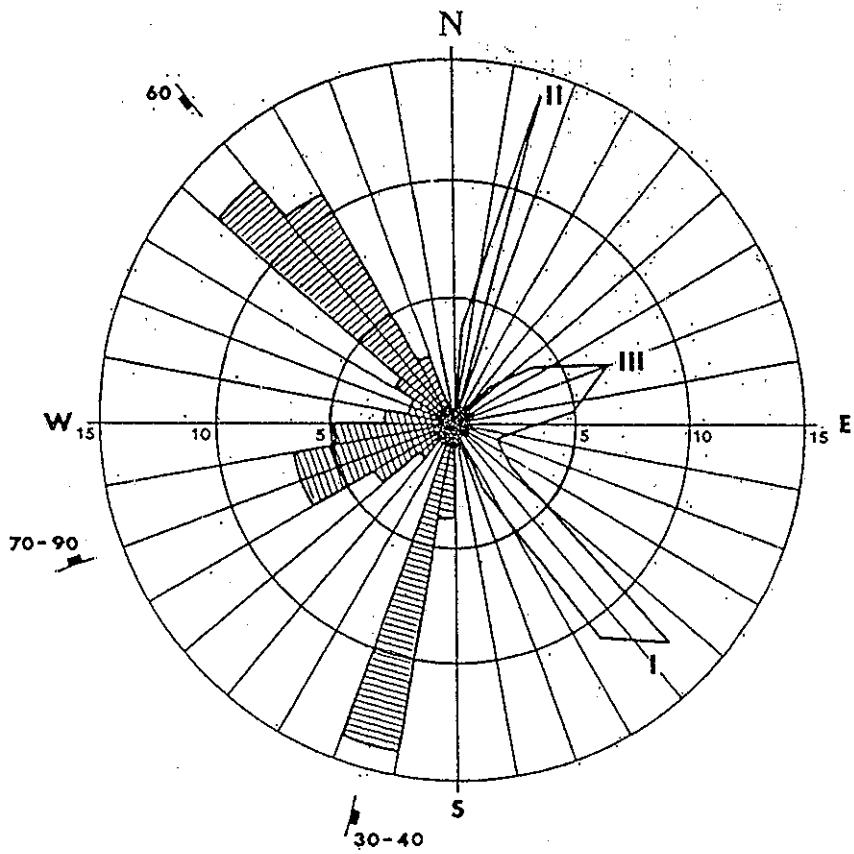


Fig. 3. Two methods of representing orientation data on a joint rosette.

he number of observations are represented along the adial axes, using numbered concentric circles repre-enting 5, 10 and 15 observations, or as convenient. he resulting strike "petals" have mirror images about he centre of the rosette. The range of dip observations or each discontinuity set cannot be represented within he rosette and must therefore be shown outside the rcumference.

Note that measurements of strike or dip direction f sub-horizontal discontinuities are inherently unreli-ble. Therefore in general, such features cannot be epresented satisfactorily using joint rosettes.

It should be noted that although the joint rosette s a widely used polar diagram it misrepresents the data o some extent. Large concentrations are exaggerated nd small concentrations are suppressed. This bias esults from the fact that areas in each angle sector ary with the square of the radial coordinate, whereas n a true histogram the area of each bar or sector hould vary with the frequency, not with the square f the frequency. (Accordingly the polar diagrams hould ideally have a square-root radial scale, Pincus 2]).

Figure 3 shows two methods of representing orien-ation data on a joint rosette. The observations grouped n the nearest 10° (or 10^{g}) sectors can be represented ther as solid radial sectors (left hand side), or their ike values averaged resulting in sharp "petals" (right hand side). The latter method reduces the bias referred to above, but may not be satisfactory if there is little dispersion of the data.

(The radius of the polar diagram can be used to good effect in plotting other parameters than the frequency of observation. A particularly useful parameter is the total observed length of discontinuities of given orientation.)

(*d*) *Spherical projection.* Several projection methods are used to represent the orientation of geological planes. The geological text books listed in the reference give comprehensive discussions of the various techniques available. In this short summary only one projection will be mentioned, the *equal area projection*. (In this method the spatial distribution of data is accurately represented on a Schmidt, or Lambert net. In the case of *equal angle projection* the angular relationships between features are accurately represented by plotting data on a Wulff net.)

A discontinuity plane (α/β) can be uniquely represented as a great circle or as a pole on a reference hemisphere, when the centre of the sphere lies in the plane of the discontinuity. (See Fig. 4a.) For engineering purposes the lower reference hemisphere is used. A two dimensional representation is obtained by projecting this information onto an equal area net.

In Fig. 4(a) the pole P of the discontinuity K is the point of intersection of the normal to the plane with the lower hemisphere. To plot the pole on a polar equal area net (Fig. 4b), the dip β is counted from the centre of the net at right angles to the strike towards the periphery.

To plot the plane as a great circle on an equatorial equal area net (Fig. 4c), the strike ($\alpha + 90°$) is counted

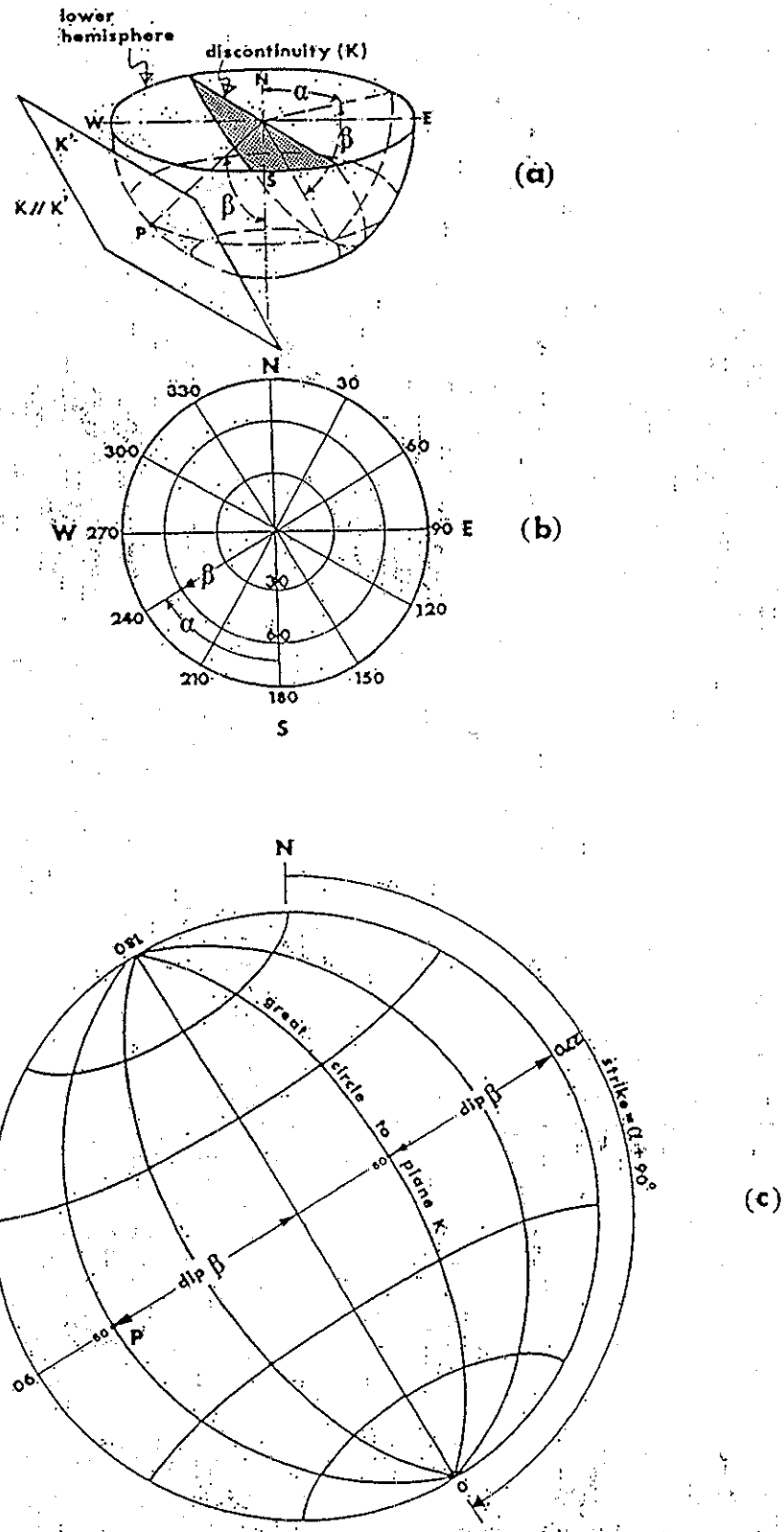


Fig. 4. Method of representing a discontinuity K as a pole P and as a great circle on a polar equal-area net (b) and on an equatorial equal-area net (c) using the lower reference hemisphere. A rotatable transparent overlay is used with the equatorial equal-area net

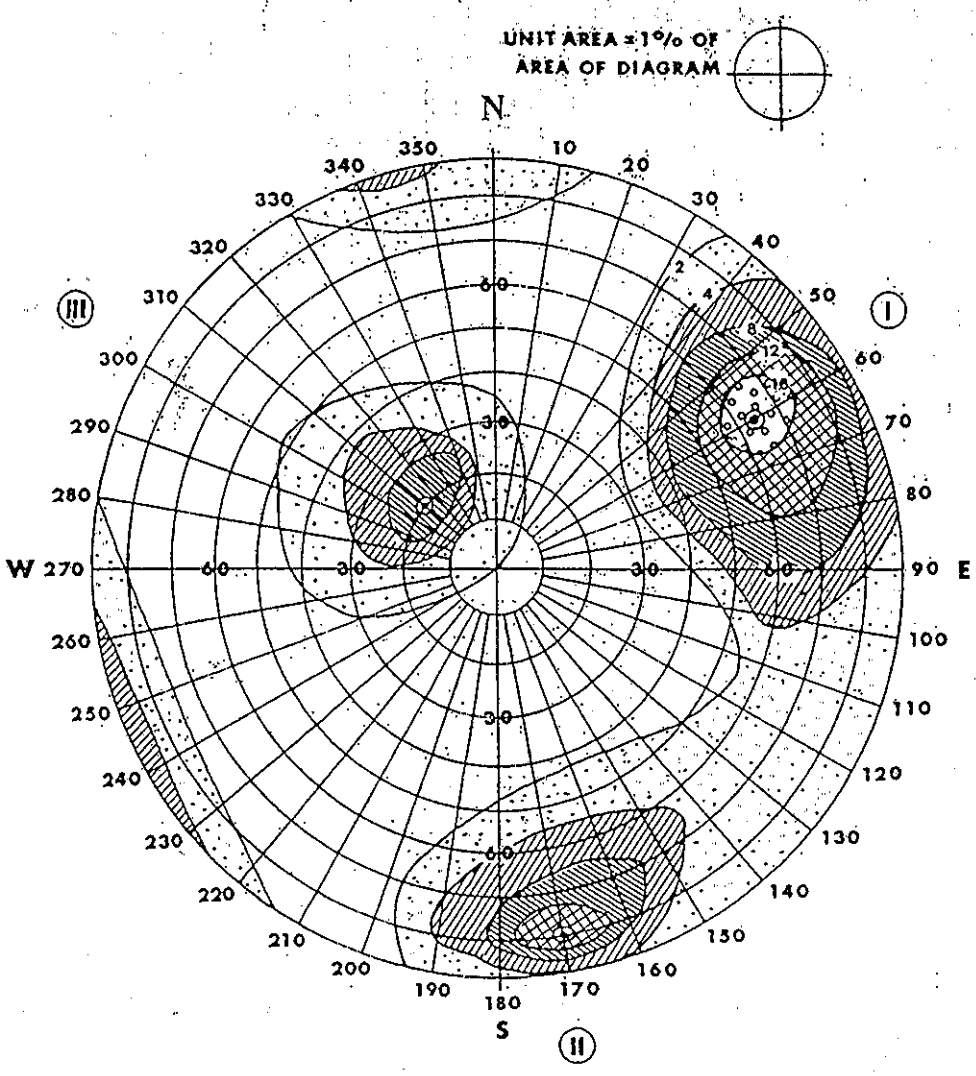


Fig. 5. Schmidt contour diagram representing the orientation of three sets of joints plotted on a polar equal-area net. The main sets I and II are approximately normal to each other, and the minor set III is nearly horizontal.

from north clockwise on the periphery, using a rotatable tracing or plastic overlay on which N has been marked. The dip is plotted at right angles to the strike, measured from the periphery towards the centre. The pole P can also be represented on the equatorial equal area net, both nets yielding the same geometrical distribution of poles.

The polar equal area net is the most convenient for plotting poles as no rotation of overlay is necessary. The first step in obtaining mean orientation data for the different discontinuity sets requires that clusters of poles can be visually recognised. The Schmidt contouring method is used to determine the pole densities, an example of which is shown in Fig. 5.

The contouring involves superimposing a square grid on the equal area net. A circle, shown in Fig. 5, which represents 1% of the total area of the equal area net, is placed with its centre at the grid intersections. The number of poles within the circle is counted and noted on each grid intersection. Pole densities can then be contoured, using up to six contour intervals.

The central value of highest concentration of poles can be taken as representing the mean orientation of the given set of discontinuities. However, since there are variations from the mean, orientation is strictly a random variable with a certain dispersion associated with each mean value. Probability techniques are recommended for a more precise analysis. (It should be noted that density contours obtained by the Schmidt method violate probability theory since poles are counted more than once.)

Figure 6 illustrates the use of equatorial equal area nets for plotting both poles and great circles to represent typical rock mechanics problems, such as slope stability. Spherical projection methods are of greatest value where stability depends on the relative three dimensional orientation of discontinuities and free surfaces.

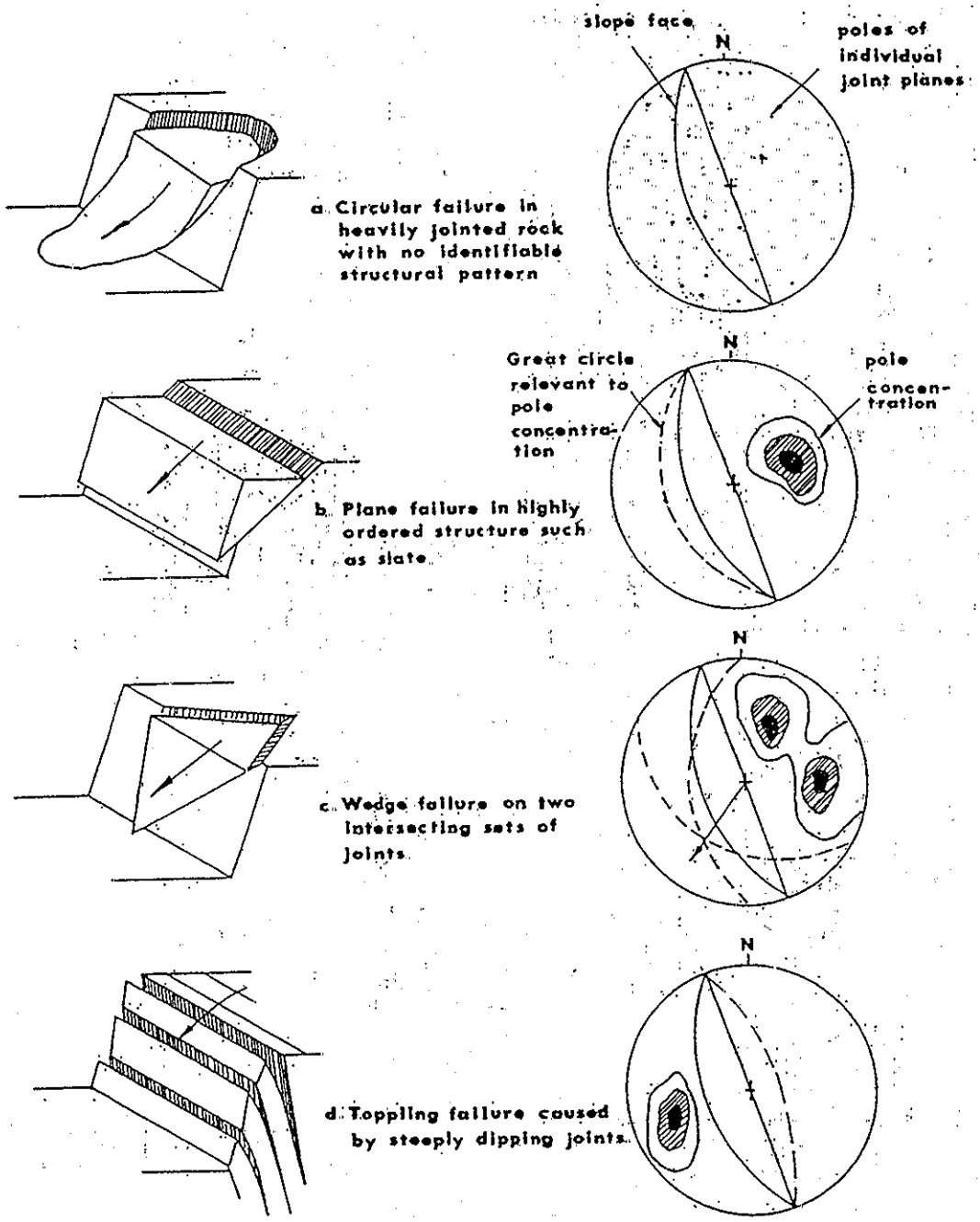


Fig 6. Representation of structural data concerning four possible slope failure modes, plotted on equatorial equal-area nets as poles and great circles. [3].

REFERENCES

1. Rocha M. & Barroso M. Some applications of the new integral sampling method in rock masses. *Symposium of ISRM on Rock Fracture*. Nancy. Paper 1–21. 12 p. (1971).
2. Pincus H. J. The analysis of aggregates of orientation data in the earth sciences. *J. Geol.* 61, 482–509 (1953).
3. Hoek E. & Bray J. *Rock Slope Engineering*. The Institution of Mining and Metallurgy. London. 309 p. (1974).
4. Badgley P. C. *Structural Methods for Exploration Geologists*. Harper Brothers. New York. 280 p. (1959).
5. Hodgson R. A. Regional study of jointing in the Comb Ridge-Navajo Mountain area Arizona and Utah. *Bull. Am. Ass. Petrol Geol.* 45. 1–38 (1961).
6. John K. W. An approach to rock mechanics. *J. Soil Mech Fdns Div. Am. Soc. civ. Engrs.* SM4, pp. 1–30 (1962).
7. Muller L. *Der Felsbau*. Ferdinand Enke-Verlag. Stuttgart. 624 p. (1963).
8. Turner F. J. & Weiss L. E. *Structural Analysis of Metamorphic Tectonites*. McGraw-Hill. New York. 545 p. (1963).
9. Pincus H. J. A procedure for rapid plotting of joint diagrams. *J. Geol. Educ.* Vol. 13. 7–8 (1965).
10. Terzaghi R. D. Sources of error in joint surveys. *Geotechnique*. 15, 287–304 (1965).
11. Nickelsen R. P. & Hough V. N. D. Jointing in the Appalachian Plateau of Pennsylvania. *Bull. geol. Soc. Am.* 78, 609–630 (1967).
12. John K. W. Graphical stability of slopes in jointed rock. *J. Soil Mech. Fdns Div. Am. Soc. civ. Engrs* 94, No. SM2. 497–526. with discussion and closure in 95. SM6. 1969. 1541–1545 (1968).
13. Broadbent C. D. & Rippere K. H. Fracture studies at the Kimberley Pit. *Proc. Symposium on Planning Open Pit Mines*. Johannesburg. 1970. Balkema. Amsterdam. 1971. pp. 171–179 (1971).

ton F. D. & Deere D. U. Significant geological factors in k slope stability. *Symposium on Planning Open Pit Mines*, ıannesburg, 1970. Balkema. Amsterdam 1971. pp. 143–151 71).
llips F. C. *The Use of Stereographic Projections in Structural ɔlogy*. Edward Arnold, London. 3rd edn. 90 p. (1971).
ɜau D. R. Geological factors significant to the stability of pes cut in rock. *Symposium on Planning Open Pit Mines*. ıannesburg. 1970. Balkema, Amsterdam 1971, pp. 33–53 (1971).
bertson A. MacG. The interpretation of geological factors for : in slope theory. *Symposium on Planning Open Pit Mines*. ıannesburg. 1970. Balkema. Amsterdam 1971, pp. 55–71 (1971).
ill J. L. The engineering geology of the Cruachan underground ver station. *Engng Geol.* 6, 289–312 (1972).
ɔcock E. A. Regional jointing in Southern Alberta. *Can J th Sci.* 10, 1769–1781 (1973).
cus H. J. Note: A modified transit for measuring strike on underside of surfaces. *Int. J. Rock Mech. Min. Sci* 10, 83–84 73).
e[illegible] R. Characterizing and extrapolating rock properties [illegible]ering practice. *Rock Mechanics* (Springer-Verlag). Suppl 5–[illegible]973).
lstead D. D. & Mahtab M. A. A Bureau of Mines direct read- azimuth protractor. USBM. Information Circular 8617. 7 p. 74).
rding E. J. & Mahar J. W. The effect of natural geologic dis- ıtinuities on behaviour of rock in tunnels. *Proc. 2nd North ıerican Rapid Excavation and Tunneling Conference*. San Fran- ɔo, Vol. I, Chap. 12. pp. 107–138 (1974).
ɜch R. *Bermekanikk* (Geoteknik. Ed. S. Hansbo). Almqvist & ksell, Stockholm, 236 p. (1974).
ıden D. M. A composite net for rock slope stability. *Q. Jl. ɜng. Geol.* 9, 119–124 (1976).
odman R. E. *Methods of Geological Engineering in Discon- ıous Rocks*. West Publishing N.Y. 472 p. (1976).

(B) *Photogrammetric Method*

This discontinuity mapping technique utilizes grammetry to determine the coordinates of at f[illegible] points on each visible discontinuity plane. [illegible]fining the orientation of the given planes. p[illegible]es may often be mapped quite precisely by hotogrammetric technique, but the accuracy de- :s rapidly as the area of the plane decreases.

The method is usually only economic if the orien- of a large number of discontinuities is required. ver, there are occasions when photogrammetry only practical alternative, for example if the rele- ock face is in the vicinity of magnetic anomalies, he rock face is unstable and/or inaccessible.

The following summary of equipment and pro- ɜs is designed as an introduction to the technique tial users should consult the detailed papers listed references.

Equipment

(*a*) Reconnaissance survey equipment; optical square, Abney level, alidade and reconnaissance diagram mounted on a plane table.

(*b*) Phototheodolite and tripod. A phototheodolite is a theodolite with a survey camera located between the upper and lower circles. The survey camera incorporates fiducial marks and has a lens of negligible distortion characteristics. Six control targets are required for location on the rock face to be photographed. In order to be seen clearly in the stereoscopic model their minimum dimensions should be $\frac{1}{400}$ of the distance to the rock face. Their colour should be chosen for maximum contrast with the rock when viewed in black and white photography. Photographic plates, photographic development facilities (on site if possible, to check for poor exposures) and light meter are also required.

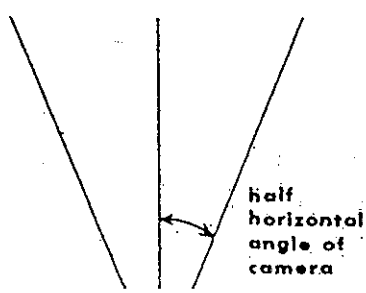
(*c*) Control survey equipment: tripods, tribrachs, tripod targets, plumbing devices, subtense bar.

(*d*) Stereoscopic plotting instrument or stereocomparator, with automatic recording equipment (i.e. punched tape). This equipment will normally be operated by a trained photogrammetrist.

Procedure

(*a*) *Reconnaissance survey*. The purpose of a reconnaissance survey is to determine suitable positions for both the camera stations overlooking the face, and for control targets on the face. (See Figs 7 and 8). The height of the face being photographed, the accuracy required, the vertical and horizontal field angles of the camera and the available camera tilt must be considered prior to photography. In many cases there will be physical limitations imposed by the site itself, as illustrated in Fig 9. Much better use of the overlap area is possible if the camera axes can be approximately normal to the face.

(*b*) *Photography*. The phototheodolite is set up on one of the base line tripods, with an interchangeable target on the other. The instrument is then levelled, the camera tilt, exposure time and counter are set, and the photographic plate is loaded. The camera is orientated at right angles to the theodolite, and the telescope is sighted on the other station. With the camera axis thus normal to the base, the photograph is taken. The phototheodolite and target are then interchanged at the



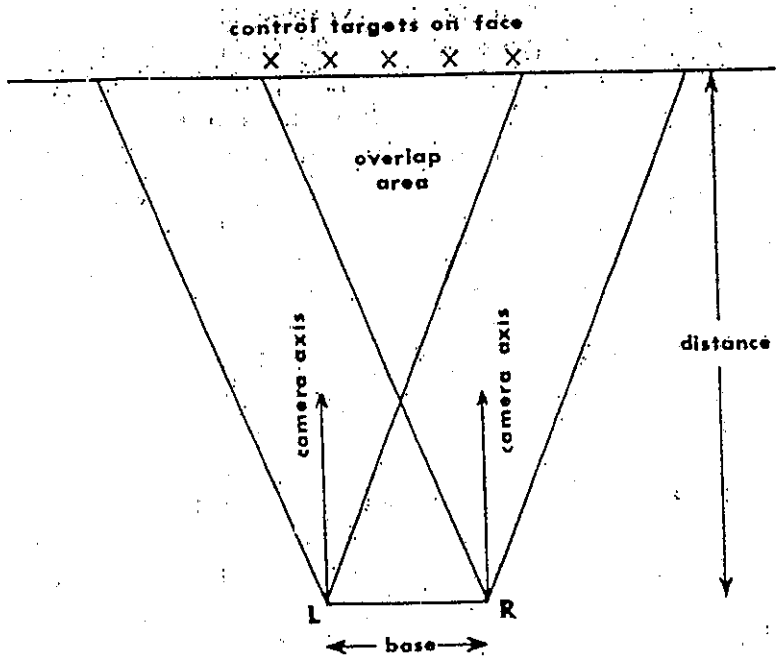


Fig. 8. Field set-up to obtain overlapping stereopair

base-line stations and the procedure is repeated. It is recommended that the photographic plates are developed in a suitable site office dark room so that, if the plates are not up to the high standard required for photogrammetric analysis, the photography may be retaken before the camera station tripods and control targets are removed. It is desirable to complete all the photography as soon as possible in order to avoid differences caused by shadow on corresponding photographs of a stereopair.

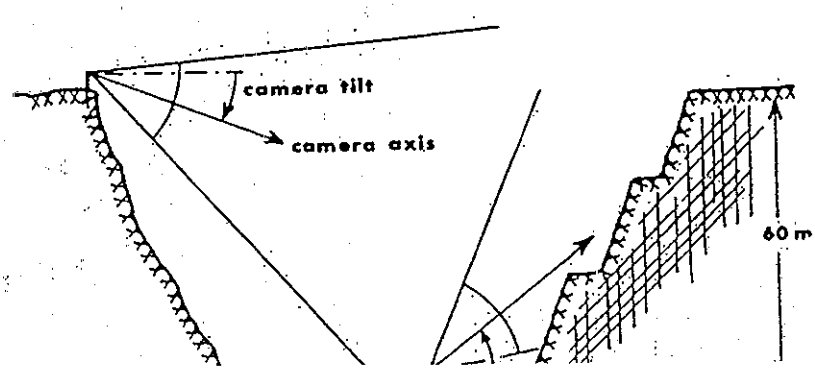
(*c*) *Control survey.* After completion of the photography a control survey has to be performed in order to determine the coordinates of at least four targets within the overlap area. The camera can be removed from the theodolite and the necessary angle measurements recorded from each end of the baseline. Generally two rounds of horizontal and vertical angles are made to the control targets and at least three other stations whose coordinates are known. From these latter observations the camera coordinates may be determined by resection.

The baseline is measured by setting an interchan able subtense bar on one station tribrach, and obse ing it from the other. The distance is calculated fr the mean subtended angle. This procedure is perform from both ends of the baseline as a check.

A minimum of one day should normally be allov for the field work associated with each stereopair. T baseline may subsequently be extended to a series consecutive camera stations if the overlap a obtained with one stereopair is insufficient to cover whole rock face.

(*d*) *Survey information.* The exact form of the sur information required depends on the program be used to analyse the results. Generally, if the theodc observations have been made from the same tribr positions as used for the photography, the survey in mation required consists of the theodolite coordin in the ground system, and the vertical and horizo theodolite observations to the targets reduced meaned as appropriate.

(*e*) *Instructions to photogrammetrist.* It is conven



to work in a routine manner in order that the information may subsequently be handled by a computer. The work is best specified by making detailed notes for the photogrammetrist and by making an enlarged photograph of the overlap area. The following information may be requested:

Joint areas—areas indicated on the enlarged photograph from which a specified number of orientations are required for statistical analysis e.g. plotting on an equal area net.

Special discontinuities—particular planes, individually identified on the enlarged photograph, for which the location, orientation and extent are required more precisely, as for example for use in a stability analysis. Generally up to ten pointings per plane are sufficient for defining these features.

(*f*) *Observational procedure.* Usually the negative plates are observed directly but if preferred by the operator diapositives can be made. An operator unaccustomed with the technique of observing discontinuities usually requires a few hours observing practice. The coordinates of at least four points are required for each visible plane. Each pointing is punched onto tape in an identical format and consists of an identifier followed by X, Y and Z coordinates of the pointing. Normally all the pointings referring to a particular discontinuity have the same identifier. The operator thus proceeds from pointing to pointing, discontinuity to discontinuity and area to area. About 10% of the larger discontinuities are identified on the large photographic print for the convenience of the engineering geologist doing the interpretation. It is important that the operator makes a number of independent checks on the accuracy of his observations at field scale. This will give all concerned a feel for the likely errors.

(*g*) *Computations.* The basic information required consists of the control survey data (*c*) and the photogrammetric punched tape (*f*). In summary, computer calculations comprise transformation of the target coordinates to the ground system and setting up the transformation matrix.

Planes are fitted to the sets of pointings by the method of least squares, and direction cosines are determined from a symmetric coefficient matrix and subsequently transformed by the transformation matrix. The planes may then be described in terms of *dip direction* and *dip*. The last part of the computational phase involves the calculation of probable errors. Special techniques are used to estimate the maximum probable errors in dip and dip direction for each joint [1].

Notes

(*a*) In any photogrammetric system the following sources of error have to be considered; film, camera, plotting instrument, recording method, control survey, earths curvature, atmospheric refraction, instrument operator. Compared to the other sources of error, the operating errors caused by the instrument operator are very significant. These are mainly due to the limitations in the operator's stereoscopic perception and due to misinterpretation. The operator must make arbitrary decisions as to the positioning of the floating mark in the instrument if discontinuity images are poorly defined. These operating errors can usually be kept to tolerable levels by using large base/distance ratios.

(*b*) In highly altered or weathered rocks it may be difficult to distinguish discontinuities and geological features even by close inspection. In such cases photogrammetry is clearly of little help. Sometimes very rough or very curved discontinuities are encountered and the validity of fitting a plane to such surfaces may be questioned. The error in plane fitting may be negligible for discontinuities defining near-perfect planes with any orientation, and for planes normal to the camera axis of any roughness. However, the error may be significant for very rough planes approaching the edge-on position when viewed on the photographic plates. This is especially true of discontinuities which strike within 5° of the direction of the camera axes. If photogrammetry is the main mapping technique being used, then more than one stereo-pair taken from different directions may be required to pick up all the discontinuities exposed on a face. Alternatively the edge-on discontinuities may be mapped conventionally in order to make the equal area net complete.

(*c*) There is a great deal of useful information that can be obtained from the photogrammetric mapping technique in addition to orientation data. For example, rock surface profiles can be plotted for use in estimating overall volumes involved in the stability analyses. If the camera to object distance is reasonable, *roughness* profiles of individual joints may be obtained. These may be used to estimate shear strength. The overall distribution of joint *spacing* can be measured and joint *persistence* may also be assessed. In addition, stereo pairs exposed at different stages during the life of a project (e.g. an open pit), provide a permanent visual record, which can be especially useful when extrapolating major features.

Presentation of results

Suggested methods for presenting orientation data will be found under (*A*) *Compass and Clinometer Method*.

The large amount of orientation data likely to be produced by systematic photogrammetric work calls for statistical treatment. A first step in the presentation of results will be the plotting of poles on equal-area nets.

REFERENCES

1. Ross-Brown, D. M.; Wickens E. H. & Markland J. T. Terrestial photogrammetry in open pits; 2—an aid to geological mapping. *Trans. Inst. Min. Metall.* (Section A. Min. Industry) 82, pp. A115–A130 (1973)
2. Linkwitz, K. Terrestrisch-photogrammetrische Kluftmessung. *Rock Mech. Engng Geol.* 1, 152–159 (1963)
3. Terzaghi R. D. Sources of error in joint surveys. *Geotechnique* 15, 287–304 (1965)

4 Savage J. F. Terrestial Photogrammetry for Geological Purposes. International Training Centre for Aerial Survey publication Series B No. 33, pp. 41–53 (1965).
5 Thompson M. M. (Ed.). *Manual of Photogrammetry*. 3rd edn, American Soc. of Photogramm. Falls Church, Virginia (1966).
6 Rengers N. Terrestial photogrammetry: a valuable tool for engineering geological purposes. Rock Mech. Engng Geol. 5, 150–154 (1967).
7 Calder P. N., Bauer A. & Macdougall A. R. Stereophotography and open pit mine design. *72nd Annual Meeting Can. Inst. Min. Metall.* April 1970. Preprint (1970).
8 Wickens E. H. & Barton N. R. The application of photogrammetry to the stability of excavated rock slopes. *Photogram. Rec.* 7(37), April, 46–54 (1971).
9 Ross-Brown D. M. & Atkinson K. B. Terrestial photogrammetry in open pits: 1—description and use of the phototheodolite in mine surveying. *Trans. Inst. Min. Metall.* (Sect. A: Min. Industry) 81, pp. A205–A213 (1972).
10 Ross-Brown D. M. Aspects of slope design in open pit mining. PhD. Thesis. University of London (1973).

2. SPACING

Scope

(*a*) The spacing of adjacent discontinuities largely controls the size of individual blocks of intact rock. Several closely spaced sets tend to give conditions of low mass cohesion whereas those that are widely spaced are much more likely to yield interlocking conditions. These effects depend upon the persistence of the individual discontinuities.

(*b*) In exceptional cases a close spacing may change the mode of failure of a rock mass from translational to circular or even to flow (e.g. a "sugar cube" shear zone in quartzite). With exceptionally close spacing the orientation is of little consequence as failure may occur through rotation or rolling of the small rock pieces.

(*c*) As in the case of orientation, the importance of spacing increases when other conditions for deformation are present, i.e. low shear strength and a sufficient number of discontinuities or joint sets for slip to occur.

(*d*) The spacing of individual discontinuities and associated sets has a strong influence on the mass permeability and seepage characteristics. In general the hydraulic conductivity of any given set will be inversely proportional to the spacing, if individual joint apertures are comparable.

Equipment

(*a*) Measuring tape of at least 3 m length, calibrated in mm divisions

(*b*) Compass and clinometer.

Procedure

(*a*) Whenever possible, the measuring tape should be held along the exposure such that the surface trace of the discontinuity set being measured is approximately perpendicular to the tape. If the tape is not perpendicular, directional bias corrections are required to obtain the true spacing.

(*b*) All distances (*d*) between adjacent discontinuities are measured and recorded over a sampling length not less than 3 m (or the thickness of the rock unit being observed if this is less than 3 m). The sampling length should preferably be greater than ten times the estimated spacing. The distances (*d*) should be measured to within 5% of their absolute values.

(*c*) The smallest angle (α) between the measuring tape and the observed joint set is measured with a compass to the nearest 5°.

(*d*) The most common (modal) spacing is calculated from the equation:

$$S = d_m \sin \alpha$$

where d_m is the most common (modal) distance

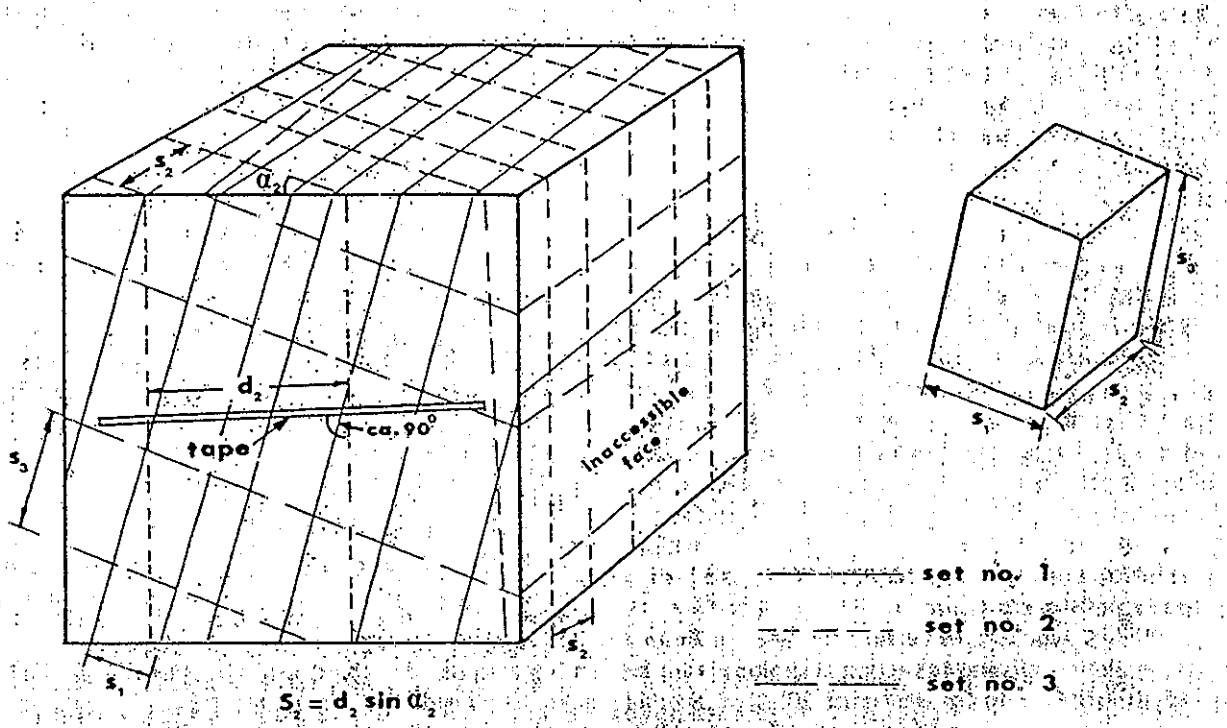


Fig. 10. Measurement of joint spacing from observation of a rock exposure

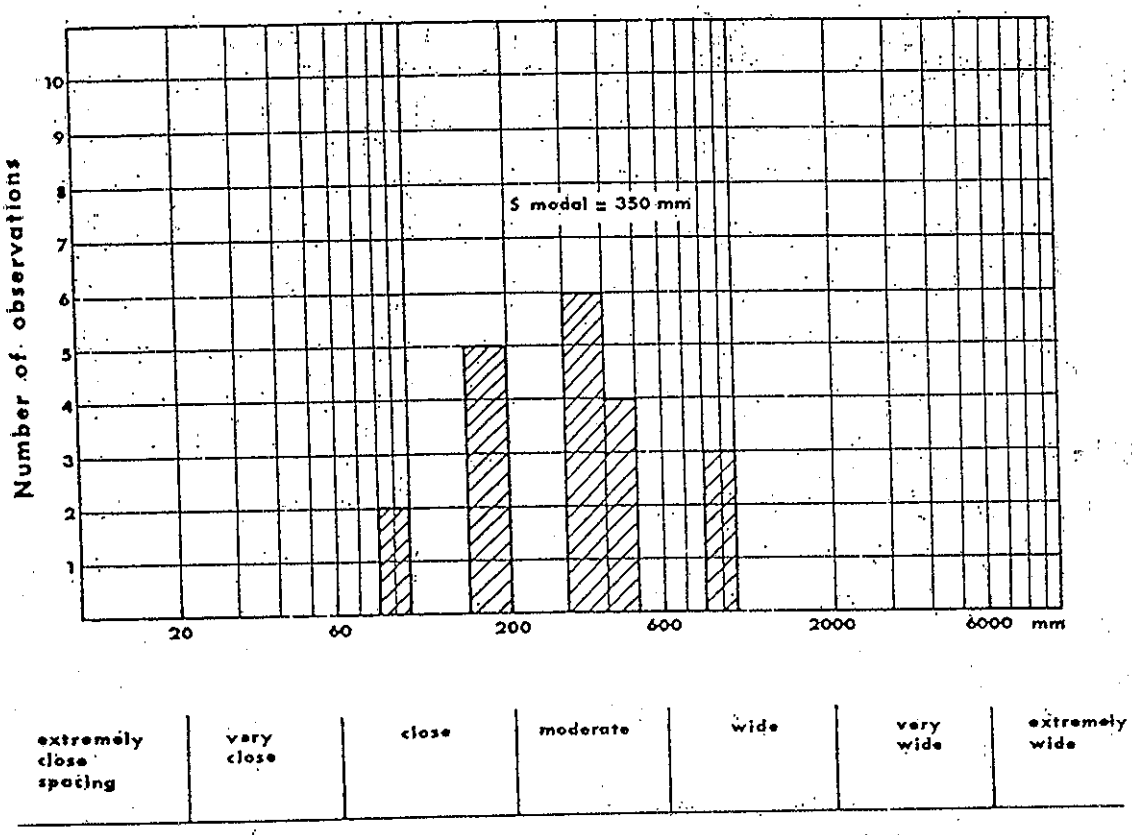


Fig. 11. Histogram showing modal, minimum and maximum spacings obtained from observations of the spacing of one set. Suggested descriptions given at base of histogram

measured. It is helpful to present the variation in spacing by means of a histogram, as illustrated in Fig 11.

Notes

(*a*) The use of a measuring tape and compass is strongly recommended, but it is not essential if the engineering geologist is experienced in taking these measurements using visual judgement. This will depend on the degree of precision required. It should be borne in mind that discontinuities such as joints may not be sufficiently parallel in a given set to justify great precision.

(*b*) The average value of individual modal spacings (S_1, S_2 etc.) represents the average dimension of typical rock blocks if persistence is assumed. Other methods of representing block size from observations of spacing are given under parameter 10. Block Size.

(*c*) In any given discontinuity set, domains with recognizably similar spacing may be separated by more massive rock containing a few widely spaced discontinuities. Block diagrams (Fig. 2b) or histograms (Fig 11) can be used to indicate this type of variability.

(*d*) In general, fractures caused by blast damage should be excluded from consideration when measuring the spacing of discontinuities.

(*e*) In cases where rock exposures are of limited extent, or absent, seismic refraction techniques can be used to estimate spacing in the upper 20-30 m. Several investigators have found a fairly reliable relationship between *frequency*, i.e. number of discontinuities per metre, and the longitudinal or compression (P) wave velocity V_p.

(*f*) The spacing or frequency of discontinuities can also be determined from analysis of drill core and from borehole viewing techniques such as borehole television cameras, photographic cameras and borehole periscopes (see 11. Drill Core for details).

Presentation of results

(*a*) The minimum, modal and maximum spacing, S (min), S, S (max) should be recorded for each discontinuity set. The distributions can conveniently be presented as histograms, one for each set (Fig. 11). The following terminology can be used:

Description	Spacing
Extremely close spacing	<20 mm
Very close spacing	20–60 mm
Close spacing	60–200 mm
Moderate spacing	200–600 mm
Wide spacing	600–2000 mm
Very wide spacing	2000–6000 mm
Extremely wide spacing	>6000 mm

(*b*) A convenient method of presenting large number of spacing measurements for which statistical treatment may be required is the use of histograms, one for each set of discontinuities. Frequency curves for each set can be drawn on the same diagram, giving an immediat

›ression of the respective modal values and disper-
ns. (Note: using mean in place of modal spacings
y help to eliminate difficulties with samples having
ltiple, poorly-defined modes, and with samples with
des at very small spacings, i.e. from negative
›onential distributions.)

c) Spacing may also be expressed as the inverse i.e.
mber of discontinuities per metre. This is termed fre-
ency.

REFERENCES

Terzaghi R. Sources of error in joint surveys. *Geotechnique*, 15, 287–304 (1965).

Weaver R. & Call R. D. Computer estimation of oriented fracture set intensity. Reprint *Symp on Computers in Mining and Exploration*. Tuscon, Arizona, 17 p. (1965).

Da Silveria A. F., Rodrigues F. P., Grossman N. F. & Mendes F. Qualitative characterization of the geometric parameters of jointing in rock masses. *Proc. 1st Congress of the Int. Soc. Rock Mech.* Lisbon, Vol. I, pp. 225–233 (1966).

Halstead P. N., Call R. D. & Rippere K. H. Geological structural analysis for open pit slope design. Kimberley pit, Ely, Nevada. Reprint, Annual AIME meeting, New York, 25 p. (1968).

Ward W. H., Burland J. B. & Gallois R. W. Geotechnical assessment of a site at Mundford, Norfolk, for a large proton accelerator. *Geotechnique*, 18, 399–431 (1968).

Broadbent C. D. & Rippere K. H. Fracture studies at the Kimberley pit. *Proc. Symp. on Planning Open Pit Mines*, Johannesburg, 1970. Balkema, Amsterdam, 1971, pp. 171–179 (1971).

Piteau D. R. Geological factors significant to the stability of slopes cut in rock. *Proc. Symp. on Planning Open Pit Mines*, Johannesburg, 1970. Balkema, Amsterdam, 1971, pp. 33–53 (1971).

Roberston A. MacG. The interpretation of geological factors for use in slope theory. *Proc. Symp. on Planning Open Pit Mines*, Johannesburg, 1970. Balkema, Amsterdam, 1971, pp. 55–71 (1971).

Grainger P., McCann D. M. & Gallois R. W. The application of the seismic refraction technique to the study of fracturing of the Middle Chalk at Mundford, Norfolk. *Geotechnique*, 23, 219–232 (1973).

Piteau D. R. Characterizing and extrapolating rock properties in engineering practice. *Rock Mechanics* (Springer-Verlag), Suppl. 2, pp. 5–31 (1973).

Priest S. D. & Hudson J. A. Discontinuity spacings in rock. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* 13, 135–148 (1976).

3. PERSISTENCE

cope

(*a*) Persistence implies the areal extent or size of a
iscontinuity within a plane. It can be crudely quanti-
ied by observing the discontinuity trace lengths on the
urface of exposures. It is one of the most important
ock mass parameters, but one of the most difficult
o quantify in anything but crude terms.

(*b*) The discontinuities of one particular set will often
›e more continuous than those of the other sets. The
ninor sets will therefore tend to terminate against the
›rimary features, or they may terminate in solid rock.

(*c*) In the case of rock slopes and dam foundations
t is of the greatest importance to attempt to assess
the degree of persistence of those discontinuities that
are unfavourably orientated for stability. The degree
to which discontinuities persist beneath adjacent rock
blocks without terminating in solid rock or terminating against other discontinuities determines the degree to which failure of intact rock would be involved in eventual failure. Perhaps more likely, it determines the degree to which "down-stepping" would have to occur between adjacent discontinuities for a failure surface to develop. Persistence is also of the greatest importance to tension crack development behind the crest of a slope.

(*d*) In the case of tunnelling, failure in the first instance may be a rather local affair; and persistence across a limited number of blocks may be all that is required provided that other conditions are compatible with failure, i.e. the existence of smooth or clay filled discontinuities or at least three sets. Planar discontinuities that can be traced without offset for 5–10 m in a tunnel construction may be of major significance to stability, while being of minor importance in the case of a 100 m high rock slope or large dam abutment.

(*e*) Frequently, rock exposures are small compared to the area or length of persistent discontinuities, and the real persistence can only be guessed. Less frequently it may be possible to record the dip length and the strike length of exposed discontinuities and thereby estimate their persistence along a given plane through the rock mass using probability theory. However, the difficulties and uncertainties involved in the field measurements will be considerable for most rock exposures encountered.

Equipment

(*a*) Measuring tape of at least 10 m length.

Procedure

(*a*) Individual rock exposures, or recognised domains, should first be described according to the relative persistence of the different discontinuity sets present. The sets of discontinuities can be distinguished by the terms *persistent*, *sub-persistent* and *non-persistent* respectively. Simple labelled field sketches such as those illustrated in Fig. 12, can be useful aids in subsequent interpretation.

(*b*) Efforts should then be made to measure the discontinuity lengths in the direction of dip and in the direction of strike. This may be impossible in the case of limited planar exposures. However, in the case of large three-dimensional exposures such as curved open pits with benches, or underground openings with intersecting tunnels, it may be possible to obtain useful size-frequency histograms for each of the discontinuity sets.

The modal trace lengths measured for each set can be described according to the following scheme:

Very low persistence	< 1 m
Low persistence	1–3 m
Medium persistence	3–10 m
High persistence	10–20 m
Very high persistence	> 20 m

(*c*) A useful procedure during the mapping of discontinuity lengths is to record the type of termination according to the following scheme. Discontinuities

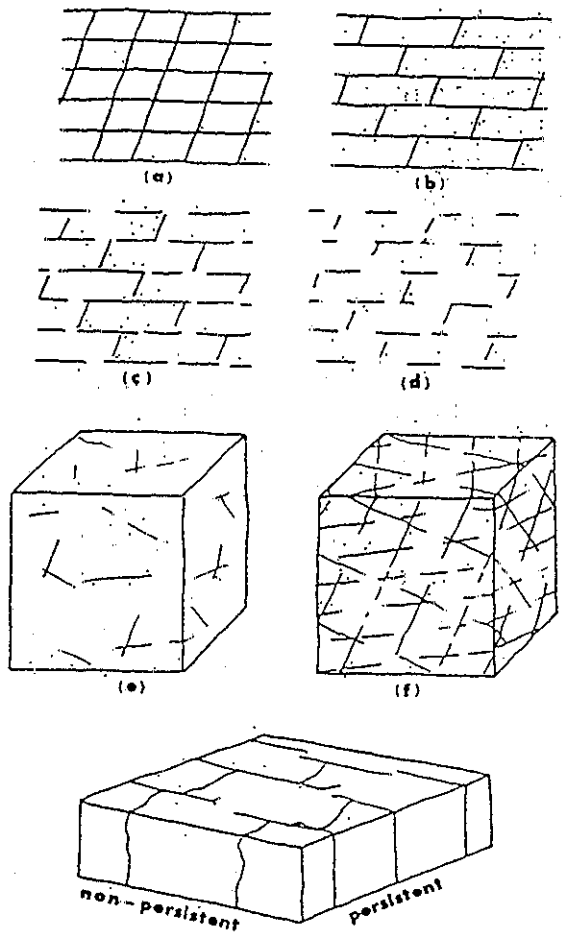


Fig. 12 Simple sketches and block diagrams help to indicate the relative *persistence* of the various sets of discontinuities. Examples adapted from [1] and [2].

which extend outside the *exposure* (x), should be differentiated from those that visibly terminate in *rock* in the exposure (r), and from those that terminate against other *discontinuities* in the exposure (d). A *systematic* set of discontinuities with a high score in (x) is obviously more persistent than a *sub-systematic* set with predominant scores in (d). *Non-systematic* discontinuities will tend to have highest scores in (r).

(*d*) Termination data (x, r or d) should be recorded for each end of the relevant discontinuities, together with the length in metres. (Example: 8(dx) = discontinuity length of 8 m, one termination against another *discontinuity* other termination invisible because feature *extends* beyond the limits of the exposure). It is important to specify the dimensions of the exposure on which measurements were made since this will obviously influence both the number of (x) observations and the relevant lengths.

Notes

(*a*) Piteau [3] has demonstrated that discontinuities where both terminations can be seen are generally smaller than discontinuities where one or no terminations can be seen. In a sample of 3844 joints at the Nchanga Mine, 1394 (36%) with an average length of 1.4 m had both ends exposed, 1538 (40%) with an average length of 2.9 m had one end exposed, and 912 (24%) with an average length of 6.3 m had no ends exposed.

(*b*) Analyses of dip lengths and strike lengths performed by Robertson [4] have indicated that discontinuities tend to be of approximately isotropic dimensions. When terminating in solid rock they may therefore tend to be circular, and presumably rectilinear when terminating against other discontinuities.

(*c*) Statistical tests simulating circular outline discontinuities with a normal distribution of diameters randomly spaced in the rock mass, indicate that the mean trace length can range from slightly smaller to slightly larger than the mean diameter [5]. This is the result of the greater probability of intersecting the larger discontinuities outweighing the fact that trace lengths (i.e chords) are inherently shorter than diameters.

(*d*) Statistical methods can be used to analyse the maximum lengths of discontinuities. Using such techniques it is possible to estimate the expected recurrence interval for discontinuities of any specified length. Alternatively it is possible to estimate the mean probability of a discontinuity exceeding a specified length occurring in any portion of the rock mass. For example, if after analysis it is found that major discontinuities with strike lengths of 50 m or more are spaced on the average at 150 m, it is possible to estimate the probability of strike lengths of 50 m or more occurring in any 100 m interval measured normal to the strike. The probability is equal to $\frac{100}{150} = 0.66$ If the complete distribution of sizes is known (*Procedure* (*b*)), the probability of occurrence of a discontinuity of a certain size can be evaluated on the basis of extreme value statistics. A useful example of its application to rock slope stability analysis is given by McMahon [6]. Note that the ill-defined lower bound to observations of trace length (inevitable if the shortest features are ignored) leads to underestimation of the frequency of discontinuities and overestimation of their size.

(*e*) The descriptive term *persistence* may in theory be quantified by defining it as the percentage of the total area of a plane through the rock mass which is formed by discontinuities coincident (co-planar) with this reference plane. In practice, waviness of most discontinuities frustrates strict interpretation. A practical alternative is to select a band width equal to the mean spacing of the discontinuities in the particular set, and to estimate the persistence within this reference band. Since, on a probability basis, only one discontinuity would be expected to occur within this band, a slightly more realistic estimate of persistence is obtained.

(*f*) When assessing the persistence of the various discontinuity sets it is important to investigate the possibility of a stepped failure surface forming, as illustrated by failure modes (2) and (3) in Fig. 13. This mode of failure may tend to occur when the set involved in shear has less than 100% persistence. Downstepping will tend to develop such that only a minimum percentage of the resulting shear surface passes through intact rock. The persistence of a potential failure surface will normally be higher than that along planes or bands

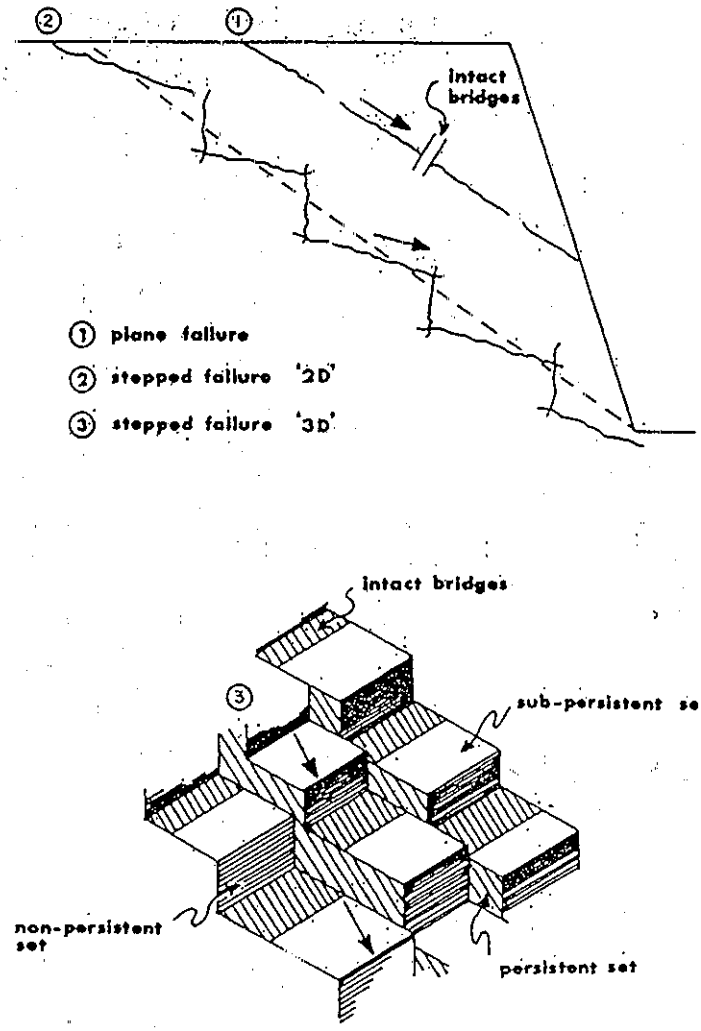


Fig 13. Idealized examples of potential failure planes showing the importance of 'intact bridges' and 'down-stepping'. Examples adapted from [4] and [7].

parallel to a single set, unless the latter have 100% persistence.

(*g*) Estimates of *persistence* for given planes, bands or specific failure surfaces have at present to be based on engineering judgement and should be purposely weighted in the direction of conservatism (i.e. closer to 100% persistence) since the shear strength of the intact rock bridges will form a dangerously high percentage of the total shear strength of the compound failure surface. The shear strength (cohesion) due to any intact rock bridges can be crudely estimated from the following relationship which is derived from the Mohr diagram, assuming a linear shear strength envelope:

$$c = \tfrac{1}{2}(\sigma_c \cdot \sigma_t)^{\frac{1}{2}}$$

where:

σ_c = uniaxial compressive strength of the intact rock

σ_t = tensile strength of the intact rock.

It is assumed for simplicity that $\sigma_c/\sigma_t = 9$, then the cohesive strength is equal to one sixth of the unconfined compressive strength. It is safer to assume 100% persistence when in doubt, since the above cohesion is usually one to two orders of magnitude greater than the shear strength of the discontinuities.

Presentation of results

(*a*) The various sets of discontinuities should be described as *systematic*, *sub-systematic* or *non-systematic* according to their relative persistence. Block diagrams or photographs should be labelled accordingly.

(*b*) Where exposures are of suitable dimensions, size-frequency histograms of trace lengths observed for each set of discontinuities should be given. (This is necessary if probability theory is to be applied subsequently). Mean trace lengths (in both strike and dip directions) should be quoted.

(*c*) Termination data which has been recorded for each discontinuity sampled (e.g. 8dx), should be presented in the form of a *termination index* (T_r) for the rock mass as a whole, or for chosen domains. T_r is defined as the percentage of the discontinuity ends terminating in rock (Σr) compared to the total number of terminations ($\Sigma r + \Sigma d + \Sigma x$). The latter is equal to twice the total sample since each trace has two ends.

$$T_r = \frac{(\Sigma r) \times 100}{2(\text{no. of discontinuities observed})}\,\%$$

(It is to be hoped that systematic collection of data

concerning T_r through application of these ISRM Suggested Methods will eventually improve the estimation of *persistence*)

(*d*) The *persistence* of potential failure surfaces (including stepped surfaces) should be estimated, if this is appropriate to the project being investigated. The estimate should perhaps be rounded upwards, to the next multiple of 10% (i.e. 92% is assumed to be 100%).

REFERENCES

1. Muller L. *Der Felsbau* Ferdinand-Enke-Verlag, Stuttgart. 624 p. (1963).
2. Price N. J. *Fault and Joint Development in Brittle and Semibrittle Rock* Pergamon, Oxford. 176 p. (1966)
3. Piteau D. R. Characterizing and extrapolating rock joint properties in engineering practice. *Rock Mechanics* (Springer-Verlag), Suppl. 2, pp. 5–31 (1973).
4. Robertson A. MacG. The interpretation of geological factors for use in slope theory. *Symp. on Planning Open Pit Mines*, Johannesburg, 1970. Balkema, Amsterdam 1971, pp. 55–71 (1971).
5. Barton C. M. An analysis of rock structure and fabric in the CSA Mine, Cobar, NSW CSIRO Division of Applied Geomechanics, Tech. Paper No. 24 (1976).
6. McMahon B. K. Design of rock slopes against sliding on preexisting fractures. *Proc. of 3rd. Cong. of Int. Soc. Rock Mech.* Denver, *Advances in Rock Mechanics*. Vol. IIB, pp. 803–808 (1974).
7. Jennings J. E. A mathematical theory for the calculation of the stability of slopes in open cast mines *Symp. on Planning Open Pit Mines*, Johannesburg, 1970, Balkema, Amsterdam 1971, pp 87–102 (1971).
8. Hodgson R. A. Regional study of jointing in the Comb Ridge-Navajo Mountain area, Arizona and Utah. *Bull. Am. Ass. Petrol Geol.* 45, 1–38 (1961).
9. John K. W An approach to rock mechanics. *J. Soil Mech Fdns Div. Am Soc. civ. Engrs* SM4, 1–30 (1962).
10. McMahon B. K. Indices related to the mechanical properties of jointed rock. *Proc of 9th. Symp. on Rock Mech., Status of Practical Rock Mech.*, Ch. 6, pp. 117–133 (1967).
11. Nickelsen R. P. & Hough V. N. D. Jointing in the Appalachian Plateau of Pennsylvania. *Bull Geol Soc. Am.* 78, 609–630 (1967).
12. Halstead P. N., Call R. D. & Rippere K. H. Geological structural analysis for open pit slope design. Kimbley Pit Ely, Nevada AIME, Preprint No. 68-Am 85 (1968).
13. Piteau D. R. Geological factors significant to the stability of slopes cut in rock. *Symp. on Planning Open Pit Mines*, Johannesburg, 1970. Balkema, Amsterdam 1971, pp. 33–53 (1971).
14. Babcock E. A. Regional jointing in Southern Alberta. *Can. J. Earth Sci.* 10, 1769–1781 (1973).
15. Bernaix J. Properties of rock and rock masses. (General report.) *Proc. 3rd. Cong. Int. Soc. Rock Mech.* Denver. *Advances in Rock Mechanics*. Vol. IA. pp. 9–38 (1973).
16. Cruden D. M. Describing the size of discontinuities *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr* (Pergamon) 14, 133–137 (1977)

4. ROUGHNESS

Scope

(*a*) The wall *roughness* of a discontinuity is a potentially important component of its shear strength, especially in the case of undisplaced and interlocked features (e.g. unfilled joints). The importance of wall roughness declines as aperture, or filling thickness, or the degree of any previous displacement increases.

(*b*) In general terms the *roughness* of discontinuity walls can be characterized by a *waviness* (large scale undulations which, if interlocked and in contact, cause dilation during shear displacement since they are too large to be sheared off) and by an *unevenness* (small scale roughness that tends to be damaged during shear displacement unless the discontinuity walls are of high strength and/or the stress levels are low, so that dilation can also occur on these small scale features).

(*c*) In practice *waviness* affects the initial direction of shear displacement relative to the mean discontinuity plane, while *unevenness* affects the shear strength that would normally be sampled in a laboratory or medium scale *in situ* direct shear test (see Fig. 14).

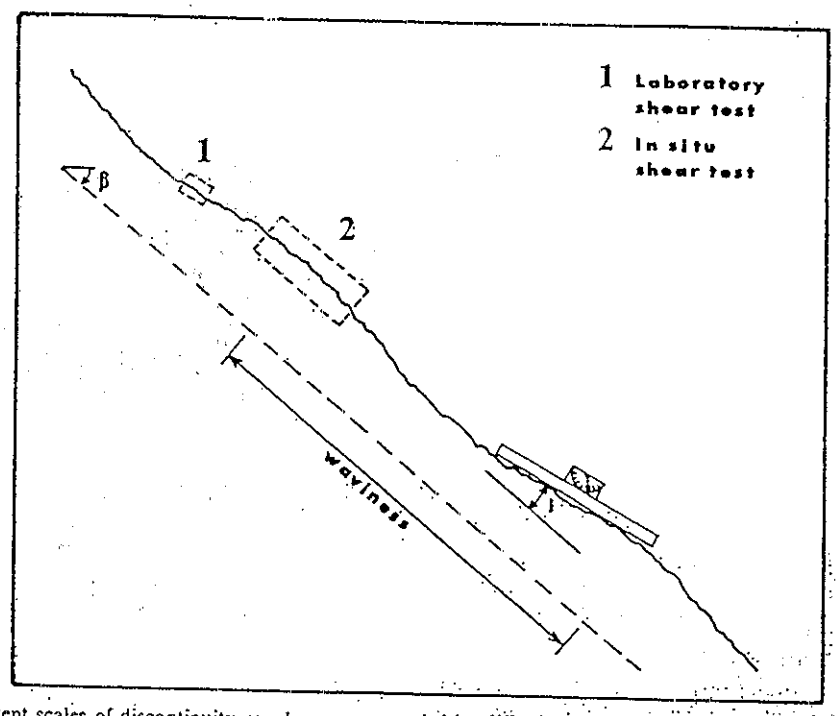


Fig. 14 Different scales of discontinuity roughness are sampled by different scales of tests. Waviness can be characterised by the angle (i).

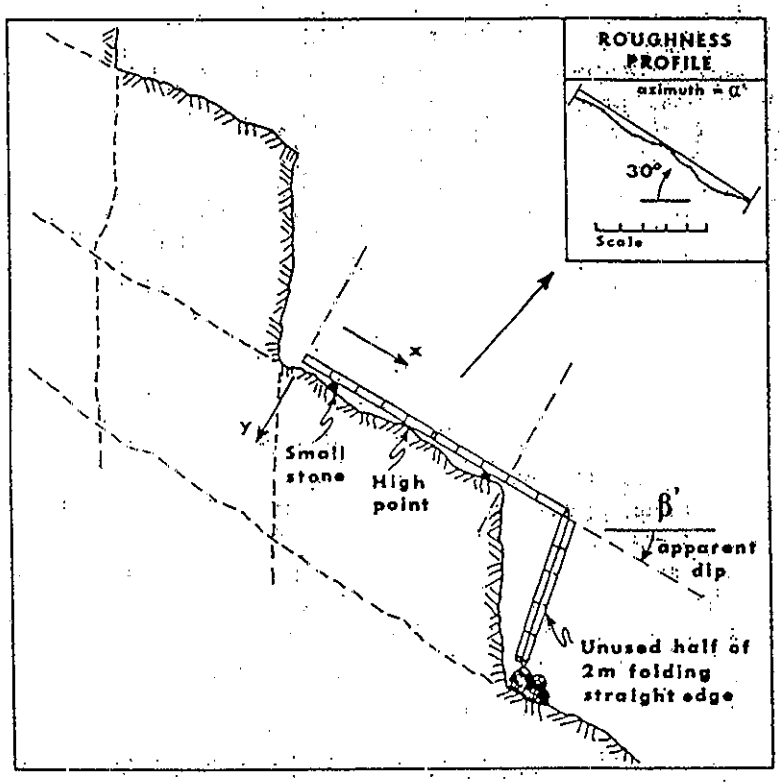


Fig 15 A method of recording discontinuity roughness in two dimensions, along the estimated direction of potential sliding

(*d*) If the direction of potential sliding is known, roughness can be sampled by linear *profiles* taken parallel to this direction. In many cases the relevant direction is parallel to the dip (dip vector). In cases where sliding is controlled by two intersecting discontinuity planes, the direction of potential sliding is parallel to the line of intersection of the planes. In the case of arch dam abutment stability, the direction of potential sliding may have a marked horizontal component.

(*e*) If the direction of potential sliding is unknown, but nevertheless of importance, roughness must be sampled in three dimensions instead of two. This can be done with a *compass and disc-clinometer*. Dip and dip direction readings can be plotted as poles on equal-area nets. Alternatively, discontinuity surfaces can be contoured relative to their mean planes using *photogrammetric* methods. This can be a useful technique if the critical surfaces are inaccessible.

(*f*) The purpose of all roughness sampling methods is for the eventual estimation or calculation of shear strength and dilation. Presently available methods of interpreting roughness profiles and estimating shear strength are summarised under the section *Presentation of results*.

Equipment

(*a*) The *linear profiling* method of sampling roughness requires the following equipment: (i) folding straight edge of at least 2 m length graduated in mm, (ii) compass and clinometer, (iii) 10 m of light wire or nylon with paint markings at 1 m intervals (red) and 10 cm intervals (blue). The line should be attached to small wooden blocks or similar at each end, so that it can be tensioned to form a straight reference line above the plane of large undulating discontinuities.

(*b*) The *compass and disc-clinometer* method of sampling roughness requires the following equipment: (i) Clar (Breithaupt) geological compass which incorporates a horizontal levelling bubble and a rotatable lid which is connected to the main body of the compass through a graduated hinge for recording dip, (ii) four thin circular plates made of light alloy of various diameters (i.e. 5, 10, 20 and 40 cm) which can be fixed in turn to the lid of the compass [1].

The photogrammetric method of sampling roughness requires assorted equipment described under *Photogrammetric Method* (page 14).

Procedure

(*a*) *Linear profiling*. Discontinuities are selected that are accessible and typical of the surface presumed to be involved if shear failure was to occur.

Depending upon the relevant dimensions of each plane either the 2 m straight edge or the 10 m wire (or sections of either) are placed or stretched above the plane of the discontinuity *parallel* to the *mean direction* of potential sliding. For convenience they should be in contact with the highest point or points of the discontinuity and they should be as straight as possible. (A small lump of "plasticene" can be helpful in preventing the straight edge from sliding down steeply dipping joints. It can be placed between the straight edge and the high spots.) The perpendicular distances (y) from the straight edge (or wire) to the surface of the discontinuity are recorded to the nearest mm, for given tangential distances (x) (see Fig. 15). It is advisable to be flex-

ible in the choice of (x) since a regular interval (for example 5 cm) might result in missing a small step or similar feature of potential importance to the shear strength. On average, (x) intervals equal to approximately 2% of the total measuring length are sufficient to give a good overall impression of roughness.

The (x) and (y) readings are recorded in parallel, together with the azimuth and dip of the measuring direction. This may be different from the orientation α/β of the discontinuity.

Profiles typical of the minimum, most common and maximum roughness are recorded using the above procedures. These profiles may apply to a whole discontinuity set, to one critical discontinuity, or to each surface measured, depending upon the detail required.

The waviness angle (i) illustrated in Fig. 14, should be recorded using the straight edge and clinometer, if the profile was so short that waviness was not automatically sampled during profiling.

The approximate wave length and amplitude of waviness too large to be sampled by profiling should be estimated, or measured where accessibility is no problem.

Photographs representing the surfaces of minimum, modal and maximum roughness should be taken, with a 1 m rule placed against the surface in question clearly visible.

(*b*) *Compass and disc-clinometer*. Discontinuities are selected that are accessible, and typical of the surface presumed to be involved if shear failure was to occur.

The small scale roughness angles (i) (Fig. 16) are measured by placing the largest circular plate (e.g. 40 cm dia) against the surface of the discontinuity in at least 25 different positions, and recording dip direction and dip for each position. (A surface area at least ten times as large as the area of the largest plate is assumed).

This procedure is repeated in turn for the other plate diameters. The overall sensitivity of the measurements is improved if a large number of positions are recorded with the smaller plate diameters, for example 50 positions with a 20 cm plate, 75 positions with a 10 cm plate and 100 positions with a 5 cm plate.

Each set of dip direction and dip data is plotted on a separate equal area net in terms of poles. Contours are drawn for each set of poles.

Photographs representing surfaces of minimum, modal and maximum roughness should be taken, with a 1 m rule placed against the surfaces in question clearly visible.

(*c*) *Photogrammetric method*. In special cases, terrestial photogrammetry can be used to obtain the coordinates of numerous points on the surface of inaccessible discontinuities using the procedures outlined under *Photogrammetric Method* (page 14). From this data it is possible to compute contour maps or profiles of the surface roughness. The minimum contour intervals will depend on the distance of the camera base from the surface in question. In some instances 1 mm intervals might be achieved, though 1 cm or 5 cm would be more likely. Profiles should be computed for the direction of potential sliding, if this is known.

Notes

(*a*) *Linear profiling*. The mm graduated ruler used to measure the perpendicular distances (y) should be tapered to a point so that the fine details of roughness can be recorded if desired.

Several automatic recording profilographs are described in the literature [1, 3]. Most of these are suitable for describing the finest details of roughness. They obviously give a much more accurate picture of roughness than that obtained by the present suggested method. Normally this accuracy is unnecessary for rock mechanics purposes.

Offsets or steps dividing a discontinuity surface into several parallel planes are indicative of lack of *persistence*, and should be carefully profiled.

There are many other methods of recording roughness in addition to the profiling method. For example the *wave-length* and *amplitude* of surface features could be measured and recorded for several different scale intervals, i.e. <1 cm, 1–10 cm, 10–100 cm, >1 m. Alternatively a very large undulating joint exposure could be rapidly recorded by laying a straight edge (for example 1 m length) against the surface at 1 m intervals in the down-dip direction and recording the dip of each position by means of a clinometer fixed to the straight edge. The length of straight edge could be varied in the same manner as with the compass method, if desired.

(*b*) *Compass and disc-clinometer*. The smallest base plates give the greatest scatter of readings and also the largest roughness angles. The largest base plates give the least scatter of readings and also the smallest roughness angles.

The large number of dip direction and dip readings (from approximately 200 plate positions) represents at least one hours work per sampled plane. This will only be justified in special circumstances. If a large number of discontinuities need to be measured, the *photogrammetric* method is recommended. Alternatively if the potential sliding direction is known, the *profiling* method is recommended, thereby reducing the amount of data collection to the single direction of potential sliding.

The maximum roughness angles for the given disc sizes can be plotted for any direction of potential sliding (See Fig. 16). The tangent of these maximum roughness angles multiplied by the appropriate base length (disc diameter) gives the displacement (dilation) that will occur perpendicular to the discontinuity for a shear displacement equal to the given base length. Several base lengths (disc diameters) are analysed in this way, so that a dilation curve can be obtained. This will give a realistic picture of the shearing process when there is minimal damage to asperities. The method is therefore most appropriate to shearing of joints in hard rocks at low effective normal stress levels. (Asperites smaller than the minimum plate diameter are assumed

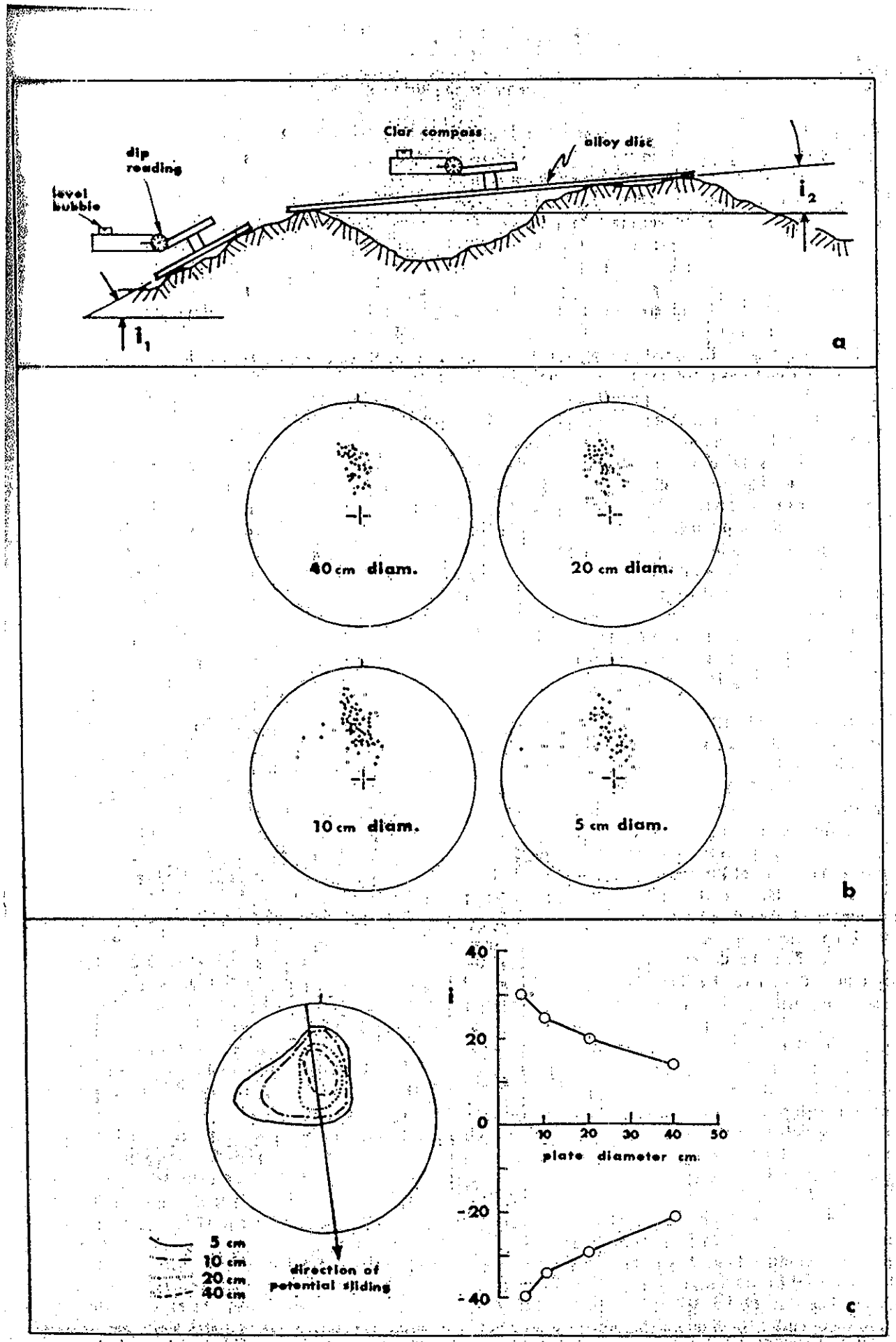


Fig. 16. A method of recording discontinuity roughness in three dimensions, for cases where the potential direction of sliding is not yet known. Circular discs of different dimensions (e.g. 5, 10, 20 and 40 cm) are fixed in turn to a Clar compass and clinometer. The dip direction and dip readings are plotted as poles on equal-area nets. Adapted from [1] and [2].

not to influence the process of dilation). See Fecker and Rengers [1] for further details.

(*c*) *Photogrammetric method.* The coordinates representing points on the surface of the given discontinuity are recorded using a stereoscopic plotting instrument or a stereo comparator, with automatic recording equipment (i.e. punched tape). Roughness profiles can be drawn by computer.

Methods are available for estimating the shear strength and dilation characteristics of discontinuities (specifically unfilled joints), based on statistical analysis of these surface coordinates [4, 5].

Presentation of Results

(*a*) *Linear profiling.* The (x) and (y) readings should be plotted to the same scale (not distorted), and inclined correctly, as shown diagrammatically in the inset to Fig. 15. Profiles representing the minimum, most common, and maximum roughness should be drawn on the same page to make comparison easier. The three profiles may represent a discontinuity set, a single critical discontinuity, or each surface sampled. This will depend on the amount of detail required. A scale should be included in all the drawings. Profiles should be identified clearly, and the azimuth and dip of the measuring direction should be stated, in case this differs from the previously recorded orientation α/β of the discontinuity.

Photographs of the relevant surfaces showing minimum, modal and maximum roughness should be presented together with the profiles.

(*b*) *Compass and disc-clinometer.* The field measurements of dip direction and dip obtained with the various diameters of discs should be plotted as poles on equal area nets, one for each disc. These can be combined and presented on a single contoured plot, as shown in Fig. 16.

Measurements from several discontinuities of a given set may be grouped on the same equal area net if desired, to show the range of roughness (and the overall variation in orientation caused by any waviness).

Photographs of the relevant surfaces showing minimum, modal and maximum roughness should be presented together with the pole diagrams.

(*c*) *Photogrammetric method.* For purposes of visual presentation in a report, the most useful figures will be profiles rather than contour diagrams of surface roughness. The profiles, which will normally be plotted by computer, should be presented with 1:1 vertical: horizontal scales, in preference to exaggerated vertical scales.

If the direction of potential sliding is unknown, the profiles should be computed and presented to represent the roughness in the line of dip (dip vector direction). Correctly orientated profiles can be produced at a later stage.

Photographs of the relevant surfaces showing minimum, modal and maximum roughness should be presented together with the profiles.

(*d*) *Descriptive terms.* In the preliminary stages of field mapping (i.e. during feasibility studies) time limitations may prevent the use of the above roughness measuring techniques. The description of roughness will be limited to descriptive terms which should be based on two scales of observation:

Small scale (several centimetres)
Intermediate scale (several metres)

I Rough (or irregular), stepped
II Smooth, stepped
III Slickensided, stepped
IV Rough (or irregular), undulating
V Smooth, undulating
VI Slickensided, undulating
VII Rough (or irregular), planar
VIII Smooth, planar
IX Slickensided, planar

The term "slickensided" should only be used if there is clear evidence of previous shear displacement along the discontinuity.

The intermediate scale of roughness is divided into three degrees; stepped, undulating and planar, and the small scale of roughness superimposed on the intermediate scale is also divided into three degrees, rough (or irregular), smooth, slickensided. The direction of striations or slickensides should be noted as shear strength may vary with direction. Roughness profiles typical of the nine classes are illustrated in Fig. 17.

(The effective roughness angles (i) displayed by the nine categories of profile mean that in terms of shear strength, I > II > III, IV > V > VI and VII > VIII > IX assuming that mineral coatings are entirely absent, or present in equal amounts. It is also evident that I > IV > VII, II > V > VIII, III > IX and VI > IX. Some of the inequalities are less certain. For example VII might be stronger than III. This would depend on whether or not dilation was inhibited. Around an underground excavation dilation is usually inhibited by the stiffness of the surrounding rock mass. Beneath a rock slope it may not be).

There may also be a large scale *waviness* superimposed on the above small and intermediate scales of observation. In such cases these characteristics should also be noted i.e. smooth, undulating (class V) with large scale *waviness* 10 m wave length, 50 m amplitude.

The descriptions associated with persistence, i.e. systematic, sub-systematic, non systematic will obviously be of the greatest importance in determining the relative importance of the above descriptions of roughness.

Estimation of shear strength

The main purpose in describing the roughness of the walls of discontinuities is to facilitate the estimation of shear strength, in particular in the case of unfilled discontinuities where estimates may be quite accurate.

In crude terms shear strength will consist of a maximum (peak) or minimum (residual) friction angle, or some intermediate angle (depending upon the degree

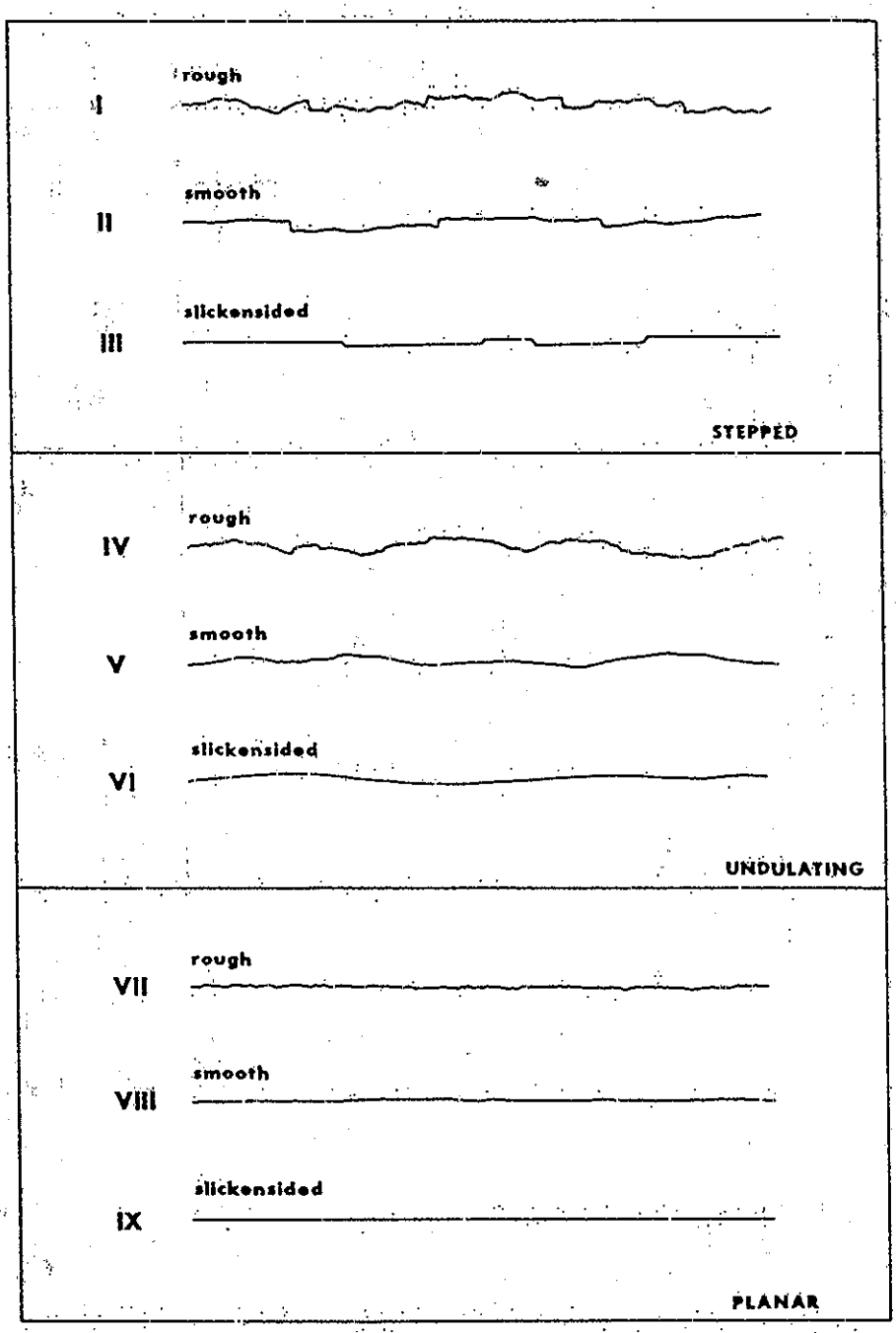


Fig. 17 Typical roughness profiles and suggested nomenclature. The length of each profile is in the range 1 to 10 metres. The vertical and horizontal scales are equal

of previous shear displacement) plus a contribution (i) due to large scale waviness, if this exists.

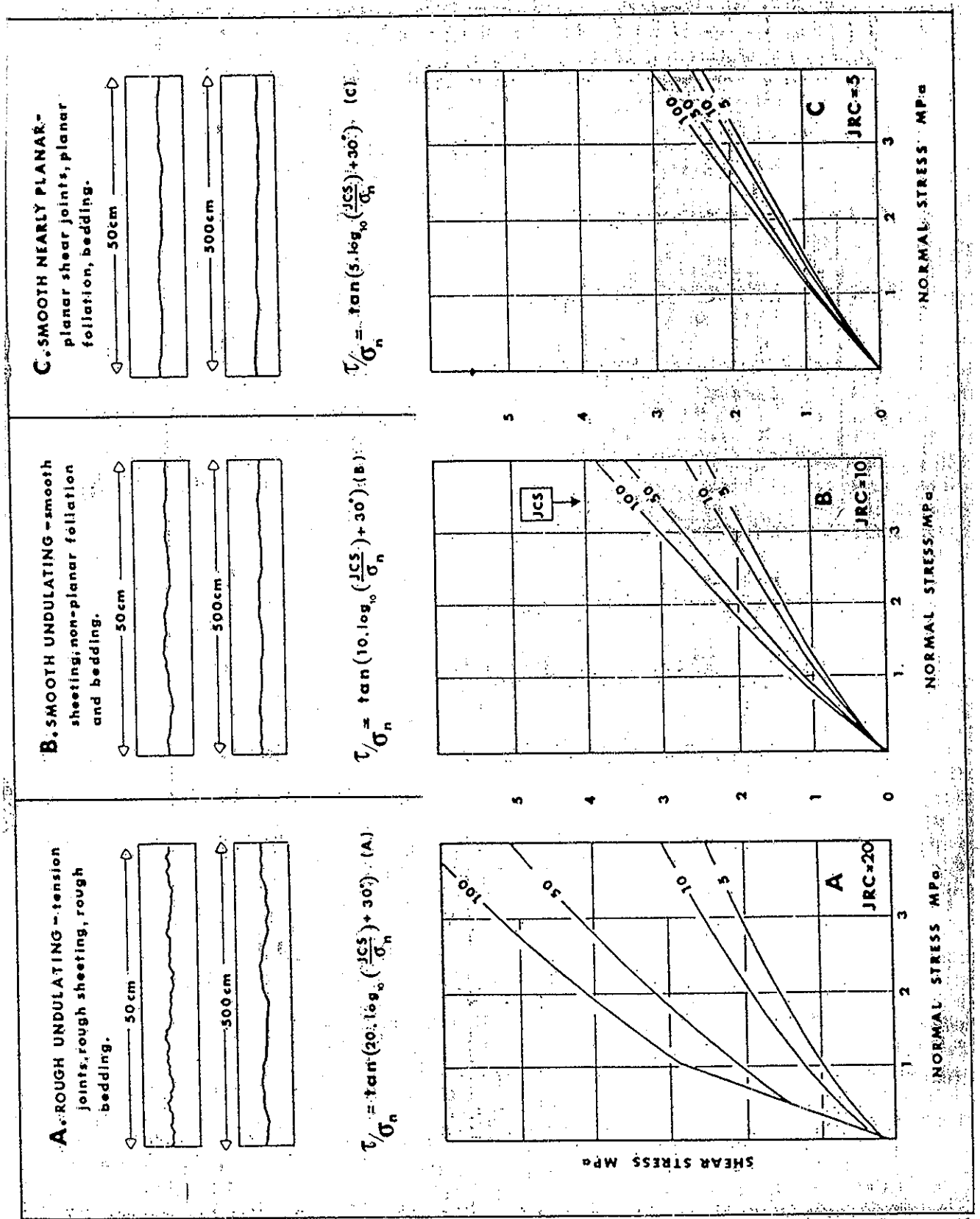
Thus $\tau = \sigma'_n \tan(\phi + i)$

τ = shear strength (peak or residual)
ϕ = friction angle (peak or residual)
σ'_n = effective normal stress
i = waviness (if present)

The value of $\tau_{(peak)}$ will depend on the value of σ'_n and on the degree of roughness. In the case of unfilled joints $\phi_{(peak)}$ values generally range from about 30 to 70° and commonly average about 45°. In the case of joints having vertical or very steep steps, or less than 100% persistence, there will also be a cohesion (c) to add to the above value of τ: (e.g. profiles I, II, and III, Fig. 17).

The value of $\phi_{(residual)}$ will depend on the degree of weathering of the discontinuity walls and on the rock type. In the absence of weathering $\phi_{(residual)}$ usually varies from about 25 to 35°, most commonly around 30°. In the case of strongly weathered walls, the value may fall to around 15°, even in the absence of actual clay fillings. A method of estimating $\phi_{(residual)}$ is described by Barton and Choubey [6]. The estimate is based on the ratio between the Schmidt hammer rebound (r) obtained on the weathered joint wall and the rebound (R) obtained on the unweathered rock.

Values of ϕ_{peak} can be estimated using the following


A. ROUGH UNDULATING – tension joints, rough sheeting, rough bedding.
50cm
500cm
$\tau/\sigma_n = \tan(20.\log_{10}(\frac{JCS}{\sigma_n}) + 30^\circ)$ (A)
B. SMOOTH UNDULATING – smooth sheeting, non-planar foliation and bedding.
50cm
500cm
$\tau/\sigma_n = \tan(10.\log_{10}(\frac{JCS}{\sigma_n}) + 30^\circ)$ (B)
C. SMOOTH NEARLY PLANAR – planar shear joints, planar foliation, bedding.
50cm
500cm
$\tau/\sigma_n = \tan(5.\log_{10}(\frac{JCS}{\sigma_n}) + 30^\circ)$ (C)
SHEAR STRESS MPa
NORMAL STRESS MPa
A JRC=20
B JRC=10
C JRC=5
JCS
100
50
10
5
0
1
2
3
4
5

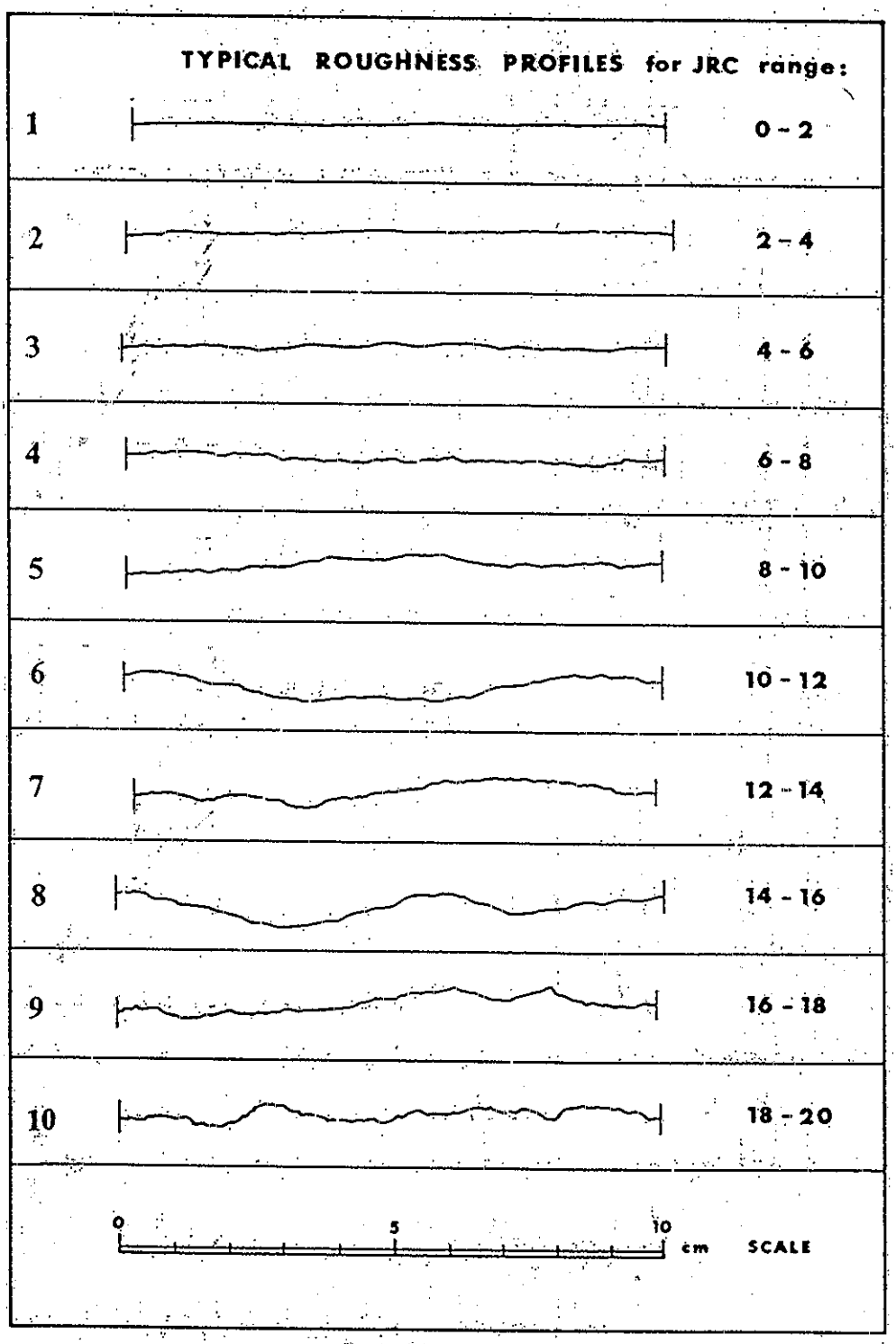


Fig. 19. Roughness profiles and corresponding range of JRC values associated with each one [6].

rmula:

$$\phi_{peak} = JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_r$$

here

JRC = joint roughness coefficient
JCS = joint wall compression strength
$\phi_r = \phi_{(residual)}$

The method of application is illustrated in Fig. 18. irstly, the measured roughness profiles are matched ith the three sets given at the top of Fig. 18, to obtain n estimate of the appropriate JRC value. (More detailed profiles are given in Fig. 19 to facilitate this quantification). Secondly, the discontinuity walls are tested with a Schmidt hammer to estimate JCS and ϕ_r. Note that in Fig. 18. ϕ_r has been assumed as 30° in every case. The above method is a surprisingly accurate and cheap method of estimating $\phi_{(peak)}$. Further details are given by Barton and Choubey [6].

Since peak shear strength is mobilized after relatively small displacements it may not be realistic to add the large scale waviness angle (i) to this estimate of $\phi_{(peak)}$. For most practical purposes $\phi_{(peak)}$ can be regarded

as the maximum value for a joint of 100% persistence. However, $\phi_{(residual)}$ is not mobilized until relatively large displacements have occurred, which generally makes the large scale waviness angle (i) a realistic addition to shear strength. In the case of completely planar discontinuities or discontinuities that have sheared to the extent that no further dilation is possible, then $\phi_{(residual)}$ will be the only shear strength component left, and will represent the absolute minimum shear strength for that discontinuity.

The above method for estimating the JRC value of a measured roughness profile is obviously subjective. Objective methods of analysing profiles are described in the literature by Fecker and Rengers [1] (compass and disc-clinometer method) and by Barton [5] (photogrammetric method). As described under Note (b), the method of analysing compass and disc-clinometer readings results in a dilation curve which is a plot of roughness (i) angles versus shear displacement. These (i) angles are added to ϕ_r to estimate the shear strength for displacements intermediate between peak and residual strength.

REFERENCES

1. Fecker E. & Rengers N. Measurement of large scale roughnesses of rock planes by means of profilograph and geological compass. *Rock Fracture Proc. of Int. Symp. Rock Mech.* Nancy. Paper I.18 (1971).
2. Hoek E. & Bray J. *Rock slope engineering*. The Institution of Mining and Metallurgy, London, 309 p (1974).
3. Fecker E. Geologische Kartierung des Gebietes nordwestlich von Neustadt Weinstrasse sowie Bau und Anwendung eines Profilographen. Diplomarbeit, Universität Karlsruhe (1970).
4. Rengers N. Influence of the surface roughness on the friction properties of rock planes. *Proc. of 2nd. Cong. of Int. Soc. Rock Mech.*, Belgrade, Vol. I, pp. 229–234 (1970).
5. Barton N. A relationship between joint roughness and joint shear strength. *Proc. Int. Symp. Rock Mech.* Nancy. *Rock Fracture*, Paper I. 8 (1971).
6. Barton N. & Choubey V. The shear strength of rock joints in theory and practice. *Rock Mechanics* (Springer-Verlag) 10, 1–54 (1977).
7. Barton N. Review of a new shear-strength criterion for rock joints. *Engng Geol.* 7, 287–332 (1973). (Also NGI Publ. No. 105. Oslo, 1974.
8. Patton F. D. Multiple modes of shear failure in rock and related materials. Ph.D. Thesis, Univ. of Illinois, 282 p (1966).
9. Patton F. D. & Deere D. U. Significant geologic factors in rock slope stability. *Proc. Symp. on Planning Open Pit Mines*, Johannesburg 1970, Balkema, Amsterdam, 1971, pp. 143–151 (1970).
10. Piteau D. R. Geological factors significant to the stability of slopes cut in rock. *Symp. on Planning Open Pit Mines*, Johannesburg, 1970. Balkema, Amsterdam 1971, pp. 33–53 (1971).
11. Rengers N. Unebenheit und Reibungswiederstand von Gesteinstrennflächen. Dr. Ing. Dissertation, Fakultät für Baueringenieur- und Vermessungswesen, Universität Karlsruhe, 129 p (1971).
12. Wickens E. H. & Barton N. R. The application of photogrammetry to the stability of excavated rock slopes. *Photogram. Rec.* 7(37), April, 46–54 (1971).
13. Ross-Brown D. M., Wickens E. H. & Markland J. Y. Terrestial photogrammetry in open pits: 2- an aid to the geological mapping. *Trans. Inst. Min. Metall* (Sect. A. Mining Industry 82, pp. A115–A130 (1973).
14. Schneider H. J. Rock friction—a laboratory investigation. *Proc. 3rd. Cong. of Int. Soc. Rock Mech.*, Denver. *Advances in Rock Mechanics*. Vol. II. A. pp. 311–315 (1974).
15. Richards L. R. The shear strength of joints in weathered rock. Ph.D. Thesis. Univ. of London, 427 p (1975).
16. Goodman R. E. *Methods of Geological Engineering in Discontinuous Rocks*. West Publishing. N.Y. 422 p (1976).
17. Schneider H. J. The friction and deformation behaviour of rock joints. *Rock Mechanics* (Springer-Verlag) 8, 169–184 (1976).

5. WALL STRENGTH

Scope

(*a*) The compressive strength of the rock comprising the walls of a discontinuity is a very important component of shear strength and deformability, especially if the walls are in direct rock to rock contact as in the case of unfilled joints. Slight shear displacement of individual joints caused by shear stresses within the rockmass often results in very small asperity contact areas and actual stresses locally approaching or exceeding the compression strength of the rock wall material, hence the asperity damage.

(*b*) Rock masses are frequently weathered near the surface, and are sometimes altered by hydrothermal processes. The weathering (and alteration) generally affects the walls of discontinuities more than the interior of rock blocks. This results in a wall strength some fraction of what would be measured on the fresher rock found in the interior of the rock blocks, for example that sampled by drill core. A description of the state of weathering or alteration both for the rock material and for the rock mass is therefore an essential part of the description of wall strength.

(*c*) There are two main results of weathering: one dominated by *mechanical disintegration*, the other by *chemical decomposition* including solution. Generally, both mechanical and chemical effects act together, but, depending on climatic regime, one or other of these aspects may be dominant. Mechanical weathering results in opening of discontinuities, the formation of new discontinuities by rock fracture, the opening of grain boundaries, and the fracture or cleavage of individual mineral grains. Chemical weathering results in discolouration of the rock and leads to the eventual decomposition of silicate minerals to clay minerals; some minerals, notably quartz, resist this action and may survive unchanged. Solution is an aspect of chemical weathering which is particularly important in the case of carbonate and saline minerals.

(*d*) The relatively thin "skin" of wall rock that affects shear strength and deformability can be tested by means of simple index tests. The apparent uniaxial compression strength can be estimated both from Schmidt hammer tests and from scratch and geological hammer tests, since the latter have been roughly calibrated against a large body of test data.

(*e*) Mineral coatings will affect the shear strength of discontinuities, to a marked degree if the walls are planar and smooth. The type of mineral coatings should be described where possible. Samples should be taken when in doubt.

(*f*) Procedures (a) and (b) concerning the weathering grade of the *rock mass* and the *rock material* are descriptive only. Procedures (c) *manual index tests* and (d) *Schmidt hammer tests* are increasingly quantitative. The latter is recommended for obtaining estimates of wall strength for subsequent calculation of shear strength when utilizing the wall roughness coefficient (JRC) described under Roughness.

Term	Description	Grade
Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surfaces.	I
Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition.	II
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.	III
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.	IV
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.	V
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destoyed. There is a large change in volume but the soil has not been significantly transported.	VI

Equipment

(*a*) Geological hammer with one tapered point.

(*b*) Strong pen knife or similar.

(*c*) Schmidt hammer (L type) with conversion table and graph:

(i) to correct for orientation of hammer (supplied by the manufacturer)

(ii) to convert corrected rebound number to an estimate of uniaxial strength (Fig. 20)

(*d*) Facilities for measuring the dry density of small rock samples, e.g. oven, balance, beaker, water.

Procedure

(*a*) *Weathering grade of rock mass.* The grade of weathering (or alteration) of the rock mass as a whole should be described first. The terms above are general and may be modified to suit particular situations.

(*b*) *Weathering grade of rock material.* The grade of weathering (or alteration) of the rock material comprising the walls of individual discontinuities or of the walls of a particular set of discontinuities (e.g. an unfavourably orientated set of joints) should be described according to the following scheme:

Term	Description
Fresh	No visible sign of weathering of the rock material.
Discoloured	The colour of the original fresh rock material is changed. The degree of change from the original colour should be indicated. If the colour change is confined to particular mineral constituents this should be mentioned.
Decomposed	The rock is weathered to the condition of a soil in which the original material fabric is still intact; but some or all of the mineral grains are decomposed.
Disintegrated	The rock is weathered to the condition of a soil in which the original fabric is still intact. The rock is friable, but the mineral grains are not decomposed.

The stages of weathering described above may be subdivided using qualifying terms, for example "slightly discoloured", "moderately discoloured", "highly discoloured".

(*c*) *Manual index tests.* The manual index tests detailed in the table on page 32 should be performed on the walls of discontinuities or on material representative of the walls. The choice and number of test locations will depend on the detail required. The approximate range of strength for the walls of a critical set of joints may be sufficient. Alternatively a single critical discontinuity may need to be characterised in detail.

The manual index tests can be performed on hand-sized pieces of freshly broken rock if the strength of intact rock bridges is of interest. Alternatively, the results of point load tests, if available, can be used to estimate the strength of the intact portions of any potential failure surface (see Persistence).

(*d*) *Schmidt hammer test.* The Schmidt hammer is applied in a direction perpendicular to the discontinuity wall of interest. The rock surface should be tested under saturated conditions to give the most conservative result. If the surfaces are unavoidably dry, this fact should be reported in the results. The surface should be free of loose particles, at least under the hammer position.

If the impulse from the spring-loaded projectile of the Schmidt hammer is sufficient to move the surface being tested, the resulting rebound will be artificially

Grade	Description	Field identification	Approx. range of uniaxial compressive strength (MPa)
S1	Very soft clay	Easily penetrated several inches by fist	<0.025
S2	Soft clay	Easily penetrated several inches by thumb	0.025–0.05
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05–0.10
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort	0.10–0.25
S5	Very stiff clay	Readily indented by thumbnail	0.25–0.50
S6	Hard clay	Indented with difficulty by thumbnail	>0.50
R0	Extremely weak rock	Indented by thumbnail	0.25–1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0–5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0–25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25–50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50–100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100–250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250

Note: Grades S1 to S6 apply to cohesive soils, for example clays, silty clays, and combinations of silts and clays with sand, generally slow draining. Discontinuity wall strength will generally be characterized by grades R0–R6 (rock) while S1–S6 (clay) will generally apply to filled discontinuities (see Filling).
Some rounding of strength values has been made when converting to SI units.

low. Such test results can normally be heard, since there is a "drummy" sound. These results should be ignored. For the above reason this field index test is unsuitable in a loose rock mass containing very closely spaced discontinuities. (In such cases small block samples can be removed and tested when clamped rigidly to a heavy base.)

Each surface of interest should be tested a number of times to ensure a representative set of results. It is suggested that tests are performed in groups of 10 (i.e. 10 tests per discontinuity, or 10 tests per unit area of a large critical discontinuity), applying the hammer to a new part of the surface before each impact. The five lowest readings of each group of 10 are discounted and the mean value (r) of the five highest readings is quoted.

The mean values of the Schmidt rebound (r) and rock density (γ) (see individual ISRM "Suggested Method") for a given discontinuity are used to estimate the value of the *joint wall compressive strength* (JCS) using Fig. 20 (see Note (c)).

The Schmidt hammer test can be performed on the surfaces of, or on material obtained from freshly broken rock when the strength of the intact rock bridges (σ_c) is of interest. Alternatively the results of point load tests, if available, can be used to estimate the strength of the intact portions of any potential failure surface (see Persistence).

Discontinuities with thin mineral coatings that appear quite persistent over a given surface, and which would probably prevent initial rock to rock contact should be tested with the Schmidt hammer as above, applying the hammer to the surface of the mineral coating. Depending upon the thickness of the mineral coating and its hardness, the estimate of JCS may or may not be relevant for estimation of shear strength. In all such cases of mineral coatings, the mineralogy should be described i.e. calcite, chlorite, talc, pyrite, graphite, kaolinite, etc. Samples should be taken when in doubt. An estimate of the areal extent of the coating (±10%) and the range of the thickness of the coating (mm) should be included.

Notes

(*a*) *Weathering grades of rock mass and rock material.* Distribution of weathering grades in a rock mass may

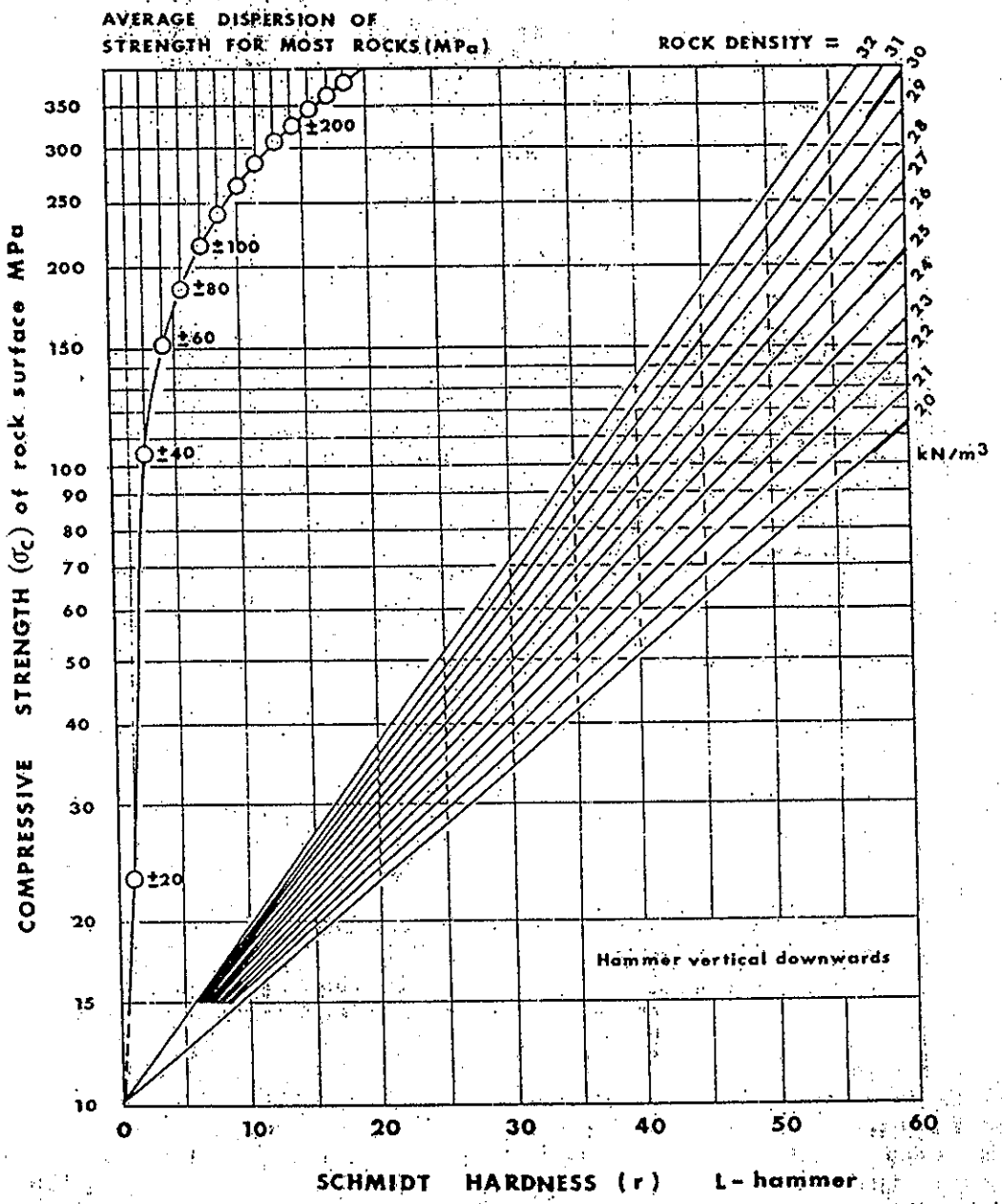


Fig 20 Correlation chart for Schmidt (L) hammer relating rock density, compressive strength and rebound number after Miller [1].

be determined by mapping natural and artificial exposures. However, it should be borne in mind that isolated natural exposures of rock and excavations of limited extent are not necessarily representative of the whole rock mass, since weathering can be extremely variable.

Furthermore, all grades of weathering may not be seen in a given rock mass, and in some cases a particular grade may be present to a very small extent. Distribution of the various weathering grades of the rock material may be related to the porosity of the rock material and the presence of open discontinuities. In logging cores the distribution of weathering grades of the rock material may be recorded, but the distribution of weathering grades of the rock mass from which the cores were obtained can only be inferred.

Rock masses which are weathered due to exposure to or infiltration from surface agents should be distinguished where possible from those that are *altered* as a result of infiltration of hydrothermal solutions. However, in many instances the effects of alteration are not easily distinguished from those brought about by weathering.

An abundant class of rock materials, notably those with high clay content, are prone to swelling, weakening or disintegration when exposed to short term weathering processes of a wetting and drying nature. Special tests are necessary to predict this aspect of mechanical performance. (See ISRM Suggested Methods for determination of swelling and slake-durability index properties.)

(*b*) *Manual index tests.* The manual index tests are

Corrections for reducing measured Schmidt hammer rebound (r) when the hammer is not used vertically downwards

Rebound r	Downwards $\alpha = -90$	$\alpha = -45$	Upwards $\alpha = +90$	$\alpha = +45$	Horizontal $\alpha = 0$
10	0	−0.8	—	—	−3.2
20	0	−0.9	−8.8	−6.9	−3.4
30	0	−0.8	−7.8	−6.2	−3.1
40	0	−0.7	−6.6	−5.3	−2.7
50	0	−0.6	−5.3	−4.3	−2.2
60	0	−0.4	−4.0	−3.3	−1.7

preferred to conventional tests on carefully prepared rock cylinders because a very large number of discontinuities can be sampled, thereby giving a more representative picture of the condition of the walls. Furthermore conventional tests cannot be applied to the thin skin of wall rock or mineral coatings that dominate the shear strength and deformability of the rock mass.

The manual index tests for determining grades S1–S6 (clay, see Filling) can be replaced by more accurate assessment using a standard soil mechanics pocket penetrometer. This contains a stylus which is pressed into the sample at a constant rate. The maximum resistance can be read off a scale which is calibrated to show the maximum compressive strength of the sample. (This value is equal to twice the undrained shear strength = $\frac{1}{2}(\sigma_1 - \sigma_3)$)

(c) *Schmidt hammer tests.* The Schmidt hammer rebound number ranges in practice from about 10 to 60. The lowest number applies to "weak" rocks (uniaxial compressive strength $\sigma_c < 20$ MPa), while the highest number applies to "very strong" and "extremely strong" rocks ($\sigma_c > 150$ MPa). "Very weak" rocks and "extremely weak" rocks cannot be tested with the L-hammer. *Manual index tests* must therefore be resorted to for rock weaker than 15–20 MPa.

For a given strength of surface the rebound number is minimum when the hammer is used vertically downwards (rebound against gravity) and maximum when used vertically upwards. The correlation given in Fig. 20 applies to vertical downwards tests only. The corrections given in the above table should be applied when the hammer is used in other directions.

Block movement (drumminess) in closely jointed rock, or crushing of loose grains are some of the reasons for unexpectedly low rebound numbers in a given set of results. Unexpectedly high readings are seldom obtained. The following two sets of actual results illustrate the suggested method of obtaining a realistic mean value:

(a) rough, planar iron-stained joints in granite

44, 36, 38, *44*, 32, *44*, *44*, 40, 34, *42*
mean of highest 5: r = 44
(mean of 8 sets of 10 tests: r = 43)

(b) rough, undulating calcite-coated joints in hornfels

28, *28*, *30*, *30*, 28, 24, 24, 28, *30*, 20
mean of highest 5: r = 29
(mean of 3 sets of 10 tests: r = 30)

The Schmidt test is one of the few tests, (with the exception of scratching tests) which takes into account the mechanical strength of the thin band of weathered wall material close to a discontinuity surface. Since it is this wall material which (in combination with *roughness*) controls the shear strength, it is of considerable importance as an index of rock quality. The *joint wall compressive strength* (JCS) is often as low as 25% of the adjacent intact rock strength (σ_c) due to weathering effects. (See section *Estimation of Shear Strength* pp. 26–30.)

Presentation of results

(a) *Weathering grades of rock mass and rock material.* The weathering grades of recognizable weathering domains in the rock mass should be recorded on simplified sketches and/or vertical sections, with a clear key indicating the different weathering grades I, II, III etc.

The weathering grade of the rock material of individual discontinuities or of specific discontinuity sets should be described, i.e. "joint set no. 1: majority of walls *moderately discoloured*, approx. 20% *fresh*".

(b) *Manual index tests.* The strength of the wall rock material of individual discontinuities or of specific discontinuity sets should be noted together with the assumed range of uniaxial compressive strength, i.e. "joint set no. 1: majority *medium strong* (R3, 25–50 MPa), approx. 20% *strong* (R4, 50–100 MPa)".

Values that are pertinent to the *discontinuity walls* should be carefully distinguished from any values that might have been recorded for the material representing the fresher rock within the rock blocks.

(c) *Schmidt hammer tests.* The mean rebound (r) for the wall rock material of individual discontinuities or of specific discontinuity sets should be noted, together with the mean rock density (γ), and the estimate of wall strength (JCS) in MPa. One set of 10 results should be selected to show the *typical* range of rebound values.

Values that are pertinent to the *discontinuity walls* should be carefully distinguished from any values that might have been recorded for the material representing the fresher rock within the rock blocks.

REFERENCES

1. Miller R. P. Engineering classification and index properties for intact rock. Ph.D. Thesis, Univ. of Illinois (1965).

2. Hucka V. A rapid method of determining the strength of rocks *in situ*. *Int. J. Rock Mech. Min. Sci.* 2, 127–134 (1965).
3. Terzaghi K. & Peck R. B. *Soil Mechanics in Engineering Practice*. 2nd edn. 729 p Wiley, New York. (1967).
4. Jennings J. E. & Robertson A. MacG. The stability of slopes cut into natural rock. *Proc. of VIIth. Int. Conf. on Soil Mechanics and Foundation Engineering*, Mexico., Vol. II. pp. 585–590 (1969).
5. Fookes P. G., Dearman W. R. & Franklin J. A. Some engineering aspects of rock weathering with field examples from Dartmoor and elsewhere. *Q. Jl. Engng Geol.* 4, 139–185 (1971).
6. Patton F. D. & Deere D. U. Significant geologic factors in rock slope stability. *Proc. Symp. on Planning Open Pit Mines*, Johannesburg 1970, Balkema, Amsterdam, pp. 143–151 (1971).
7. Piteau D. R. Geological factors significant to the stability of slopes cut in rock. *Planning Open Pit Mines*, Johannesburg, *Symp.* 1970. Balkema Amsterdam 1971, pp. 33–53 (1971).
8. Robertson A. MacG. The interpretation of geological factors for use in slope theory. *Symp. Planning Open Pit Mines*, Johannesburg, 1970. Balkema Amsterdam 1971, pp. 55–71 (1971).
9. Geological Society Engineering Group Working Party Report on "The preparation of maps and plans in terms of engineering geology". *Q. Jl. Engng Geol.* 5, 295–382 (1972).
10. International Society for Rock Mechanics. Suggested methods for determining water content, porosity, density, absorption and related properties and swelling and slake-durability index properties. ISRM Commission on Standardization of Laboratory and Field Tests (1977); this volume pp. 79–94.
11. Barton N. Review of a new shear-strength criterion for rock joints. *Engng Geol.* Amsterdam 7, 278–332 (1973). (also NGI Publication No. 105 1974, Oslo.)
12. Dearman W. R. The characterization of rock for civil engineering practice in Britain; La Géologie de l'Ingénieur. Société Géologique de Belgique, Liège. pp. 1–75 (1974).
13. Franklin J. A. Rock quality in relation to the quarrying and performance of rock construction materials. *Proc. of 2nd. Int. Cong. of the Int. Assoc. of Engng Geology*. São Paulo. Brazil. Vol. I, IV-PC-2, 11 p (1974).
14. Martin G. R. & Millar P. J. Joint strength characteristics of a weathered rock. *Proc. of 3rd. Cong. of Int. Soc. Rock Mech.* Denver. *Advances in Rock Mechanics*. Vol. II A. pp. 263–270. (1974).
15. Richards L. R. The shear strength of joints in weathered rock. Ph.D. Thesis, Univ. of London, 427 p (1975).
16. Barton N. & Choubey V. The shear strength of rock joints in theory and practice. *Rock Mechanics* (Springer-Verlag) 10, 1–54 (1977).

6. APERTURE

Scope

(*a*) Aperture is the perpendicular distance separating the adjacent rock walls of an open discontinuity, in which the intervening space is air or water filled. Aperture is thereby distinguished from the width of a filled discontinuity (see Fig. 21). Discontinuities that have been filled (e.g. with clay) also come under this category if filling material has been washed out locally.

(*b*) Large apertures can result from shear displacement of discontinuities having appreciable roughness and waviness, from tensile opening, from outwash, and from solution. Steep or vertical discontinuities that have opened in tension as a result of valley erosion or glacial retreat may have very large apertures.

(*c*) In most sub-surface rock masses apertures are small and will probably be less than half a millimetre, compared to the tens, hundreds or even thousands of millimetres width of some of the outwash or extension varieties. Unless discontinuities are exceptionally smooth and planar it will not be of great significance to the shear strength that a "closed" feature is 0.1 mm wide or 1.0 mm wide. However, indirectly as a result of hydraulic conductivity, even the finest may be significant in changing the effective normal stress and therefore also the shear strength.

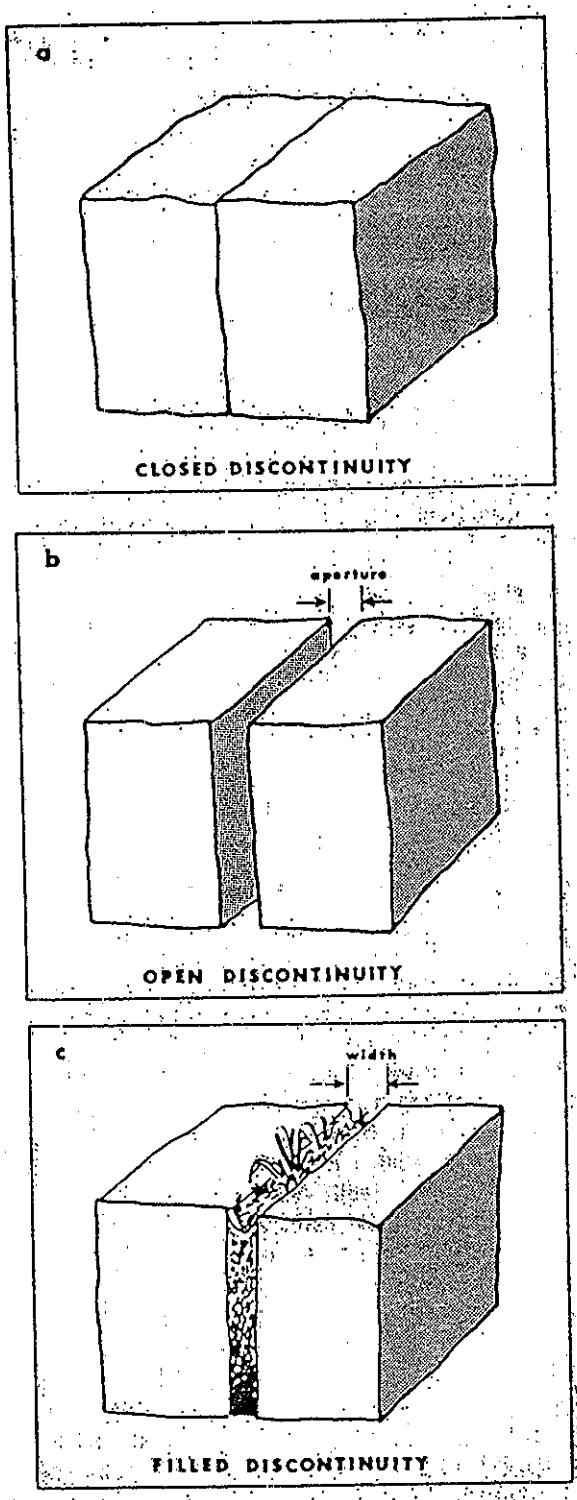


Fig. 21. Diagrams showing the suggested definitions of the *aperture* of open discontinuities and the *width* of filled discontinuities.

(*d*) Unfortunately, visual observation of small apertures is inherently unreliable since, with the possible exceptions of drilled holes and bored tunnels, visible apertures are bound to be disturbed apertures, either

due to disturbance by blasting, or due to surface weathering effects. The influence of apertures is best assessed by water permeability testing. (This is the subject of an individual ISRM document.)

(*e*) Apertures are recorded from the point of view of both their loosening and conducting capacity. Joint water pressure, inflow of water and outflow of storage products (both liquid and gas) will all be affected by aperture.

Equipment

(*a*) Measuring tape of at least 3 m length, calibrated in mm.

(*b*) Feeler gauge (for estimating the width of fine apertures.)

(*c*) White spray paint.

(*d*) Equipment for washing the rock exposure.

Procedure

(*a*) Dirty underground exposures should be washed clean. It is helpful to spray white paint along the desired lines of survey, so that the finest discontinuities are more easily visible. Good lighting is essential.

(*b*) Fine apertures can be measured approximately with feeler gauges, while the larger apertures can be measured with a rule graduated in mm. The apertures of all discontinuities intersecting the survey line will be recorded. Alternatively the variation in aperture of a major discontinuity can be measured along the trace of the discontinuity.

Notes

(*a*) The apertures visible in a rock exposure are inherently disturbed apertures, due either to localized surface weathering or to the mode of excavation. For these reasons measured apertures are likely to be larger than those existing within the rock mass. Tunnels that are machine bored (and borehole walls) should give a much more reliable indication of the undisturbed apertures. Borehole walls can be surveyed by means of periscopes, borehole cameras, and TV equipment, and by means of pressure sensitive packers, as described by Fairhurst and Roegiers [1].

(*b*) The borehole periscope is recommended when the depth from the surface does not exceed 30 metres. Greater depths result in distortion of the optical path which consists of a series of rigid tubes supporting a system of lenses and prisms. A mm calibrated scale, differently coloured from the rock, should be located on the outside of the periscope in such a position that the apparent apertures can be recorded. These readings must be corrected for orientation if the borehole does not intersect the discontinuities approximately at right angles.

(*c*) The core recovery method known as the *integral sampling method* [2] is recommended for obtaining aperture data in special circumstances. The method essentially consists of recovering a core sample which has previously been reinforced with a grouted bar. The reinforcing bar is co-axially overcovered with a larger diameter coring crown.

(*d*) Even undisturbed apertures give a poor indication of their water conducting potential. The wall roughness may reduce the actual conductivity to a fraction of its theoretical smooth-wall equivalent as a result of friction and tortuosity effects. In addition, there is much evidence that flow in joints may be tube-like rather than sheet-like [3]. *In situ* permeability testing will be a much more reliable indicator of the influence of apertures than direct measurement (*Field Permeability* forms the subject of an individual ISRM document).

(*e*) Apertures measured across discontinuities that are displaced by previous shearing (for example in an unstable slope) may vary widely from point to point. The "dead areas" caused by asperity contact and undetected debris will again make aperture measurements rather unreliable as a basis for conductivity estimation [4].

Presentation of results

(*a*) Apertures can be described by means of the following terms:

Aperture	Description	
<0.1 mm	Very tight	Closed features
0.1–0.25 mm	Tight	
0.25–0.5 mm	Partly open	
0.5–2.5 mm	Open	Gapped features
2.5–10 mm	Moderately wide	
>10 mm	Wide	
1–10 cm	Very wide	Open features
10–100 cm	Extremely wide	
>1 m	Cavernous	

(*b*) Modal (most common) apertures should be recorded for each discontinuity set.

(*c*) Individual discontinuities having apertures noticeably wider or larger than the modal value should be carefully described, together with location and orientation data.

(*d*) Photographs of extremely wide (10–100 cm) or cavernous (>1 m) apertures should be appended.

REFERENCES

1. Fairhurst C. & Roegiers J. C. Estimation of rock mass permeability by hydraulic fracturing—a suggestion. Discussion. *Proc. Int. Soc. Rock Mech. Symp.*, Stuttgart. *Percolation Through Fissured Rock*. D2. pp. 1–5 (1972).
2. Rocha M. & Barroso M. Some applications of the new integral sampling method in rock masses. *Proc. Int. Symp. on Rock Mech.* Nancy. *Rock Fracture*. Paper 1-21 (1971).
3. Wolters R., Reinhardt M. & Jäger B. Beobachtungen über Art, Anordnung und Ausdehnung von Kluftöffnungen. *Proc. Int. Soc. Rock Mech. Symp.* Stuttgart. *Percolation Through Fissured Rock*. TI-I. 13 p. (1972).
4. Sharp J. C. & Maini Y. N. T. Fundamental considerations on the hydraulic characteristics of joints in rock. *Proc. Int. Soc. Rock Mech. Symp.* Stuttgart. *Percolation Through Fissured Rock*. TI-F. 15 p. (1972).
5. Cecil O. S. Correlations of rockbolt—shotcrete support and rock quality parameters in Scandinavian tunnels. Ph.D. Thesis, University of Illinois. 414 p. (1970).
6. Schneider T. R. Seelisberg tunnel: Geologie des Bauprojektes. Schweitz. Nationalstrasse. NZ. Zürich (1970).

7 Neustadt L. Genesis und quantitative Characteristik der Klüftigkeit (in Bezug auf den Wasserbau). *Proc. Int. Symp. on Rock Mech.*, Nancy. *Rock Fracture*. Paper I-15 (1971).
8 Morfeldt C. O. Drainage problem in connection with tunnel construction in Precambrian granitic bedrock (in Sweden). *Proc. Int. Soc Rock Mech. Symp.*, Stuttgart. *Percolation Through Fissured Rock*. Paper T4-G. 9 p (1972).
9 Rocha M. Discussion. *Proc. Int. Soc. Rock Mech Symp.* Stuttgart. *Percolation Through Fissured Rock*. D1. pp. 11–15 (1972).
10 Bieniawski Z. T. Geomechanics classification of rock masses and its application in tunneling. *Proc. 3rd. Cong. of Int. Soc. Rock Mech.*, Denver. *Advances in Rock Mechanics*. Vol IIA, pp. 27–32 (1974).
11 Korhonen K-H., Gardemeister R., Jääskeläinen H., Niini H. & Vähäsarja P. Engineering geological rock classification (in Finnish) Geotechnical Laboratory, Report 12, Technical Research Centre of Finland, 78 p (1974).

7. FILLING

Scope

(*a*) Filling is the term for material separating the adjacent rock walls of discontinuities, e.g. calcite, chlorite, clay, silt, fault gouge, breccia etc. The perpendicular distance between the adjacent rock walls is termed the *width* of the filled discontinuity, as opposed to the *aperture* of a gapped or open feature.

(*b*) Due to the enormous variety of occurrences, filled discontinuities display a wide range of physical behaviour, in particular as regards their shear strength, deformability and permeability. Short-term and long-term behaviour may be quite different such that it is easy to be misled by favourable short term conditions.

(*c*) The wide range of physical behaviour depends on many factors of which the following are probably the most important.

(i) Mineralogy of filling material
(ii) Grading or particle size
(iii) Over-consolidation ratio
(iv) Water content and permeability
(v) Previous shear displacement
(vi) Wall roughness
(vii) Width
(viii) Fracturing or crushing of wall rock

(*d*) Every attempt should be made to record the above factors, using quantitative descriptions where possible, together with sketches and/or colour photographs of the most important occurrences. Certain index tests are suggested for a closer investigation of major discontinuities considered to be a threat to stability. In special cases the results of these field descriptions may warrant the recommendation for large scale *in situ* testing, at least in the case of dam foundations or major slopes.

Equipment

(*a*) Measuring tape of at least 3 m length, graduated in mm.

(*b*) Folding straight-edge of at least 2 m in length.

(*c*) Plastic bags for taking samples of the filling material of up to 1 or 2 kg in weight. In some cases undisturbed samples may be required for shear testing. Various soil mechanics tube samplers can be used for this operation.

(*d*) Geological hammer with one tapered point.

(*e*) Strong pen knife or similar.

Procedure

(*a*) *Width*. The minimum and maximum widths of simple filled discontinuities (e.g. clay filled joints) should be measured to the nearest 10%, and an estimate made of the most common (modal) width. Marked differences between the minimum and maximum widths may indicate that shear displacement has occurred if the walls are essentially unaltered or unweathered.

In cases where fillings are thin it may be helpful to try to measure the mean amplitude of wall roughness using the straight edge, and compare this with the mean width of the filling as illustrated in Fig. 22. This will be especially valuable when assessing shear strength and deformation characteristics in detailed studies.

The principal dimensions of complex filled discontinuities (e.g. shear zones, crushed zones, faults, fault zones, dykes and lithological contacts) should be estimated, or measured to the nearest 10% when possible. In the case of important occurrences it is helpful to make field sketches such that the condition of the wall rock (i.e. degree of associated fracturing and/or alteration) is also communicated. See examples in Fig. 23.

(*b*) *Weathering grades*. Filled discontinuities that have originated as a result of preferential weathering along discontinuities may have fillings composed of *decomposed* rock, or *disintegrated* rock. The relevant type should be recorded.

Decomposed:—	The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed.
Disintegrated:—	The rock is weathered to the condition of a soil, in which the original material fabric is still intact. The rock is friable, but the mineral grains are not decomposed.

(*c*) *Mineralogy*. For all types of filled discontinuities the finest fraction of the filling or gouge is of most interest since this usually controls the long term shear strength. The mineralogical composition of the finer filling material should therefore be determined, especially in cases where active clays or swelling clays are suspected. Samples should be taken when in doubt concerning the mineralogy.

In cases where swelling clay such as montmorillonite is identified or suspected, and where this condition might be critical for stability, samples should be taken for free swelling and swelling pressure tests. (It is of advantage to record the *in situ* water content of these samples where possible. Such samples should therefore be sealed.)

(*d*) *Particle size*. The method of describing the grad-

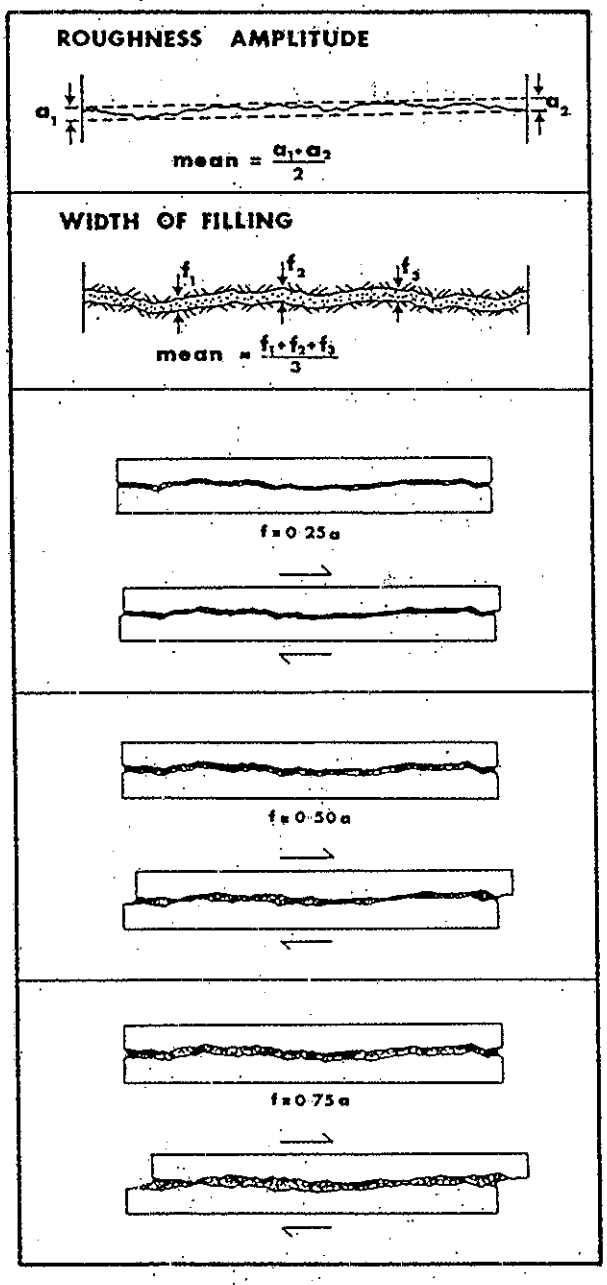


Fig. 22. In the case of simple filled discontinuities, the amplitude of the wall roughness and the thickness of the filling can help to indicate the amount of shear displacement required for rock contact (stiffening) to occur. (Zero volume change assumed during shear).

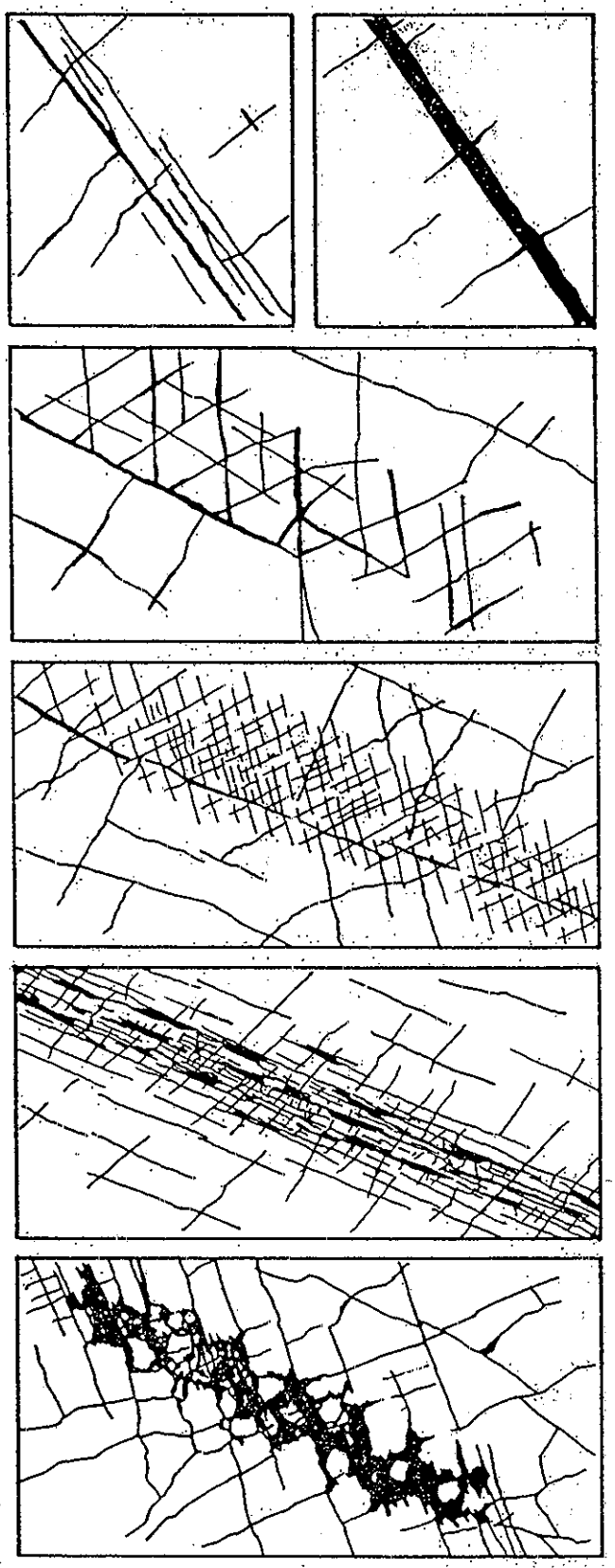


Fig. 23. Examples of field sketches of complex filled discontinuities [1].

ing or particle size will depend on the type of occurrence. A rough quantitative description of the grading of discontinuity fillings can be given by estimating the percentages of clay, silt, sand and rock particles. ($\pm 10\%$). Several kilos of filling material may need to be extracted and fingered before making these estimates.

Particle size can be classified according to the modified Wentworth scale below:

boulders	200–600 mm
cobbles	60–200 mm
coarse gravel	20–60 mm
medium gravel	6–20 mm
fine gravel	2–6 mm
coarse sand	0·6–2 mm
medium sand	0·2–0·6 mm
fine sand	0·06–0·2 mm
silt, clay	<0·06 mm

If a detailed soil mechanics investigation is warranted the finest fraction can be analysed in the laboratory to determine:

clay fraction (% < 2 μ)

% passing No. 200 sieve (74 μ)

Atterberg index tests to determine liquid limit and plasticity index: PI = (LL − PL) %

(*e*) *Filling strength*. Filling material, in particular the finer fraction which is usually weakest, can be assessed by means of the manual index tests tabulated below, as recommended under Wall Strength.

The undrained shear strengths of the soils represented in grades S1 to S6 are equal to one half of the given uniaxial compressive strengths (care should be taken in applying these estimates to fissured clays.)

If a detailed soil mechanics investigation is warranted (e.g. drained shear strength determination) due to the critical nature of an individual filled discontinuity, then undisturbed samples of the filling material may be required. Various tube samplers are available for this sampling operation.

(*f*) *Previous displacement*. Care should be taken to determine whether a given filled discontinuity has suffered previous shear displacement or not. (Slickensides, shears, displaced cross joints, etc.) This should be recorded in conjunction with an estimate of the approximate over-consolidation ratio (OCR) of any clay filling.

(*g*) *Water content and permeability*. The water content and permeability of the filled discontinuity as a whole and of the clay filling in particular should be described as below (see also under Seepage). The decision to make actual measurements of these properties will depend on the importance of the occurrence to the project

W1 The filling materials are heavily consolidated and dry; significant flow appears unlikely due to very low permeability.

W2 The filling materials are damp, but no free water is present.

W3 The filling materials are wet, occasional drops of water.

W4 The filling materials show signs of outwash, continuous flow of water (estimate litres/minute).

W5 The filling materials are washed out locally, considerable water flow along out-wash channels (estimate litres/minute and describe pressure i.e. low, medium, high).

Grade	Description	Field identification	Approx. range of uniaxial compressive strength (MPa)
S1	Very soft clay	Easily penetrated several inches by fist	<0.025
S2	Soft clay	Easily penetrated several inches by thumb	0.025–0.05
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05–0.10
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort.	0.10–0.25
S5	Very stiff clay	Readily indented by thumbnail	0.25–0.50
S6	Hard clay	Indented with difficulty by thumbnail	>0.50
R0	Extremely weak rock	Indented by thumbnail	0.25–1.0
R1	Weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0–5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0–25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25–50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50–100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100–250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250

Note. Grades S1 to S6 apply to cohesive soils, for example, clays, silty clays and combinations of silts and clays with sand, generally slow draining. Some rounding of the strength values has been made when converting to S.I. units.

W6 The filling materials are washed out completely, very high water pressures experienced, especially on first exposure (estimate litres/minute and describe pressure).

Notes

(*a*) The manual index tests for determining grades S1 to S6 can be replaced by more accurate assessments using a standard soil mechanics penetrometer. This contains a stylus which is pressed into the sample at a constant rate. The maximum resistance can be read off a scale which is calibrated to show the maximum compressive strength of the sample. (This value is equal to twice the undrained shear strength $= \frac{1}{2}(\sigma_1 - \sigma_3)$.)

(*b*) Hydrothermal alteration of gouge material and/or the deposition of hydrothermal products will complicate the mineralogical identification of fillings since products not associated with the petrography of the crushed rock or the wall rock may be present.

(*c*) If previous displacement has occurred through the potential weakest layers of a filled discontinuity, i.e. through the clay filling or clay gouge, as evidenced by slickensides and shears, then the over-consolidation ratio (OCR) of the clay will not be important since the discontinuity will be close to residual strength. However, if previous displacement through these weak layers is not suspected then the over-consolidation ratio will be important since the peak drained shear strength of the intact clay may be much higher than the residual strength. Short term stability will be deceptively high, especially in the case of unloading, due to the reduced or negative pore pressures. However, in time swelling and softening may occur due to increased pore pressure and water content and possibly also due to strain softening caused by engineering loading, for example by excavation of an overlying rock slope. This potential for reduction in strength with time should not be underestimated during field assessment.

(*d*) Faults frequently contain highly permeable brecciated gouge adjacent to highly impermeable clay gouge. The water conducting capacity will therefore be strongly anisotropic, and may even be confined to flow parallel to the plane of the fault. It may be premature to describe a fault zone as "dry" or "impermeable" if the tunnel or exploratory adit has not completely penetrated the feature.

Presentation of results

The detail of presentation will be dependent on the importance of the individual filled discontinuity (or set) to the project as a whole. In general the description should be arranged as below, so as to include a description of those factors of particular relevance to the project in hand.

(*a*) Geometry:	width wall roughness field sketch
(*b*) Filling type:	mineralogy particle size weathering grade soil index parameters swelling potential
(*c*) Filling strength:	manual index (S1–S6) shear strength over-consolidation ratio displaced/undisplaced
(*d*) Seepage:	water content (rating as W1–W6) permeability quantitative data

REFERENCES

1. Korhonen K-H., Gardemeister R., Jääskeläinen H., Nini H. & Vähäsarja P. Engineering geological rock classification (in Finnish). Geotechnical laboratory, Report 12. Technical Research Centre of Finland, 78 p (1974).
2. Müller L. *Der Felsbau*. Ferdinand Enke-Verlag, Stuttgart, 624 p (1963).
3. Brekke T. L. & Selmer-Olsen R. Stability problems in underground construction caused by montmorillonite-carrying joints and faults. *Engng. Geol.* 1, 3–19 (1965).
4. Skempton A. W. Some observations on tectonic shear zones. *Proc. 1st. Cong. of Int. Soc. Rock Mech.* Lisbon. Vol. 1, pp. 329–335 (1966).
5. Skempton A. W. & Petley D. J. The strength along structural discontinuities in stiff clays. *Geotech. Conf. on Shear strength Properties of Natural Soils and Rocks*. Oslo 1967. Proc. Vol. 2, pp. 29–46 (1968).
6. Cecil O. S. Correlations of rock bolt-shotcrete support and rock quality parameters in Scandinavian tunnels. Ph.D. Thesis, Univ. of Illinois. 414 p (1970).
7. Selmer-Olsen R. *Ingeniorgeologi* (part 1). Tapir, Trondheim, 230 p (1971).
8. Brekke T. L. & Howard T. R. Stability problems caused by seams and faults. *Proc. 1st. North American Rapid Excavation and Tunneling Conference*, Chicago. Vol. 1, pp. 24–41 (1972).
9. Brekke T. L. & Howard T. R. Functional classification of gouge materials from seams and faults in relation to stability problems in underground openings. Dept. of Civ. Eng. Univ. of California, Berkeley, 153 p (1973).
10. Deere D. U. The foliation shear zone—an adverse engineering geologic feature of metamorphic rocks. *J. Boston Soc. civ. Engrs* 60, 163–176 (1973).
11. Barton N. A review of the shear strength of filled discontinuities in rock. Fjellsprengningsteknikk, Bergmekanikk. Cong. Oslo. Tapir, Trondheim, pp. 19.1–19.38 (1973). (Also NGI Publ. No. 105, Oslo 1974).
12. Barton N., Lien R. & Lunde J. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics* (Springer-Verlag), 6, 189–236 (1974).
13. Cording E. J. & Mahar J. W. The effect of natural geologic discontinuities on behaviour of rock in tunnels. *Proc. 2nd. North American Rapid Excavation and Tunneling Conference*. San Francisco, Vol. 1. Chap. 12, pp. 107–138 (1974).
14. Dearman W. R. The characterization of rock for civil engineering practice in Britain. La Geologie de l'Ingenieur, Société Géologique de Belgique, Liège. pp. 1–75 (1974).
15. International Society for Rock Mechanics. Suggested methods for determining shear strength. ISRM Commission on Standardization of Laboratory and Field Tests (1974). This volume pp. 129–140.
16. Norwegian Rock Mechanics Group. Suggested terminology, definitions and map symbols for rock mechanics and engineering geology (in Norwegian). Tapir, Trondheim (1974).
17. Selmer-Olsen R. & Rokoengen K. About swelling tests and stability of clay zones in hard rock. *Proceedings of 3rd. Cong. of Int. Soc. Rock Mech.*, Denver, *Advances in Rock Mechanics*. Vol. II-B, pp. 1061–1068 (1974).

8. SEEPAGE

Scope

(*a*) Water seepage through rock masses results mainly from flow through water conducting discon-

tinuities ("secondary" permeability). In the case of certain sedimentary rocks the "primary" permeability of the rock material may be significant such that a proportion of the total seepage occurs through the pores. The rate of seepage is roughly proportional to the local hydraulic gradient and to the relevant directional permeability, proportionality being dependent on laminar flow. High velocity flow through open discontinuities may result in increased head losses due to turbulence.

(*b*) The prediction of groundwater levels, likely seepage paths and approximate water pressures may often give advance warning of stability or construction difficulties. The field description of rock masses must inevitably precede any recommendation for field permeability tests so these factors should be carefully assessed at this early stage.

(*c*) Irregular groundwater levels and perched water tables may be encountered in rock masses that are partitioned by persistent impermeable features such as dykes, clay filled discontinuities or permeable beds. The prediction of these potential flow-barriers and associated irregular water tables is of considerable importance, especially for engineering projects where such barriers might be penetrated at depth by tunnelling, resulting in high pressure inflows.

(*d*) Seepage of water caused by drainage into an engineering excavation may have far reaching consequences in cases where a sinking ground water level would cause settlement of structures founded on overlying clay deposits.

(*e*) The approximate description of the local hydrogeology should be supplemented with detailed observations of seepage from individual discontinuities or particular sets, according to their relative importance to stability. A short comment concerning recent precipitation in the area, if known, will be helpful in the interpretation of these observations. Additional data concerning groundwater trends and rainfall and temperature records will be useful supplementary information.

(*f*) In the case of rock slopes, the preliminary design estimates will be based on assumed values of effective normal stress. If, as a result of field observations one has to conclude that pessimistic assumptions of water pressure are justified (i.e. a tension crack full of water with zero exit pressure at the toe of the unfavourable discontinuity) then this will clearly have the greatest consequences for design. So also will the field observation that ice formation is possible or probable. Deterioration of rock slopes and tunnel portals through ice wedging and/or increased water pressure caused by iceblocked drainage paths are serious seasonal problems in many countries.

Equipment

(*a*) Visual observation (in the case of tunnels good lighting is essential).

(*b*) Air photographs, rainfall and temperature records as appropriate and depending upon availability.

Procedure

(*a*) Available air photographs should be studied to obtain an overall view of the local drainage pattern and likely groundwater levels. (Groundwater may be indicated by growth of vegetation along faults and basic dykes.) Information on seasonal variations of groundwater levels, and on rainfall and temperature records should be obtained where possible.

(*b*) Description of the local hydrogeology will usually be limited in the preliminary stages of field mapping. There will probably be no boreholes for pumping tests, no wells for water level determination and drawdown tests, no tracer tests, and no piezometer installations. The hydrogeology will therefore have to be assessed from geological predictions of the likely locations of acquifers, from predictions of the likely orientation and location of impermeable flow barriers, and from predictions of the likely resultant seepage directions and ground water levels. The need for exploratory boreholes for water level determination, tracer testing, piezometer installation and pumping or drawdown tests should be assessed, and their optimum location indicated on appropriate plans.

(*c*) The mutual interaction of the planned engineering project and the assumed groundwater flow regime should be assessed and important consequences summarized. The effect of seepage towards or into a planned excavation such as a tunnel or slope should be described with a view to preliminary analysis. The predicted effect of any resultant drawdown of groundwater levels on existing installations, and on the settlement of clay foundations should be summarized.

(*d*) Seepage from individual *unfilled* and *filled discontinuities* or from specific sets exposed in a tunnel or in a surface exposure, can be assessed according to the following descriptive scheme:

Unfilled discontinuities

Seepage rating	Description
I	The discontinuity is very tight and dry, water flow along it does not appear possible
II	The discontinuity is dry with no evidence of water flow.
III	The discontinuity is dry but shows evidence of water flow, i.e. rust staining, etc.
IV	The discontinuity is damp but no free water is present
V	The discontinuity shows seepage, occasional drops of water, but no continuous flow.
VI	The discontinuity shows a continuous flow of water. (Estimate l/min and describe pressure i.e. low, medium, high)

Filled discontinuities

Seepage rating	Description
I	The filling materials are heavily consoli-

dated and dry, significant flow appears unlikely due to very low permeability.

II The filling materials are damp, but no free water is present.

III The filling materials are wet, occasional drops of water.

IV The filling materials show signs of outwash, continuous flow of water (estimate l/min).

V The filling materials are washed out locally, considerable water flow along out-wash channels (estimate l/min and describe pressure i.e. low, medium, high).

VI The filling materials are washed out completely, very high water pressures experienced, especially on first exposure (estimate l/min and describe pressure).

(*e*) In the case of a rock engineering construction which acts as a drain for the rock mass, for example a tunnel, it is helpful if the overall flow into individual sections of the structure are described. This should ideally be performed immediately after excavation since groundwater levels, or the rock mass storage, may be depleted rapidly. Descriptions may be based on the following scheme:

Rock mass (e.g. tunnel wall)

Seepage rating	Description
I	Dry walls and roof, no detectable seepage.
II	Minor seepage, specify dripping discontinuities
III	Medium inflow, specify discontinuities with continuous flow (estimate l/min/10 m. length of excavation).
IV	Major inflow, specify discontinuities with strong flows (estimate l/min/10 m. length of excavation).
V	Exceptionally high inflow, specify source of exceptional flows (estimate l/min/10 m. length of excavation).

(*f*) A field assessment of the likely effectiveness of surface drains, inclined drill holes, or drainage galleries should be made in the case of major rock slopes. This assessment will depend on the orientation, spacing and apertures of the relevant discontinuities.

(*g*) The potential influence of frost and ice on the seepage paths through the rock mass should be assessed. Observations of seepage from the surface trace of discontinuities may be misleading in freezing temperatures. The possibility of iceblocked drainage paths should be assessed from the point of view of surface deterioration of a rock excavation, and from the point of view of overall stability.

Notes

(*a*) *Local* rainfall records should be obtained where possible, to help in the interpretation of seepage observations. This is especially important in the case of observation of surface outcrops, slopes, and tunnels at shallow depth.

(*b*) In the case of open pit mines, boreholes are drilled for mineral exploration and rock mechanics is commonly entertained only at a subsequent stage, if mineral evaluation is encouraging. The pre-existence of boreholes will allow a comprehensive hydrogeological study to be performed, including tracer tests, piezometer installation, falling-head and pumping tests. Borehole walls can be surveyed for seepage horizons by means of periscopes, borehole cameras and T.V. equipment.

(*c*) Testing performed in drill holes (e.g. falling head and Lugeon tests) for estimating rock mass permeability forms the subject of a separate ISRM "suggested method". The description of any available lugeon values is obviously an important supplement to the present suggested methods for description of rock masses and discontinuities. (See also Drill Core.)

(*d*) Bedding joints and beds of sedimentary rocks having high "primary" permeability tend to be persistent features with the potential for hydraulically connecting large areas of sedimentary rock masses. Such efficient hydraulic connection will be inherently less marked in igneous and metamorphic environments i major regional joints and faults are absent.

(*e*) Faults sometimes contain highly permeable brec cia adjacent to highly impermeable clay gouge. Th hydraulic conductivity may therefore be strongly aniso tropic, and may even be confined to flow parallel t the plane of the fault. It may be premature to describ a fault zone as dry if a tunnel or exploratory adit ha not completely penetrated the feature.

(*f*) The highest location of seeping joints on a rocl slope may be important indirect input for a preliminar stability analysis. Likewise the depth of a tunnel o its location relative to major weakness zones will b important, since this may imply potentially seriou inflows.

Presentation of results

(*a*) Air photos, geological maps, or plans of suitabl scale should be marked with arrows to indicate th general groundwater flow pattern that has been inte preted as a result of available hydrogeological dat If appropriate, rainfall and temperature records ca be appended.

(*b*) Anticipated impermeable flow barriers such a dykes, major clay-filled discontinuities and impe meable beds, should be drawn on simplified geologic maps and vertical cross-sections, together with antic pated groundwater levels. Optimum locations for inve tigatory boreholes (and any existing boreholes), shoul be indicated as appropriate.

(*c*) The anticipated mutual interaction of the planne engineering project and the assumed groundwater flo regime should be described where possible. If sufficie data are available for reliable predictions, anticipat pre-construction and post-construction phreatic su faces should be sketched. The likely effect of extrer

weather conditions should be indicated if possible. Possible effects of frost and of artificial drainage measures should be appended.

(*d*) Local seepage observations for individual discontinuities, for specific sets, or for the rock mass as a whole can be presented as seepage ratings I–VI. If enough observations are available, sketches showing the distributions of ratings can be contoured, drawn as histograms, or, in the case of tunnels, presented on longitudinal sections in parallel with structural data, in the same way that Lugeon values are presented parallel with borehole geology.

REFERENCES

1. Casagrande A. Control of seepage through foundations and abutments of dams. *Geotechnique*, 11, 159–182 (1961).
2. Terzaghi K. Stability of steep slopes on hard unweathered rock. *Geotechnique* 12, 251–270 (1962).
3. Müller L. The rock slide in the Vajont Valley. *Felsmechanik und Ing Geol.* 2, 148–212 (1964).
4. Londe P. La stabilité des massifs rocheux: application aux barrages. *Annls: Inst. tech. Bâtiment Trav. publ.* pp. 1617–1637 (1968).
5. Snow D. T. Rock fracture spacing, opening, and porosities. ASCE Proc. Vol. 94 No. SM1, pp. 73–91 (1968).
8. Sharp J. G. Drainage characteristics of sub-surface galleries. *Proc. of 2nd. Cong. of Int. Soc. Rock Mech.* Belgrade. Vol. 3, Paper 6-10 (1970).
7. Sabarly F., Pautre A. & Londe P. Quelques réflexions sur la drainabilité des massifs rocheux. *Proc. of 2nd. Cong. of Int. Soc. Rock Mech.*, Belgrade, Vol 3, Paper 6-12 (1970).
8. Cecil O. S. Correlations of rock bolt-shotcrete support and rock quality parameters in Scandinavian tunnels. Ph.D. Thesis. Univ. of Illinois, 414 p (1970).
9. Moreseldt C. O. Significance of groundwater at rock constructions of different types. *Proc. Int. Symp. on Large Permanent Underground Openings*: Oslo, 1969, pp. 305–317 (1969).
10. Sharp J. C., Maini Y. N. T. & Harper T. R. Influence of groundwater on the stability of rock masses: 1-hydraulics within rock masses. *Inst. of Min. and Metall. Trans.* Vol. 81, Sect. A pp. A13–A20 (1972).
11. Wolters P. Reinhardt M. & Jäger B. Beobächtungen über Art, Anordnung und Ausdehnung von Kluftöfnungen. *Proc. of Int. Soc. Rock Mech. Symp.* Stuttgart: *Percolation Through Fissured Rock*. T1-1, 13 p (1972).
12. Morfeldt C. O. Drainage problem in connection with tunnel construction in Precambrian granitic bedrock (in Sweden). *Proc. of Int. Soc. Rock Mech. Symp.*, Stuttgart. *Percolation Through Fissured Rock*. T4-G, 9 p (1972).
13. Louis C. Reconnaissance des massifs rocheux par sondages et classifications géotechniques de roches. Sols et Fondations. No. 319, July–August, pp. 97–122 (1974).
14. Brawner C. O. Rock mechanics in open pit mining. *Proc. 3rd. Cong. of Int. Soc. Rock Mech.* Denver. Vol. 1A pp. 755–773 (1974).
15. Hoek E. & Bray J. *Rock Slope Engineering*. The Institution of Mining and Metallurgy, London. 309 p (1974).
16. Harper T. R. The transient groundwater pressure response to rainfall and the prediction of rock slope instability. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* Vol. 12, pp. 175–179 (1975).

9. NUMBER OF SETS

Scope

(*a*) Both the mechanical behaviour and the appearance of a rock mass will be dominated by the number of sets of discontinuities that intersect one another. The mechanicaal behaviour is especially affected since the number of sets determines the extent to which the rock mass can deform without involving failure of the intact rock. The appearance of the rock mass is affected since the number of sets determines the degree of overbreak that tends to occur with excavation by blasting. (See Fig. 24.)

(*b*) The number of sets of discontinuities may be a dominant feature of rock slope stability, though traditionally the orientation of discontinuities relative to the face is considered of primary importance. However, if insufficient sets exist the probability of instability may be reduced almost to zero. On the other hand a large number of sets having close spacing may change the potential mode of slope failure from translational or toppling to rotational/circular.

(*c*) In the case of tunnel stability three or more sets will generally constitute a three-dimensional block structure having considerably more "degrees of freedom" for deformation than a rock mass with less than three sets. For example a strongly foliated phyllite with just one closely spaced joint set may give equally good tunnelling conditions as a massive granite with three widely spaced joint sets. The amount of overbreak in a tunnel will usually be strongly dependent on the number of sets.

Equipment

(*a*) Geological compass and clinometer.

(*b*) Visual recognition and/or photographic recording.

Procedure

(*a*) The number of sets will often be a function of the size of area mapped. In a preliminary investigation it is important to record all sets present. The recognition of individual sets will usually proceed simultaneously with the *orientation* measurements. Up to 150 joints may need to be measured, and the number of sets can usually be determined by contouring joint poles plotted on polar equal area nets (see Orientation).

(*b*) If orientations are consistent, careful sampling may reduce the number of joints that have to be measured to define the number of sets.

(*c*) In the detailed stages of field investigations, the number of sets present *locally* should be recorded as a supplement to procedure (*a*). The stability of a given section of tunnel or rock slope, or the deformability of a given foundation will be a function of the relevant number of sets found *locally*, rather than of the total number mapped under procedure (*a*).

(*d*) Visual recognition of the number of sets should be accompanied by some system of numbering for identification purposes. For example the most systematic and persistent set can be labelled "set No. 1" and so on. (See Fig. 24). Alternatively sets can be numbered in the order of their importance to stability.

Notes

(*a*) Systematic joint sets should be distinguished from non-systematic joints when recording the number of sets. In general systematic joints will be persistent features with individual joints parallel or sub-parallel in plan, while non-systematic joints display random rather

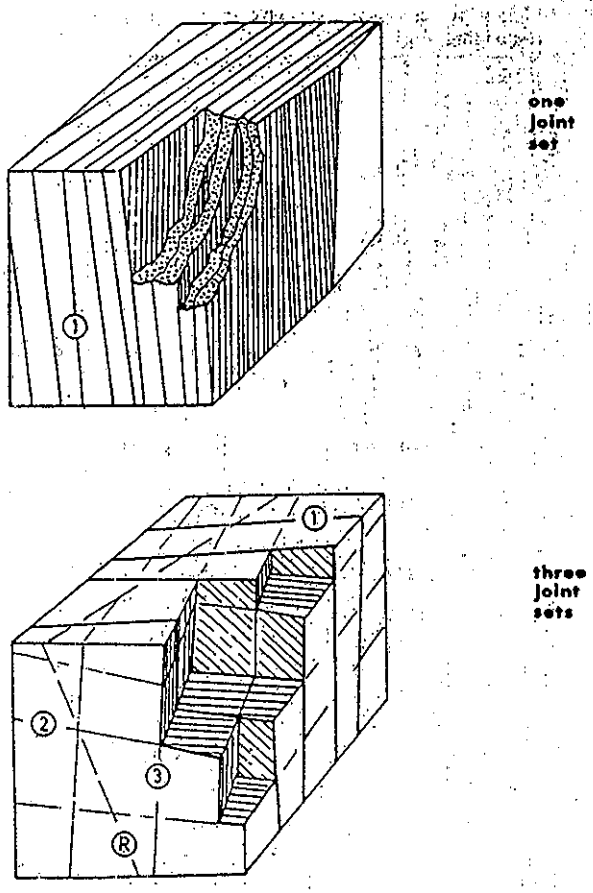


Fig. 24. Examples that demonstrate the effect of the number of joint sets on the mechanical behaviour and appearance of a rock mass.

than oriented patterns in plan and section. Problems of set identification when sets cannot readily be distinguished in the field may be reduced by utilizing statistical tests for identifying trends in the distribution of poles plotted on polar equal area nets (See Fig. 5, under Orientation.)

(*b*) Incipient discontinuities such as those that may develop parallel to bedding, or parallel to foliation or cleavage, should be included in the *local* estimate of the number of sets, if it is considered that the method of excavation employed will sufficently disturb the rock mass to cause development of these features into equivalent bedding joints, foliation joints, etc.

(*c*) As noted under procedures (*a*) and (*c*), the number of sets recorded will tend to be a function of the size of area mapped, and should be interpreted accordingly. The *spacing* of individual sets will play an important role in this interpretation. For example, four sets recognised following a "conventional" survey of an area (using the pole contouring method) may include some sets with such wide spacing that these would be of little relevance to the stability of a short length of tunnel, though possibly of considerable importance to the stability of a major slope.

Presentation of results

(*c*) The number of joint sets present can be represented visually as part of the presentation of *orientation* data. (See Fig. 2: block diagram; Fig. 3: joint rosettes; Fig. 5: Schmidt pole contour diagram.)

(*b*) The number of joint sets occurring *locally* (for example along the length of a tunnel) can be described according to the following scheme:

I	massive, occasional random joints
II	one joint set
III	one joint set plus random
IV	two joint sets
V	two joint sets plus random
VI	three joint sets
VII	three joint sets plus random
VIII	four or more joint sets
IX	crushed rock, earth-like

Major individual discontinuities should be recorded on an individual basis.

REFERENCES

1. Hodgson R. A. Regional study of jointing in the Comb Ridge Navajo Mountain area Arizona and Utah. *Bull. Am. Ass. Petrol Geol.* 45, 1–38 (1961).
2. John K. W. An approach to rock mechanics. *J. Soil Mech. Fdns Div. cir. Engrs* SM4, pp. 1–30 (1962).
3. Müller L. *Der Felsbau.* Ferdinand Enke-Verlag, Stuttgart, 624 p (1963).
4. Price N. J. *Fault and Joint Development in Brittle and Semi-Brittle Rock.* Pergamon, Oxford. 176 p (1966).
5. Nickelsen R. P. & Hough V. N. D. Jointing in the Appalachian Plateau of Pennsylvania. *Bull. Geol. Soc. Am.* 78, 609–630 (1967).
6. Cecil O. S. Correlations of rock bolt-shotcrete support and rock quality parameters in Scandinavian tunnels. Ph.D. Thesis, Univ. of Illinois. (also Swedish Geotechnical Institute, Procedings No. 27, Stockholm 1975, 275 p.) (1970).
7. Piteau D. R. Geological factors significant to the stability of slopes cut in rock. *Symp. on Planning Open Pit Mines,* Johannesburg, 1970. Balkema, Amsterdam, 1971, pp. 33–53 (1971).
8. Robertson A. MacG. The interpretation of geological factors for use in slope theory. *Symp. on Planning Open Pit Mines,* Johannesburg, 1970. Balkema, Amsterdam, 1971, pp. 55–71 (1971).
9. Babcock E. A. Regional jointing in Southern Alberta. *Canadian J. Earth Sci.* 10, 1769–1781 (1973).
10. Barton N., Lien R. & Lunde J. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics,* 6, 189–236 (1974).
11. Hoek E. & Bray J. *Rock Slope Engineering.* The Institution of Mining and Metallurgy, London, 309 p (1974).
12. Goodman R. E. *Methods of Geological Engineering in Discontinuous Rocks.* West Publishing, N.Y. 472 p (1976).

10. BLOCK SIZE

Scope

(*a*) *Block size* is an extremely important indicator of rock mass behaviour. Block dimensions are determined by discontinuity *spacing*, by the *number of sets*, and by the *persistence* of the discontinuities, delineating potential blocks.

(*b*) The *number of sets* and the *orientation* determine the shape of the resulting blocks, which can take the approximate form of cubes, rhombohedrons, tetrahedrons, sheets, etc. However, regular geometric shapes are the exception rather than the rule since the joints in any one set are seldom consistently parallel. Jointing

in sedimentary rocks usually produces the most regular block shapes.

(*c*) The combined properties of *block size* and inter-block shear strength determine the mechanical behaviour of the rock mass under given stress conditions. Rock masses composed of large blocks tend to be less deformable, and in the case of underground construction, develop favourable arching and interlocking. In the case of slopes, a small *block size* may cause the potential mode of failure to resemble that of soil, (i.e. circular/rotational) instead of the translational or toppling modes of failure usually associated with discontinuous rock masses. In exceptional cases "block" size may be so small that flow occurs, as with a "sugar-cube" shear zone in quartzite.

(*d*) Rock quarrying and blasting efficiency are likely to be largely a function of the natural *in situ block-size*. It may be helpful to think in terms of a block size distribution for the rock mass, in much the same way that soils are categorized by a distribution of particle sizes.

(*e*) Block size can be described either by means of the average dimension of typical blocks (block size index I_b), or by the total number of joints intersecting a unit volume of the rock mass (volumetric joint count J_v)

Equipment

(*a*) Measuring tape of at least 3 m length, calibrated in mm divisions.

Procedure

(*a*) *Block size index* (I_b) The index can be estimated by selecting by eye several typical block sizes and taking their average dimensions. Since the index may range from millimetres to several metres, a measuring accuracy of 10% should be sufficient.

Each domain should be characterized by a modal I_b, together with the range, i.e. typical largest and smallest block size indices.

The number of sets should always be recorded in parallel with I_b since if there are only one or two sets, any subsequent attempt to convert I_b to typical block *volumes* may be unrealistic.

(*b*) *Volumetric joint count* (J_v). The volumetric joint count is defined as the sum of the number of joints *per metre* for each joint set present. Random discontinuities can be included, but will generally have little effect on the results.

The number of joints of each set should be counted along the relevant joint set perpendicular. A sampling length of 5 or 10 m is suggested. Each joint count will then be divided by 5 or 10 to express the results as number of joints per metre.

A typical result for three joint sets and a random discontinuity counted along 5 or 10 m perpendicular sampling lines might appear as below:

$$J_v = 6/10 + 24/10 + 5/5 + 1/10$$

$$J_v = 0.6 + 2.4 + 1.0 + 0.1 = 4.1/\text{m}^3 \text{ (medium-size blocks)}$$

The following descriptive terms give an impression of the corresponding block size:

Description	J_v (joints/m^3)
Very large blocks	<1.0
Large blocks	1–3
Medium-sized blocks	3–10
Small blocks	10–30
Very small blocks	>30

Values of $J_v > 60$ would represent crushed rock, typical of a clay-free crushed zone.

(*c*) *Rock masses.* Rock masses can be described by the following adjectives, to give an impression of block size and shape:

(i) *massive* = few joints or very wide spacing
(ii) *blocky* = approximately equidimensional
(iii) *tabular* = one dimension considerably smaller than the other two
(iv) *columnar* = one dimension considerably larger than the other two
(v) *irregular* = wide variations of block size and shape
(vi) *crushed* = heavily jointed to "sugar cube"

See Fig. 25 for examples of the above.

Notes

(*a*) *Block size index* (I_b). The purpose of the block size index is to represent the average dimensions of typical rock blocks. The average value of individual modal spacings (S_1, S_2, etc., see Spacing) may not give a realistic value of I_b if there are more than three sets, since the fourth set, if widely spaced, will artificially increase I_b, but may have little influence on actual block sizes as observed in the field.

In the case of sedimentary rocks, two mutually perpendicular sets of cross joints plus bedding constitute an extremely common cubic or prismatic block shape. In such cases I_b is correctly described by:

$$I_b = \frac{S_1 + S_2 + S_3}{3}$$

(*b*) *Volumetric joint count* (J_v). Field mapping can be performed very rapidly as a measuring tape can be dispensed with when individual joint spacings are not of interest. 5 or 10 m can be paced out or estimated with reasonable accuracy by most observers (i.e. to within ±10% of the correct length). The observer should face in the direction of strike for each joint set that is to be counted and count perpendicular to the strike, thereby removing the angular correction factor.

It should be noted that

$$J_v \text{ is not equal to } \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_n}$$

The calculation of J_v is based on the *mean* spacings, not *modal* spacings. Generally the results will be similar, but spacing tends to be log-normally distributed.

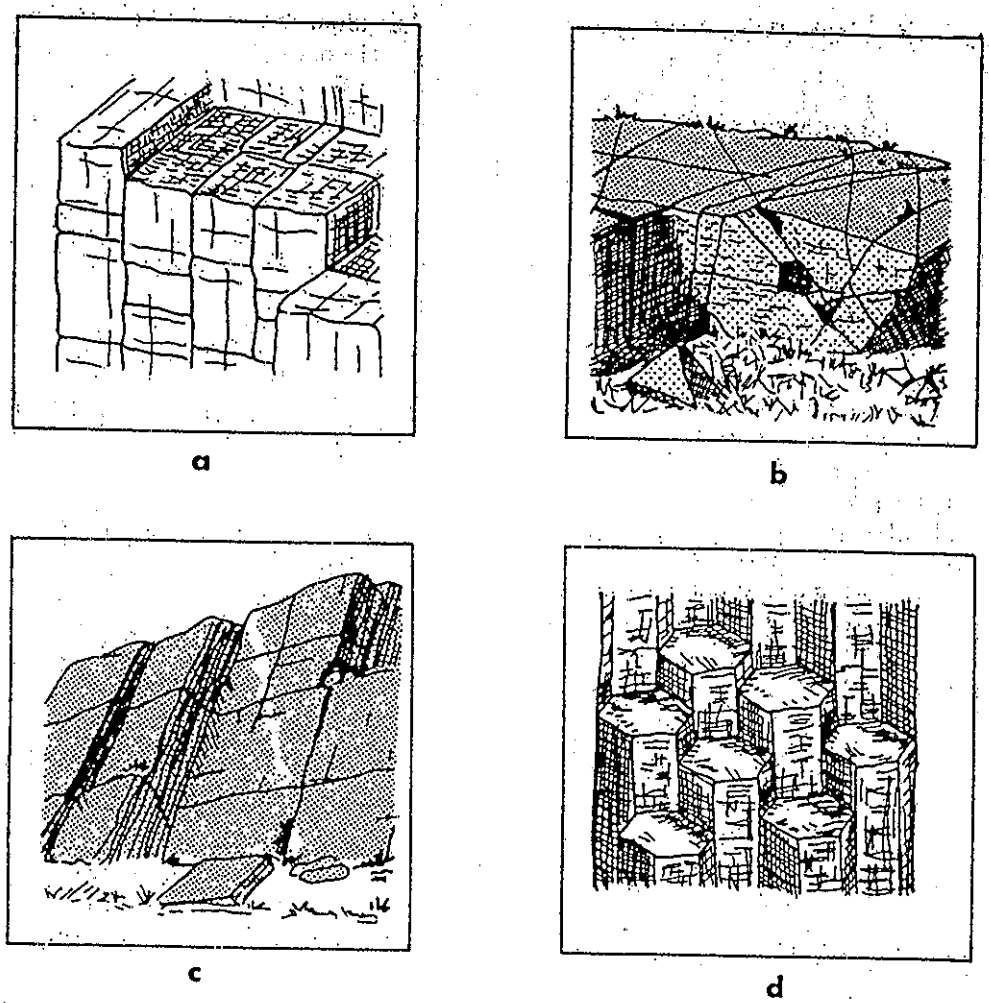


Fig 25. Sketches of rock masses illustrating (a) *blocky* (b) *irregular* (c) *tabular* and (d) *columnar* block shapes.

The occasional random discontinuities will not noticeably affect the value of J_t unless the spacing of the systematic joints is wide or very wide (i.e. 1–10 m). In such cases they should be included with appropriately wide spacing, for example 10 m.

In view of the widespread use of RQD in various rock mass classification methods it is of value to present an approximate correlation between J_r and RQD

$$RQD = 115 - 3.3\,J_r \text{ (approx.)}$$
$$(RQD = 100 \text{ for } J_r < 4.5)$$

This relationship can be used for estimating the order of magnitude of RQD when borecore is unavailable.

(*e*) *Orientation data.* *Orientation* data will provide additional descriptive data for a clearer expression of the form of an anisotropic block structure if present, i.e. "*steeply dipping* sheets, slabs, beds" etc. or "*vertical* columnar blocks" etc. When block dimensions are reasonably isotropic only the block shape need be described, i.e. cubic, rhombohedral, prismatic, tetrahedral, irregular, etc. as appropriate.

Presentation of results

(*a*) Record the modal *block size index* (I_b) and I_b values typical for the largest and smallest block sizes for the domain or domains of interest. (Also record the *number of sets* and describe the *persistence*).

(*b*) Record the *volumetric joint count* (J_t) for the domain or domains of interest. (Also record the *number* of sets and describe the *persistence*).

(*c*) Describe the rock mass and its "blockiness" in general terms as: massive, blocky, tabular, columnar, crushed or as appropriate.

Where possible, block size and shape should also be communicated by means of photographs and/or field sketches of typical exposures (see Fig. 25).

REFERENCES

1. John K. W. An approach to rock mechanics. *J. Soil Mech. Fdns. Div. Am. Soc. civ. Engrs.* SM4, pp. 1–30 (1962).
2. Müller L. *Der Felsbau.* Ferdinand Enke-Verlag, Stuttgart. 624 p. (1963).
3. Price N. J. *Fault and Joint Development in Brittle and Semi-brittle Rock.* Pergamon, Oxford. 176 p. (1966).
4. Piteau D. R. Characterizing and extrapolating rock joint properties in engineering practice. *Rock Mechanics* (Springer-Verlag) Suppl. 2, pp. 5–31 (1973).
5. Franklin J. A. Rock quality in relation to the quarrying and performance of rock construction materials. *Proc. of 2nd. Int. Cong. of the Int. Ass. of Eng. Geology.* São Paulo, Brazil. Vol. 1, IV-PC-2. 11 p. (1974).

6 Korhonen K-H. Gardemeister R. Jääskeläinen H. Niini H. & Vähäsarja P. Engineering geological rock classification (in Finnish). Geotechnical Laboratory. Report 12. Technical Research Centre of Finland. 78 p (1974)
7 Palmstrom A. Characterizing the degree of jointing and rock mass quality (in Norwegian) (Internal Report. Ing A.B. Berdal. Mariesvei 20. 1322 Hovik. Oslo. 26 p (1975)
8 Barton N. Unsupported underground openings. *Proc Rock Mechanics Meeting* BeFo. Stockholm. pp. 61-94 (1976)

11. DRILL CORE

Scope

(*a*) Drill core description is here intended primarily to provide information on the discontinuities.

(*b*) In the preliminary stages of field mapping, drill core is unlikely to be available. However, the need for drilling and the optimum locations and orientations of holes should be described, based on existing information concerning the likely orientation of discontinuities.

(*c*) If drill core is available it can first be described by means of the following parameters: *total core recovery* (R), discontinuity *frequency* (F), and *rock quality designation* (RQD). However, these parameters alone do not usually provide sufficient information for design purposes.

(*d*) Drill cores (and drill holes) represent line samples of the rock mass. Structural features such as discontinuity *orientation*, *spacing* and the *number of sets* cannot normally be adequately sampled by one hole without prior knowledge of the orientation and the number of sets.

(*e*) Carefully planned and executed core drilling followed by detailed core description and hole inspection can provide approximate information about many of the ten specific rock mass parameters described under the preceding "suggested methods" i.e. 1. *Orientation*, 2. *Spacing*; 3. *Persistence*, 4. *Roughness* 5. *Wall strength*, 6. *Aperture*, 7 *Filling*, 8. *Seepage*, 9. *Number of sets*, 10 *Block size*.

Equipment

(*a*) Measuring tape of at least 3 m length, calibrated in mm divisions. Protractor or similar scale for measuring the angles between the core axis and the discontinuities.

(*b*) Materials for washing the core.

(*d*) Subsequent measurements in the drill holes may require the use of at least one of the following: borehole periscope, camera, TV camera, water level indicator (electrical contact type), together with the associated cables and winding gear appropriate for the length of hole and the equipment selected.

Procedure

(*a*) Dirty rock core should in general be washed clean prior to making observations. However, this procedure should be avoided in the case of filled discontinuities and argillaceous rocks likely to be sensitive to wetting and drying.

(*b*) Before making detailed observations the core as a whole should be examined to determine the structural boundaries (domains) and geological features to be measured. The markers indicating depths of geological horizons and the start and end of each run should be carefully checked for errors.

(*c*) *Total core recovery* (R) defined as the summed length of all pieces of recovered core expressed as a percentage of length drilled should be measured and recorded to the nearest 2% if possible. When the core is highly fragmented the length of such portions is estimated by assembling the fragments and estimating the length of core that the fragments appear to represent.

Core recovery is normally used to describe individual core runs or whole boreholes, and not specific structurally defined rock units. The results obtained in a rock mass of poor quality will be strongly dependent on the drilling equipment and on the skill of the drilling crew. Core grinding may result in excessive lost core. Core that is damaged in this way should always be recorded.

Total core recovery (R) is in the first instance usually obtained directly from the drillers log, and is therefore based on individual lengths of uptake. These unit lengths will vary with the rate of drilling and the quantities of the rock drilled through.

Instructions should always be given to the drilling crew so that the depth drilled at the start and end of zones of core loss are carefully recorded. The relevant lengths lost can then be replaced by wooden blocks with markings on both ends.

(*d*) *Frequency* (F) defined as the number of natural discontinuities intersecting a unit length of recovered core should be counted for each metre of core.

Since the orientation of the discontinuities is not considered at this stage, it is clear that differently orientated holes will usually produce different results.

Artificial fractures resulting from rough handling or from the drilling process should be discounted only when they can be clearly distinguished from natural discontinuities.

(*e*) *Rock quality designation* (RQD) is a modified core recovery percentage in which all the pieces of sound core over 10 cm long are counted as recovery, and are expressed as a percentage of the length drilled. The smaller pieces resulting from closer jointing, faulting, or weathering are discounted.

If the core is broken by handling or by the drilling process (i.e. if the fractures are fresh breaks rather than natural surfaces) the fresh broken pieces should be fitted together and counted as one piece, provided they form the requisite length of 10 cm.

Material that is obviously weaker than the surrounding rock such as over-consolidated gouge is discounted, even if it appears as intact pieces that are 10 cm or more in length. (This type of material will normally only be recovered when using the most advanced drilling equipment and experienced or carefully supervised drilling crews.)

The length of individual core pieces should be assessed along the centre line of the core, so that discontinuities that happen to parallel the drill hole will

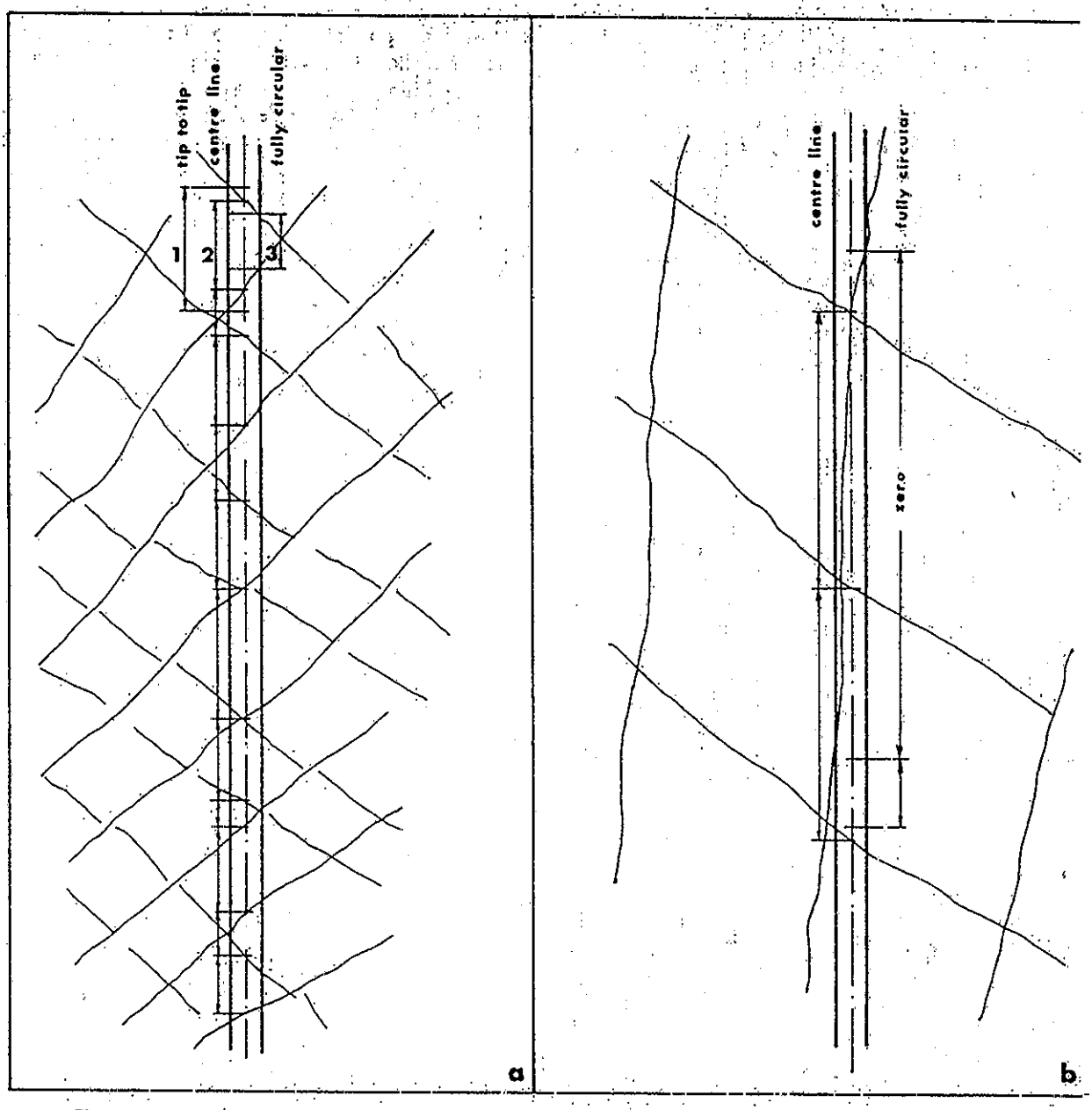


Fig. 26. Examples of three possible interpretations of the length of core pieces. The centre line length is suggested as the most realistic measurement and is recommended

not unduly penalize the RQD values of an otherwise massive rock mass. (See Fig. 26).

It is suggested that RQD values are determined for variable rather than fixed lengths of core run. Values of individual beds, structural domains, weakness zones etc should therefore be logged separately, so as to indicate any inherent variability, and provide a more accurate picture of the location and width of zones with low or zero RQD values.

Supplementary data

Subsequent to the general procedure for logging *total core recovery* (R), *frequency* (F), and *rock quality designation* (RQD), the following supplementary procedures are suggested for determining as much quantitative data as possible concerning the ten parameters:

1. Orientation
2. Spacing
3. Persistence
4. Roughness
5. Wall strength
6. Aperture
7. Filling
8. Seepage
9. Number of sets
10. Block size

A combination of core logging, drill hole viewin (borehole periscope, TV camera) and/or water injectio tests are suggested for assessing those parameters tha

be more or less disturbed in the recovered core, for example *aperture, filling, seepage.*

Orientation

Efforts should be made to log the apparent orientation of discontinuities intersecting the core, using a protractor to measure the acute angles of intersection (θ) relative to the core axis ($\pm 5°$). If the relevant hole is vertical, the angles ($90-\theta$) will represent the true dip of the discontinuities, but without orientated core the *dip direction* will remain unknown.

If two or more non-parallel drillholes have been drilled in a rock mass where there are recognisable markers such as bedding or foliation, the dip direction and dip of these features can be deduced using graphical techniques [1].

If existing surface mapping has already indicated the approximate orientation of certain joint sets, then carefully orientated drill holes can be used to check the orientation of these features at depth. In the case of anticipated vertical and horizontal jointing it is helpful to drill steeply inclined holes (i.e. 60°) in preference to 45°, so that the differently orientated sets can be recognised during core logging by their different core intersection angles.

The true orientation of discontinuities (dip direction and dip) can be obtained from a single drill core if orientation devices are employed during the drilling process. Several methods are available:

(*a*) Orientation of the core based on the measured orientation in each run (Craelius method). This method works well if adjacent pieces of core can be matched. Zones of core loss and perpendicularly intersected discontinuities reduce the effectiveness of the method locally.

(*b*) Orientation of the core by means of a hardened steel groove scriber and compass photo device (Christensen-Huegel method).

(*c*) *Integral sampling method* in which the cores that are recovered have previously been reinforced with a grouted bar whose azimuth is known from positioning rods. The reinforcing bar is co-axially overcored with a larger diameter coring crown.

The orientation of discontinuities (dip direction and dip) can be obtained by drill hole inspection using special television cameras, and periscopes. TV cameras can be orientated such that a discontinuity plane shows as a straight line on the CRT screen. The dip direction and dip can be readily determined. Cameras have been used to depths of 400 m, though generally 150 metres is seldom exceeded due in part to water pressure problems. Minimum hole size for the cameras is generally 76 mm.

The borehole periscope can be used in smaller holes, but due to distortion of the optical path the depth is usually limited to about 30 m.

2. *Spacing*

In rock with marked foliation or bedding features it should be possible to match the individual core pieces such that the actual spacing of obliquely intersected foliation joints, bedding joints or other regular intersecting joint sets can be estimated. The *spacing* (S) will depend on the length (L) measured along the core axis between adjacent natural discontinuities of one set, and the acute angle (θ) that these features subtend with the core axis. Thus:

$$S = L \sin\theta$$

The angles (θ) between the core axis and the individual joints of a given set will be inherently less reliable than those recorded from observations of rock exposures due to the possibility of joint undulation and roughness.

When a joint set is intersected perpendicularly by the drill hole, spacing can obviously be measured directly since (S) is equal to (L).

When the rock has no consistent or clear marker features such as foliation or bedding, the estimation of spacing for any given set of joints will depend on the degree to which the core pieces can be matched. Zones of core loss will clearly frustrate this objective. However, if the joints that intersect the core have markedly different core intersection angles (θ) and/or markedly different surface features (i.e. mineral coatings, roughness) it may be possible to estimate the relevant spacings in a sufficient number of places along the core to make the exercise worthwhile.

Borehole viewing devices that can be orientated (periscope, TV camera) will clearly increase the reliability of spacing measurements.

3. *Persistence*

Unless holes are drilled in a very closely spaced pattern, as may be the case for operations such as grout curtain injection, it will usually not be possible to assess the *persistence* from drill core or drill hole observations.

If closely spaced holes are available, very careful correlation of discontinuities will be required before any reliable conclusions can be drawn concerning the persistence of a given discontinuity or set.

4. *Roughness*

Gross features of discontinuity wall roughness and corresponding full scale shear strength cannot obviously be assessed by means of drill core alone. However, it is usually possible to assign to a surface some degree of *planarity* (*planar, curved, irregular*) and some degree of *smoothness* (*slick, smooth, rough*). This suggested procedure is broadly consistent with the roughness description shown in Fig. 17, but with dimensions reduced to the scale of centimetres and millimetres respectively.

Drill hole inspection with periscopes or TV cameras will not generally provide an improved picture of roughness unless the rock type is so weak and/or the drilling so poorly performed that grinding of the core pieces has occurred.

5. *Wall strength*

The individual suggested methods for describing *wall strength* ((a) weathering grade of rock mass, (b) weathering grade of rock material, (c) manual index tests, (d) Schmidt hammer test) can also be applied to the description of drill core.

Since the drill core provides a ready-made line sample of the rock mass, such features as the depth of penetration of weathering into the discontinuity walls can be directly observed and therefore described quite accurately. Furthermore the drill core provides ready-made samples for mechanical testing (i.e. Schmidt hammer testing of rigidly clamped core pieces for describing *wall strength* or point load testing across the core diameter for describing *material strength*). Franklin *et al.* [2] strongly advocate logging the point load strength index (I_s) simultaneously with recovery of the core from the core barrels.

When assessing wall strength, care should be taken to check if the relevant core pieces fit together. Lack of fit may indicate lost filling material, shear displacement, or partial grinding away of strongly weathered walls during the drilling process.

6. *Aperture*

The aperture of discontinuities intersected by drill holes can only be guessed unless the integral sampling method is used. If the core pieces on either side of a discontinuity can be fitted together by hand so that no visible void spaces remain, it is likely that the discontinuity is a *tight* feature *in situ* (i.e. very tight <0.1 mm, or *tight* 0.1–0.5 mm). However it is not certain that the feature is *tight*, it could also be "gapped" *in situ* (i.e. moderately wide 0.5–2.5 mm, or wide 2.5–10 mm, etc.). Alignment of the walls of the relevant core pieces should be checked in this respect.

If two pieces of adjacent core cannot be mated tightly across a discontinuity and if voids are visible, the term *open* can be used in describing the discontinuities. It is recognised that what appears to be an open or partially open discontinuity in the drill core actually may have been tight *in situ*, if softer filling materials have not been recovered, or if some wear of weathered material has occurred during the drilling operation.

Drill hole inspection using TV cameras or periscopes should be successful in distinguishing between the above *tight* and *open* categories, although it is unlikely that the apertures of the finest joints can be measured accurately. From the point of view of seepage potential the open discontinuities are most important, so this limitation should not be important where highly permeable rock masses are concerned. Methods are available for estimating the theoretical smooth wall apertures of water conducting discontinuities by statistical analysis of water injection tests [3]. However, the real apertures may be several times the theoretical smooth wall apertures due to wall roughness and tortuosity effects.

7. *Filling*

Unless the integral sampling method or best quality drilling equipment is used (i.e. double or triple tube core barrels, split inner tubes, and controlled flushing) the softer filling materials are unlikely to be recovered in significant amounts. Possibly only traces of clay minerals will be visible on the discontinuity walls sampled by conventional drill core. Both traces and larger amounts of recovered filling should be described as to width, mineralogy and strength. The interpretative nature of these descriptions should be made clear.

Where *total core recovery* is less than 100%, and it is suspected that significant amounts of filling or weathered material has been lost in the drilling process, attempts should be made to assess the thickness, location and orientation of the suspected filled zones. The drillers log describing the rate of advance and water loss, type of cuttings and colour of flushing fluid may be invaluable here.

The uncertainties surrounding the parameter *filling*, and its extreme importance where deformation, stability and water seepage are concerned, strongly justify the use of special recovery techniques and the use of borehole viewing techniques.

8. *Seepage*

Observations of drill core may provide indirect evidence of water seepage levels. Reddish-brown iron (Fe^{3+}) staining usually indicates the zone of rock mass that lies above the mean ground water level. Oxidation in discontinuity walls lying beneath the ground water level may also occur, but at a greatly reduced rate. Frequently the strongest iron staining is found in the zone where the ground water level commonly fluctuates.

Drill holes obviously provide the means of checking ground water levels directly using simple battery operated electrical contact devices which are lowered into the holes. Additional information on standing water levels should be obtained from the drillers log for each drill hole. Drill hole walls can be surveyed for seepage horizons using periscopes and TV cameras.

Testing performed in drill holes (i.e. falling head tests, Lugeon packer tests, tracer tests, piezometer measurements) for estimating rock mass permeability, and for estimating the hydraulic conductivity of individual discontinuities and sets of discontinuities, forms the subject of a separate ISRM suggested method. The logging and presentation of any available Lugeon values gives important supplementary data, which can conveniently be presented as a log, parallel with that for *total core recovery*, *frequency* and *RQD*, etc.

9. *Number of sets*

The amount of information obtainable from drill core and drill hole observation will obviously depend on the orientation of the holes relative to existing sets, and on their length relative to the joint spacings. If existing surface mapping has already indicated the approximate orientation of certain discontinuity sets, then

arefully orientated holes can be used to check the umber of sets at depth. Drill core observation will e easier if holes are drilled to intersect the different ets at recognisably different angles. Usually at least wo non-parallel holes will be required.

The number of sets observed at the surface is likely o be more than the number observed at depth. Comparison of surface observations with tunnel excavations uggests that this is not just due to the limitations of drill hole sampling.

0. *Block size*

The term *block size* is a composite description of he rock mass which is influenced by *spacing, number of sets, persistence* and *orientation*. A log of block size produced from observations of rock core can clearly only give an approximate picture of the true block size.

A rapid method of estimating the approximate block size from drill core is to select by eye several typical pieces of core and take their average dimensions (±10%). Each rock unit or domain may be assessed in this way. If the relevant hole is orientated such that all sets present are intersected (i.e. a diagonal hole in the case of a cubic joint system) then these average core pieces will roughly represent the *block size index* (I_b) defined under the relevant suggested method. A depth log showing the variation of this index can be a very useful supplement to drill core description.

Notes

(*a*) When estimating *frequency* or *RQD* from drillcore it is necessary to discount fresh artificial breaks (fractures) clearly caused by the drilling process, and also those made deliberately when fitting core into the core boxes. The following criteria are suggested:

(i) A rough brittle surface with fresh cleavage planes in individual rock minerals indicates an artificial fracture.

(ii) A generally smooth or somewhat weathered surface with soft coating or infilling materials such as talc, gypsum, chlorite, mica or calcite obviously indicates a natural discontinuity.

(iii) In rocks showing foliation, cleavage or bedding it may be difficult to distinguish between natural discontinuities and artificial fractures when these are parallel with the incipient weakness planes. If drilling has been carried out carefully then the questionable breaks should be counted as natural features, to be on the conservative side.

(iv) Depending upon the drilling equipment part of the length of core being drilled may occasionally rotate with the inner barrels in such a way that grinding of the surfaces of discontinuities and fractures occurs. In weak rock types it may be very difficult to decide if the resulting rounded surfaces represent natural or artificial features. When in doubt the conservative assumption should be made, i.e. assume that they are natural.

(v) It may be useful to keep a separate record of the frequency of artificial fractures (and associated lower RQD) for assessing the possible influence of blasting on the weaker sedimentary and foliated or schistose metamorphic rocks.

(*b*) The degree of fracturing of the core during the drilling process may be partly a function of core diameter in the weaker rock types. Since some artificial fracturing is very difficult to distinguish from natural discontinuities (e.g. in the case of weak fissile, cleaved, or foliated rock) it is preferable that the core is not less than NX diameter (55 mm) where rock strength is in question. Use of smaller core diameters (i.e. 32 or 42 mm) puts an increasing responsibility on the drilling crew for the results obtained. A method of correcting RQD to the standard NX size has been suggested by Heuzé [4].

(*c*) Several possible interpretations of the length of core pieces are possible i.e. tip to tip (maximum) length, centre line length or fully circular length. These are illustrated in Fig. 26. Tip to tip measurement involves double-counting at each end of a core piece, while fully circular measurement ignores core pieces that happen to have been drilled with a small subtended angle to one discontinuity in otherwise massive rock. Centre line measurement is therefore strongly recommended.

(*d*) The results of core logging (frequency and RQD) can be strongly time dependent and moisture content dependent in the case of certain varieties of shales and mudstones having relatively weakly developed diagenetic bonds. A not infrequent problem is "discing", in which an initially intact core separates into discs on incipient planes, the process becoming noticeable perhaps within minutes of core recovery. The phenomena are experienced in several different forms:

(i) Stress relief cracking (and swelling) by the initially rapid release of strain energy in cores recovered from areas of high stress, especially in the case of shaly rocks.

(ii) dehydration cracking experienced in the weaker mudstones and shales which may reduce RQD from 100% to 0% in a matter of minutes, the initial integrity possibly being due to negative pore pressure.

(iii) slaking cracking experienced by some of the weaker mudstones and shales when subjected to wetting.

All these phenomena make core logging of *frequency* and *RQD* unreliable. Whenever such conditions are anticipated core should be logged by an engineering geologist as it is recovered and at subsequent intervals until the phenomenon is predictable. An added advantage is that the engineering geologist can perform mechanical index tests such as the point load or Schmidt hammer test, while the core is still in a saturated state.

(*e*) In certain cases it may be helpful to log the *solid core recovery* in addition to the *total core recovery* (R) defined earlier. The *solid core recovery* includes as recovery only those pieces of core that have a complete

circumference. *Total* and *solid* core recovery will only be equivalent when no fragmental material is recovered, i.e. when the rock is massive, or when loss of sample is represented wholly by material carried away in the flushing system.

(*f*) Colour photographs provide a useful and convenient method of recording the appearance of cores and are of considerable value as a permanent record and means of rapid reference. The photograph of each core box should incorporate a suitable metric scale along the entire length of the box. Zones of core loss should be replaced by wooden blocks with legible depth markings. Wetting of the core before photography produces excellent contrast between different rock types and any form of mineralogical banding, but does not help in the observation of discontinuities, due to the general darkening that occurs with wetting.

Presentation of results

In view of the different requirements in rock engineering projects, no attempt will be made to suggest a standardized core log format. If a standard format was employed it would be certain that for one given project, much irrelevant information would be presented, while for another, unusual features of great significance would be missed out because the format did not allow for their inclusion. Since it is impractical to include all the parameters given below, the following should only be used as a check list, so that *relevant* information is included, but irrelevant data excluded.

(*a*) *General information.* (i) Drill hole number. (ii) Site, project name. (iii) Grid reference. (iv) Elevation at drill hole collar. (v) Orientation of hole; dip direction and dip (α/β). (iv) Make of machine, type of feed, type of core barrel and bit, flush system.

(*b*) *Depth logs of relevant parameters selected from the following.* (i) Symbolic log showing rock type (with geological key). (ii) Point load *strength index* (I_s). (iii) *Total core recovery* (R). (iv) *Solid core recovery*. (v) *Lugeon* packer tests (units of Lugeons) and ground water levels. (vi) *Frequency* (F). (vii) *Rock quality designation* (RQD). (viii) *Block size index* (I_b). (ix) Symbolic log showing dip of main discontinuities.

(*c*) *Supplementary data.* Parameters from the following list are probably best presented in writing in a broad column at the side of the above depth logs, unless sufficient data is available to justify separate logs of the relevant data, for specific sets of discontinuities.

(i) Spacing (estimate number of sets).
(ii) Roughness.
(iii) Weathering grades.
(iv) Schmidt hammer tests (wall strength JCS).
(v) Aperture.
(vi) Filling and iron staining.

REFERENCES

1. Phillips F. C. *The Use of Stereographic Projections in Structural Geology.* Edward Arnold. London. 3rd edn. 90 p (1971).
2. Franklin J. A., Broch E. & Walton G. Logging the mechanical character of rock. *Trans Inst. Min. Metall.* Section A. 80, A1–A9 (1971).
3. Snow D. T. Rock fracture, spacing, opening, and porosities. ASCE, Proceedings, Vol. 94, No. SM1, pp. 73–91 (1968).
4. Heuzé E. F. Sources of error in rock mechanics field measurements and related solutions. *Int. J. Rock Mech. Min. Sci.* 8, 297–310 (1971).
5. Deere, D. U. Technical description of rock cores for engineering purposes. *Felsmechanik und Ingenieurgeologie.* 1, 16–22 (1963).
6. Knill J. L. & Jones K. S. The recording and interpretation of geological conditions in the foundations of the Roseires, Kariba and Latiyan dams. *Geotechnique.* 15, 94–124 (1965).
7. Terzaghi R. Sources of error in joint surveys. *Geotechnique.* 15, 287–304 (1965).
8. Deere D. U., Hendron A. J., Patton F. D. & Cording E. J. Design of surface and near-surface construction in rock. *Proc. 8th Symp on Rock Mech.*, Minnesota. pp. 237–302 (1967).
9. Moye G. D. Diamond drilling for foundation exploration. *J. Instr. Engrs.* Aust., CE 9, 95–100 (1967).
10. Ege J. R. Stability index for underground structures in granite rock, in Nevada Test Site. *Mem. Geol. Soc. Am.* No. 110 pp. 185–198 (1968).
11. Ward W. H., Burland J. B. & Gallois R. W. Geotechnical assessment of a site at Mundford, Norfolk, for a large proton accelerator. *Geotechnique.* 18, 399–431 (1968).
12. Deere D. U., Meritt A. H. & Coon R. F. Engineering classification of *in situ* rock. Tech. Rept. no. AFWL-67-144. Air Force System Command, Kirtland Air Force Base, New Mexico (1969).
13. Geological Society Engineering Group Working Party Report on "The logging of rock cores for engineering purposes". *Q. J. Engng Geol.* 3, 1–24 (1970).
14. Rocha M. & Barroso M. Some applications of the new intregral sampling method in rock mechanics. *Proc. of Symp. of ISRM. Rock Fracture.* Nancy, Paper 1–21 (1971).
15. Bergman M. Rock mass investigation in depth: reliability of different methods for drillhole investigations. *Proc. of 3rd Cong. of ISRM.* Denver. *Advances in Rock Mechanics.* Vol. II A. pp. 15–20 (1974).
16. Dearman W. R. The characterization of rock for civil engineering practice in Britain. *Colloquie Géologie de l'Ingenieur.* Liège. pp. 1–75 (1974).
17. Franklin J. A. Rock quality in relation to the quarrying and performance of rock construction materials. *Proc. of 2nd. Int. Cong. of the Int. Assoc. of Engng Geology.* São Paulo. Vol. I. paper IV-PC-2. 11 p (1974).
18. Hansagi I. A method for determining the degree of fissuration of rock. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* 11, 379–388 (1974).
19. Hoek E. & Bray J. *Rock Slope Engineering.* The Institution of Mining and Metallurgy. London. 309 p (1974).
20. Louis C. Reconnaissance de massifs rocheux par sondages et classifications géotechniques de roches. Sols et Foundations. No 319. July–August. pp. 97–122 (1974).
21. Rankilor P. R. A suggested field system for logging rock cores for engineering purposes. *Bull. Ass. Engng Geol.* 11, 247–258 (1974).
22. Van Schalkwyk A. The application of computer techniques for the manipulation and storage of exploratory borehole data. *Proc. of 2nd. Int. Cong. of the Int. Assoc. Engng.* Geology. São Paulo. VI-22 (1974).
23. Priest S. D. & Hudson J. A. Discontinuity spacings in rock. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* 13, 135–148 (1976).
24. Van Schalkwyk A. Rock engineering testing in exploratory boreholes. *Proc. of Symp. on Exploration for Rock Engineering.* Johannesburg. Vol. 1, pp. 37–55 (1976).
25. South African Core Logging Committee. A guide to core logging for rock engineering. *Proc. of Symp. on Exploration for Rock Engineering.* Johannesburg. Vol. 1. pp. 71–86 (1976).