Chapter 2

Description and classification of soils and rocks

INTRODUCTION

From an engineering viewpoint, the ground beneath a site can conveniently be divided into the categories shown in Table 2.1, which are based upon generalizations of its expected behaviour in construction works.

These broad generalizations are, of course, limited in accuracy. But they give the geotechnical engineer a good basis on which to consider, at the start of a project, both the likely construction problems and the methods of investigation which might be used. In practice, it is found that the ground varies continuously beneath a site, and it is not often possible to find sharp transitions from one type of material to another. This then, calls for more refined, systematic, description and classification of soils and rocks.

The history of soil classification and description has been described by Child (1986). Early attempts to divide soil into different categories used laboratory testing, typically particle size distribution. Casagrande (1947) considered that much of this 'textural soil classification' was unreliable, because it did not always reflect the effects of silt and clay on the engineering behaviour of the mass. On the basis of his experience he considered that only the Airfield Classification System, which was based on the results of particle size analysis and plasticity (Atterberg limit) tests (see Chapter 8), was suitable for an assessment of soil for airfield pavement design. It was noted that, with experience, soil might be classified on the basis of visual/manual description alone; Casagrande therefore suggested what he termed 'descriptive soil classification'.

American practice therefore developed in two directions. Soil Classification, based on Casagrande's Airfield Classification System, became standardized (ASTM D2487— 92). As originally proposed, the system is based solely on particle size distribution and plasticity tests, and soils are designated by letters alone, depending upon which group they fall into. At the same time, a soil description system has been developed, based upon visual examination and simple land tests (ASTM D2488—69).

Table 2.1 Categories of ground beneath a site

Material type	Strength	Compressibility	Permeability
Rock	Very high	Very low	Medium to high
Granular soil	High	Low	High
Cohesive soil	Low	High	Very low
Organic soil	Very low	Very high	High
Made ground	Medium to very low	Medium to very high	Low to high

Early British practice was summarized in Cooling *et al.*'s discussion of Casagrande's 1947 paper. Almost without change, this appeared in the first British Code of Practice on Site Investigations, Code of Practice No. 1 (1950). It was republished as Table 1 of CP 2001:1957. Soil description was based on estimated mass engineering behaviour, and used simple visual description and hand tests. A classification system based on Casagrande's work was included for roads and airfields work, that is to assess the behaviour of materials during compaction and under pavements. Subsequent development occurred primarily through work by Akroyd (1957), Dumbleton (1968, 1981), Dumbleton and Nixon (1973), and the Engineering Group Working Party on 'The Preparation of Maps and Plans in Terms of Engineering Geology' (Geological Society of London 1972). BS 5930:1981 contained two parts, a soil description based on mass behaviour, and the British Soil Classification System (BSCS) based on the work of Dumbleton (1968, 1981).

SOIL AND ROCK DESCRIPTION

> Soil and rock description is to a certain degree subjective. In order to minimise the subjective element a systematic examination should be carried out using a standard terminology, whether the material be in a natural exposure, trial pit face or samples recovered from a borehole.
>
> The use of a standardised scheme of description ensures that:
>
> (i) all factors are considered and examined in logical sequence
> (ii) no essential information is omitted
> (iii) no matter who describes the sample, the same basic description is given using all terms in an identical way
> (iv) the description conveys an accurate mental image to the readers
> (v) any potential user can quickly extract the relevant information.
>
> *Norbury et al., 1986*

The engineering description of the ground conditions beneath a site is a progressive exercise which at each step involves further departure from strictly factual description, and thus an increased interpretative element. Three steps are involved.

1. The description of individual samples from a borehole, each sample being described in isolation and in completely factual terms, noting any disturbance or obvious loss of material caused by sampling. Any two geologists or engineers with sufficient and comparable experience should produce almost identical descriptions.
2. The combination of these individual descriptions to form a stratum description on the borehole log. In so doing, the engineer or geologist will take into account the information on the ground conditions, depths to strata changes, groundwater levels, field and laboratory test results, etc., given on the driller's daily record sheets. Interpretation is necessary, and so, as Norbury *et al*. have stated 'strictly, there is no such thing as a "factual" borehole log'.
3. The drawing together of individual borehole, trial pit and exposure records, to arrive at an assessment of the mass properties of the various strata, their geometric distribution, and their variability, in a summary in the text of the ground investigation report.

In this process the skills of soil and rock description, coupled with experience, are paramount.

SOIL DESCRIPTION

The description of soil is currently covered, in the UK, by BS 5930:1981. At the time of writing BS 5930 is under revision. The system of soil description given below therefore does not follow completely the code, but takes into account both current good practice, and changes which have been proposed.

Samples must be described in a routine way, with each element of the description having a fixed position within the overall description:

a) consistency or relative density;
b) fabric or fissuring;

c) colour;
d) subsidiary constituents;
e) angularity or grading of principal soil type;
f) PRINCIPAL SOIL TYPE (in capitals);
g) more detailed comments on constituents or fabric;
h) (geological origin, if known) (in brackets); and
i) soil classification symbols (optional).

Descriptions should be simple, since very detailed comments on all aspects of a soil lead to confusion. Some examples are given below:

Very stiff fissured dark grey CLAY (London clay)
(a) (b) (c) (f) (h)

Loose brown very sandy subangular coarse GRAVEL with pockets of soft grey clay
(a) (c) (d) (e) (f) (g)

Firm laminated brown SILT and CLAY
(a) (b) (c) (f) (f)

Soil types

In routine soil description, the material being considered is first placed into one of the *principal soil types* in Table 2.2.

Table 2.2 Principal soil types

Soil type	Description
CLAY	Cohesive soil
BOULDERS	Granular soils
COBBLES	Granular soils
GRAVEL	Granular soils
SAND	Granular soils
SILT	Granular soils
PEAT	Organic soil
MADE GROUND	Man-made soils and other materials

Most soils will be composed of a variety of different particle sizes, some of which may be cohesive. Whilst *classification* is concerned only to determine the proportion by weight of each constituent, *description* is carried out to ascertain probable engineering behaviour. In this sense, we adhere to the proposals of CP 2001 (1957) and reject BS 5930:1981, following Norbury *et al*. (1986) (Table 2.3).

Table 2.3 Comparison of CP 2001 and BS 5930

CP 2001	BS 5930
Soils possessing cohesion and plasticity are described as fine soils, although the majority of the soil by weight may be coarse or very coarse soil. It is not possible to give a percentage of clay and/or silt above which they become the principal component, since the mass behaviour depends on the mineralogy of the soil particles. The description is based on engineering judgement.	Soils with more than 35% clay and/or silt are described as either clay or silt. Soils with less than 35% are described in terms of coarse or very coarse soils, irrespective of whether they have cohesion and plasticity. The description is therefore based on the particle size distribution, but the division between silt and clay is strictly on the Atterberg limits. These factors can be difficult to assess visually for some materials and laboratory tests are required to confirm descriptions.

The fundamental difference between these two proposals is that under the BS 5930:1981 proposals, many 'clays' i.e. materials of low strength, very low permeability and high compressibility, must be termed silts, sands or gravels. In practice the addition of about only 12% by weight of clay is required to make a well-graded granular material perform in engineering works as a cohesive soil. Therefore the stance of BS 5930:1981 is considered untenable.

The first stage of the description process is the identification of the principal soil type, on the basis of the expected behaviour of the soil mass. In soils where the granular fraction dominates behaviour (termed '*granular soils*'), the principal soil type is identified on the basis of a particle size.

As an aid to visual identification, it should be noted that coarse silt represents the normal limit of resolution of individual grains with the unaided eye. The principal soil type is the (single) component of the soil (i.e. boulders, cobbles, gravel, sand or silt) which is thought to represent (in a coarse soil) the greatest proportion by weight (Table 2.4). Where two types are thought to be equal, two components may be given (for example, sand and gravel, boulders and cobbles). BS 5930:1981 rightly noted that the properties of very coarse soils (i.e. boulders and cobbles) cannot be reliably estimated from boreholes-trial pits or exposures must be used.

Table 2.4 Identification of principal soil type

Principal soil type	Particle size (mm)		Particle size (mm)
Boulders	>200		
Cobbles	60—200		
		coarse	20—60
Gravel	2—60	medium	6—20
		fine	2—6
		coarse	0.6—2
Sand	0.06—2	medium	0.2—0.6
		fine	0.06—0.2
Silt	<0.06		

In soils where the cohesive fraction dominates the engineering behaviour (termed 'cohesive soils'), the soil is described as *clay*. BS 5930:1981 differentiates between silts and clays on the basis of their position relative to (i.e. above or below) Casagrande's 'A' line — see later. Since some pure clay minerals plot below the A-line (see Dumbleton and West (1966), for example) and some silts plot above the A-line (Clayton 1983) this approach is not considered reasonable. Silts are fine-grained non-cohesive soils — as such they have relatively good effective strength, compressibility and drainage properties, from an engineering point of view. The separate identification of silts and clays can therefore most easily be made on the basis of dry strength. Silts have very low to low dry strengths, whilst clays have medium to very high dry strengths (see, for example, ASTM D2488).

Peat is the name given to soil which is primarily composed of plant matter, usually found decomposing in mires and fens. The ASTM sub-committee on peat and organic soils have concluded that an organic soil should not be called a peat unless its organic content is 75% or more. Any organic content should be recorded during sample description, since even a small proportion can lead to large increases in compressibility and decreases in the strength of an otherwise good material.

Topsoil is the thin layer of aerated organic matter, close to ground surface, which supports living vegetation.

Hobbs (1986) has suggested that a full description of a peat should include details of colour, degree of decomposition, wetness, main constituents (e.g. fibre type), mineral soil content, smell, chemistry, tensile strength, and plasticity. Such complex description is clearly outside the scope of description that can be justified for normal ground investigation, and indeed will not normally yield useful correlations with geotechnical behaviour. The proposals of Burwash and Weisner (1984, 1987) would seem to be adequate for most purposes:

1. determine organic content, to confirm that the material is a peat (>75% organic matter);
2. describe degree of humification in accordance with the von Post method (Table 2.5); and
3. where possible, give basic fibre details.

Examples of description would be:

- black amorphous PEAT (H_6);
- brown non-woody fine fibrous PEAT (H_2).

It is extremely important to attempt, during the description of near-surface soils, to identify made ground. Such material may be:

- compressible;
- highly variable;
- chemically contaminated.

Made ground is ground filled by man's activities, rather than as a result of geomorphological processes. Made ground may comprise (for example) compacted granular fill, in which case it may be extremely difficult to detect from the description of an isolated sample. At the other extreme, it may result from the tipping of household waste. In either case, sample description can only hope to identify made ground by searching for man-made artefacts, such as fragments of brick, clinker, tile, glass, etc., and at the other end of the scale, concrete, cars and parts of machinery, paper and plastics.

Table 2.5 Assessment of degree of humification (after von Post (1922))

Degree of humification	Decomposition	Plant structure	Content of amorphous material	Material extruded on squeezing (passing between fingers)	Nature of residue
H1	None	Easily identified	None	Clear, colourless water	
H2	Insignificant	Easily identified	None	Yellowish water	
H3	Very slight	Still identifiable	Slight	Brown, muddy water; no peat	Not pasty
H4	Slight	Not easily identified	Some	Dark brown, muddy water; no peat	Somewhat pasty
H5	Moderate	Recognizable, but vague	Considerable	Muddy water and some peat	Strongly pasty
H6	Moderately strong	Indistinct (more distinct after squeezing)	Considerable	About one-third of peat squeezed out; water dark brown	
H7	Strong	Faintly recognizable	High	About one-half of peat squeezed out; any water very dark brown	Fibres and roots more resistant to decomposition
H8	Very strong	Very indistinct	High	About two-thirds of peat squeezed out; also some pasty water	
H9	Nearly complete	Almost not recognizable		Nearly all the peat squeezed out as a fairly uniform paste	
H10	Complete	Not discernible		All the peat passes between the fingers; no free water visible	

As with very coarse soils, made ground is best described in an exposure or a trial pit. But in this case there will be much greater safety considerations. Made ground can often produce poisonous gas (as a result of decomposition of organic material), will contain sharp materials, likely to injure (glass, metals, etc.), and may give rise to instability in the sides of trial pits. It is not advisable to allow work to be carried out from within trial pits in made ground unless proper support, breathing apparatus, and full protective clothing are available.

There are no formal systems in use for the description of made ground. Where the made ground resembles natural soil, then normal soil descriptions can be used, but with additional comments added to inform the reader as to how the material has been identified as made ground (for example, 'scattered brick and tile fragments'). In all cases the following should be noted:

- organic matter, and its degree of decomposition;
- smell;
- striking colours;
- signs of heat (combustion);
- presence or absence of large objects (concrete blocks, cars, fridges, etc.)
- voids, hollow objects;
- other compressible materials; and
- anything by which the made ground may be dated (for example, product labels, old newspapers).

Secondary constituents

Where soils are composed of a variety of different constituents, a simple scheme is required to allow them to be identified. Norbury *et al.* (1986) have criticized the complexity of the proposals in BS 5930, and there is certainly considerable evidence that they have not been applied in practice. It is suggested, following Norbury *et al*., that the scheme of Table 2.6 should be adopted.

Table 2.6 Identification of soils

Principal soil type	Approx. % of secondary constituent (by weight)	Terminology used: Before principal constituent	Terminology used: After principal constituent
Granular	0	—	—
soil types	<5	Slightly (sandy*)	With a little (sand*)
(boulders,	5—20	— (sandy*)	With some (sand*)
cobbles,	20—40	Very (sandy*)	With much sand
gravel,	about 50	(SILT*) and	(SAND*)
sand, silt)			
Cohesive	0	—	—
soil types	<35	Slightly (sandy*)	With a little (sand*)
(clay)	35—65	— (sandy*)	With some (sand*)
	>65	Very (sandy*)	With much (sand*)

* Use appropriate secondary soil type (e.g. gravel, sand, silt or clay). The description silty CLAY' is not used, and where the sample contains silt or clay the above terms are used to indicate an estimated degree of cohesiveness.

In practice it is very difficult to estimate the secondary constituents of soils by eye and by feel, and particularly so in cohesive soils. Therefore it is important to check descriptions to ensure that they reflect the estimated properties of the different materials, and that the implied total percentage does not exceed 100%. It should be borne in mind that the description of the presence of even small quantities of fines is helpful in granular soil (because the permeability of granular soil is dominated by its fine fraction), but is not particularly helpful in cohesive soil, where the description of fabric (i.e. the spatial distribution of different particle sizes) is more critical, since this has a large effect on mass strength and permeability. Not more than two secondary constituents should be used, of which only one should appear before the principal soil type. It should be remembered that the method of sampling can change the proportion of different soil types.

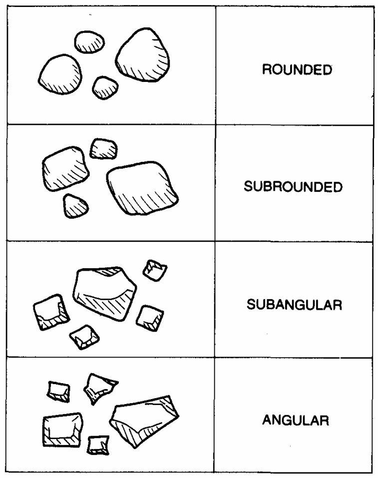
Examples are:

- Slightly silty SAND;
- Silty SAND and GRAVEL with a little clay;
- Very sandy CLAY.

Grading and particle shape

Coarse granular materials, such as sands and gravels have particles of sufficient size to allow a visual assessment of their shape, angularity and grading. Only extremes of shape, such as flat or equidimensional particles should be noted. Angularity is defined in BS 812 and ASTM D2488, and should be given only for boulders, cobbles, gravels and coarse sands. Since roundness depends largely on the method and distance of transportation of the material from its original bedrock, angularity can be useful not only in assessing sands and gravels for use as aggregate, but also in determining the origins of a material and the weathering processes which have brought it to its present state. Figure 2.1 illustrates angularity.

Fig. 2.1 Angularity of coarse soil particles.



Because of their limited accuracy, visual estimates of grading should be restricted to extremes of grading in the coarser soil fractions. The terms ‘uniform’ (i.e. containing a restricted range of particle size) or ‘well-graded’ (i.e. containing a wide range of particle sizes) can be used immediately before the main constituent. For the purposes of classification of soil for earthworks in the UK, the boundary between ‘well-graded’ and ‘uniform’ may be taken at a value of the coefficient of uniformity (D60/D10) equal to 10.

Density and strength

The relative density of granular soils is assessed on the basis of field testing. If field tests are not carried out, then the density description should not be used.

Simple field tests, suitable for application in trial pits, are given in BS 5930: 1981:

Loose	Can be excavated with a spade: 50mm wooden peg can easily be driven
Dense	Requires pick for excavation: 50mm wooden peg hard to drive
Slightly	Visual examination: pick removes soil in lumps which can be cemented abraded.

In boreholes, SPT results are routinely used to provide an estimate of density. Traditionally in the UK Terzaghi and Peck's (1948) classification for sand has been used, regardless of the granular soil type as shown in Table 2.7.

Table 2.7 Classification for sand

SPT N(blows/300 mm)	Relative density
0—4	Very loose
4—10	Loose
10—30	Medium dense
30—50	Dense
>50	Very dense

However, different components have used different systems. In some the density descriptor (for example, medium dense) used on a borehole log was obtained simply by averaging the SPT in a given stratum, and looking up the appropriate term in Table 2.7. In others, following Gibbs and Holtz (1957) work on the effect of overburden presence on the SPT 'N' value, 'N' values were corrected to a common overburden pressure before the descriptor was chosen. These two procedures will produce quite different results either at very shallow depth or at very great depths.

Following Skempton (1986) and Clayton (1992) the procedure proposed is as follows.

1. 'N' values should be corrected for effective overburden pressure and energy (see Chapter 10), to give $(N_1)_{60}$
2. In coarse granular soils, it should be noted that the N value will be high for the *in situ* relative density. There is now increasing evidence that coarse (i.e. gravel) particles significantly increase penetration resistance, and the classification given below, which was derived for sands, will overestimate the density of gravels.
3. The density descriptor should be selected from Table 2.8.

Table 2.8 Selection of density descriptor

$(N_1)_{60}$ (blows/300 mm)	Density descriptor
0—3	Very loose
3—8	Loose
8—25	Medium dense
25—42	Dense
42—58	Very dense

During sample description, simple hand tests are used to describe the consistency of cohesive soils. Consistency is the estimated undrained shear strength of the intact blocks of a soil. Large diameter triaxial tests carried out on fissured clays will normally give much lower values of undrained shear strength because of the weakening effect of the fissures. Three simple tests are commonly used to determine consistency. These use the hand and fingers, a pocket penetrometer or a hand vane tester (Fig. 2.2). The vane test will normally give undrained shear strengths in engineering units, but most pocket penetrometers are calibrated in terms of unconfined compressive strength; this should be halved to find the undrained shear strength. All tests used to find consistency are carried out on small samples at a rate much faster than is used in laboratory strength testing. For these reasons their results should not normally be used in engineering calculations. Some penetrometers have in-built (empirical) corrections, which are intended to correct for size and rate effects. These corrections must be removed by calculation before the strength descriptor is selected from the list below.

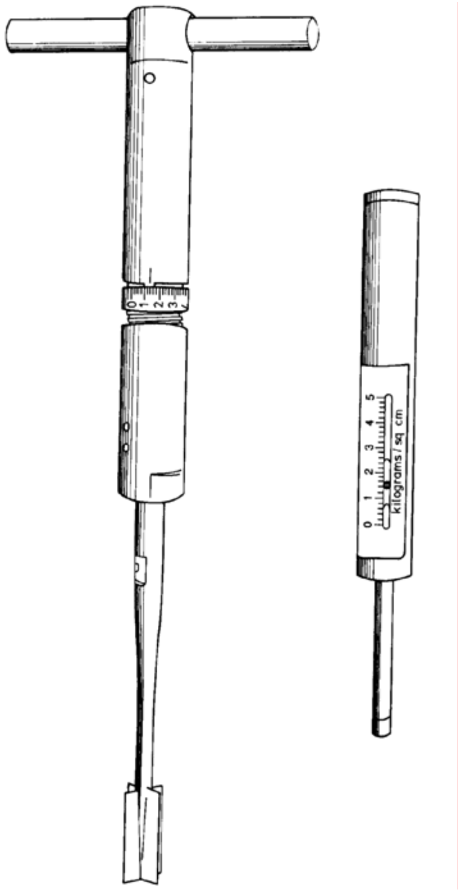


Fig. 2.2 Hand vane and pocket penetrometer.

Where strength measurements cannot be made, the consistency of a soil can be determined with reasonable accuracy using hand and fingers, as follows:

1. Very soft Exudes between fingers when squeezed in hand
2. Soft Moulded by light finger pressure
3. Firm Can be moulded by strong finger pressure
4. Stiff Cannot be moulded by fingers Can be indented by thumb
5. Very stiff Can be indented by thumb nail
6. Hard Cannot be indented by thumb nail (not included in BS 5930:1981).

Where a sample straddles a boundary, this can be indicated as, for example, ‘firm to stiff’. On the engineer’s borehole record, where a gradual or sharp change in consistency occurs within a clay formation, it is common to use a description such as ‘firm, becoming stiff from 12 m, etc.’. Alternatively, if the formation has been fully described before the depth has been reached at which a change in consistency occurs, one can put ‘...becoming stiff at 12 m’ opposite the relevant depth. Consistency is related directly to strength as shown in Table 2.9.

The blows necessary to drive open-drive sampler tubes such as the SPT split spoon or U100 into the ground are sometimes used in estimating consistency. Opinions on the use of the SPT N value to determine undrained shear strength vary. De Mello (1969) shows that correlations between strength and N value are generally poor, but Stroud (1974) has found good correlations for heavily overconsolidated soils, mainly in the UK (Fig. 9.3). There seems little point in attempting to correlate the blows required to drive a U100 sampler with soil strength, because the weight of the hammer and ancillary equipment can vary considerably. Some drilling companies use a jarring link and sinker bar at the base of the hole, while others rod up to the top of the hole and drive the tube with an SPT trip hammer.

Table 2.9 Relationship between consistency and undrained shear strength of intact soil

Consistency	Undrained shear strength			
	p.s.i.	p.s.f.	t.s.f.	kN/m^2
Very soft	<2.5	<375	<0.20	<20
Soft	2.5—5.0	375—750	0.20—0.40	20—40
Firm	5—10	750—1500	0.40—0.75	40—75
Stiff	10—20	1500—3000	0.75—1.50	75—150
Very stiff	20—40	3000—6000	1.50—3.00	150—300
Hard	>40	>6000	>3.00	<300

Fabric and structure

Fabric can be defined as the arrangement of different particle size groups within a soil, whereas structure relates to the arrangement of individual particles within a particular size group. Fabric is of considerable importance in determining the mass behaviour of soil, as Rowe (1968a,b, 1971, 1972) has shown, and every effort should be made to assess its form during engineering sample description. Structure is normally difficult to detect either with the naked eye or a hand lens.

Fabric may exist in the form of silt filled fissures, laminations or varves consisting of very thin successive layers or gradations of clays, silts and sands, pockets and lenses of granular material which have a limited lateral extent within a mass of clay, or layers or seams which have a reasonable thickness and considerable lateral extent. The only satisfactory way to look for fabric is to cut halfway into an undisturbed sample along its length, and then pull it into two equal semi-cylindrical parts (e.g. Fig. 7.4). In this way the uncut part of the flat face will not be smeared, and coarse fabric will be immediately obvious. The presence of fine silt layers or dustings in clays can be detected by leaving one-half of the sample exposed. The silt, which dries much faster than the clay, will show up as lighter. Fissures should be examined to see whether they contain silt or sand sized material, and will therefore speed consolidation. The spacing of these features is of obvious relevance to the choice of sample and test specimen size. BS 5930:1981 defines descriptive terms for the spacing of bedding and other discontinuities.

Colour

Colour changes may indicate the extent of weathering, or changes of strata. The absolute colour of a soil is rarely of enormous importance, which is perhaps fortunate as many people are colour-blind and also because colour is highly subjective. The colour seen by one person will depend on the type of light source, the background, the size of the object and the colours that have been seen immediately before. Therefore, if colour is to be recorded objectively certain precautions should be taken. When describing soils in a laboratory, daylight fluorescent tubes should be used in order to try to obtain the same colours as will have been seen in the field. Colours should be compared with a standard chart specifically designed for this purpose. One such, based on the Munsell colour classification system is the Geological Society of America's 'Rock-colour Chart'.

SOIL CLASSIFICATION

Soil classification systems are set up to allow the expected properties of the soil in a given situation to be conveyed in a shorthand form. Geotechnical soil classification systems are often termed 'textural classification', probably because of their agricultural origin ('textural' referred to the appearance of the ground several weeks after ploughing (Smart 1986) — grouping of soils into 'light', 'medium' and 'heavy' corresponds to the need for one, one-and-a-half and two horses respectively to plough one furrow). In current usage, soil classification systems are primarily aimed at road and airfield applications.

The British Soil Classification System appeared first in BS 5930:1981. A corrected version was subsequently published by Dumbleton (1981), see Table 2.10. The US Soil Classification is standardized in ASTM D2487—69. In both systems the soil is classified on the basis of particle size distribution and plasticity (Atterberg limit) tests (see Chapter 8 for further information), and described in terms of Group Symbols (such as 0, for gravel). Where required, these symbols can be applied to engineering borehole records.

Soil classification works well for granular soils, but is less satisfactory for cohesive soils. In fact, the soils are divided into 'coarse' and 'fine' based solely on the percentage by weight passing a given sieve size (35% finer than 0.06mm in the BSCS). The same soil 'mix' will behave differently, if clay is present, depending upon the type of clay mineral (e.g. kaolin, illite, montmarillonite). Also, most soil classification systems do not take into account the differences that occur when the same clay fraction is mixed with granular soils of different gradings (see, for example, Smart, 1986). When soils are classified as 'fine soils' then the division between silt and clay is made by reference to Casagrande's A-line (Fig. 8.4). 'Clays' are said to plot above the A-line and silts below. Yet crushed upper chalk (demonstrably a silt-sized material, and 98.5% calcium carbonate) plots above the A-line, while pure kaolin (and kaolin—silt mixtures) plots below the A-line. Smart (1986) has rightly concluded that 'samples plotting below the A-line should not automatically be called silt, nor should samples above it be called clay'. But whatever their faults, soil classification systems must be applied to the letter when they are used.

Note I The name of the soil group should always be given when describing soils, supplemented, if required, by the group symbol, although for some additional applications (e.g. longitudinal sections) it may be convenient to use the group symbol alone.
Note 2. The group symbol or sub-group symbol should be placed in brackets if laboratory methods have not been used for identification, e.g. (GC).
Note 3. The designation FINE SOIL or FINES, F, may be used in place of SILT, M, or CLAY, C, when it is not possible or not required to distinguish between them.
Note 4. GRAVELLY if more than 50% of coarse material is of gravel size. SANDY if more than 50% of coarse material is of sand size.
Note 5. SILT (M-SOIL), M, is material plotting below the A-line, and has a restricted plastic range in relation to its liquid limit, and relatively low cohesion. Fine soils of this type include clean silt-sized materials and rock flour, micaceous and diatomaceous soils, pumice, and volcanic soils, and soils containing halloysite. The alternative term 'M-soil' avoids confusion with materials of predominantly silt size, which form only a part of the group. Organic soils also usually plot below the A-line on the plasticity chart, when they are designated ORGANIC SILT, MO.
Note 6. CLAY. C. is material plotting above the A-line, and is fully plastic in relation to its liquid limit.

Table 2.10 The British Soil Classification System (Dumbleton 1981)

Soil groups (see note 1)					Subgroups and laboratory identification			
		GRAVEL and SAND may be qualified Sandy GRAVEL and Gravelly SAND, etc. where appropriate	Group symbol (see notes 2 & 3)		Subgroup symbol (see note 2)	Fines (% less than 0.06 mm)	Liquid limit %	Name
COARSE SOILS less than 35% of the material is finer than 0.06 mm	GRAVELS More than 50% of coarse material is of gravel size (coarser than 2mm)	Slightly silty or clayey GRAVEL	G	GW	GW	0 to 5		Well graded GRAVEL
				GP	GPu GPg			Poorly graded/Uniform/Gap graded GRAVEL
		Silty GRAVEL	G-F	G-M	GWM GPM	5 to 15		Well graded/Poorly graded silty GRAVEL
		Clayey GRAVEL		G-C	GWC GPC			Well graded/Poorly graded clayey GRAVEL
		Very silty GRAVEL	GF	GM	GML, etc	15 to 35		Very silty GRAVEL; subdivide as for GC
		Very clayey GRAVEL		GC	GCL		<35	Very clayey GRAVEL (clay of low,
					GCI		35 to 50	intermediate,
					GCH		50 to 70	high,
					GCV		70 to 90	very high,
					GCE		>90	extremely high plasticity)
	SANDS More than 50% of coarse material is of sand size (finer than 2mm)	Slightly silty or clayey SAND	S	SW	SW	0 to 5		Well graded SAND
				SP	SPu SPg			Poorly graded/Uniform/Gap graded SAND
		Silty SAND	S-F	S-M	SWM SPM	5 to 15		Well graded/Poorly graded silty SAND
		Clayey SAND		S-C	SWC SPC			Well graded/Poorly graded clayey SAND
		Very silty SAND	SF	SM	SML, etc	15 to 35		Very silty SAND, subdivided as for SC
		Very clayey SAND		SC	SCL		<35	Very clayey SAND (clay of low,
					SCI		35 to 50	intermediate,
					SCH		50 to 70	high,
					SCV		70 to 90	very high
					SCE		>90	extremely high plasticity)

Table 2.10 *continue* (Dumbleton 1981)

Soil groups (see note 1)					Subgroups and laboratory identification			
GRAVEL and SAND may be qualified Sandy GRAVEL and Gravelly SAND, etc. where appropriate			Group symbol (see notes 2 & 3)		Subgroup symbol (see note 2)	Fines (% less than 0.06 mm)	Liquid limit %	Name
FINE SOILS more than 35% of the material is finer than 0.06 mm	Gravelly or sandy SILTS & CLAYS 35% to 65% fines	Gravelly SILT	FG	MG	MLG, etc	35 to 65		Gravelly SILT: subdivide as for CG
		Gravelly CLAY (see note 4)		CG	CLG		<35	Gravelly CLAY of low plasticity
					CIG		35 to 50	of intermediate plasticity
					CHG		50 to 70	of high plasticity
					CVG		70 to 90	of very high plasticity
					CEG		>90	of extremely high plasticity
		Sandy SILT (see note 4)	FS	MS	MLS, etc	35 to 65		Sandy SILT; subdivide as for CG
		Sandy CLAY		CD	CLS, etc			Sandy CLAY; subdivide as for CG
	SILTS & CLAYS 65% to 100% fines	SILT (M-SOIL)	F	M	ML, etc	65 to 100		SILT: subdivide as for CG
		CLAY (see note 5 & 6)		C	CL		<35	CLAY of low plasticity
					CI		35 to 50	of intermediate plasticity
					CH		50 to 70	of high plasticity
					CV		70 to 90	of very high plasticity
					CE		>90	of extremely high plasticity
ORGANIC SOILS	Descriptive letter ‘O’ suffixed to any group or sub-group symbol.						Organic matter suspected to be a significant constituent. Example MHO. Organic SILT of high plasticity.	
PEAT	Pt	Peat soils consist predominantly of plant remains which may be fibrous or amorphous.						

ROCK DESCRIPTION

An engineering description of rock should, in theory, embody those features which are significant in influencing its engineering performance. Most rocks are cut by discontinuities which characteristically have little or no tensile strength. The engineering performance (strength, compressibility, permeability and durability) of any mass of rock containing such fractures will be significantly influenced by their presence. The description of these fractures is clearly an important aspect of rock description in general. A complete rock description is commonly divided into three parts:

1. a description of the rock material (or intact rock):
   the term rock material here refers to rock that has no through-going fractures significantly reducing its tensile strength;
2. a description of the discontinuities; and
3. a description of the rock mass:
   the term rock mass here refers to the rock material and the discontinuities. information from (1) and (2) are therefore combined to provided an overall description of the rock mass.

Ideally, the best means of obtaining a comprehensive description of the rock mass is by careful examination of large exposures. In many site investigations on rock, however, it is not always possible to gain access to such large exposures and hence the rock mass description has to be derived mainly from borehole information. Boreholes provide a reasonable means for examining the rock material but do not permit a comprehensive description of the discontinuities.

In soil description, the soil name is derived from the particle size or organic constituent which is present in sufficient proportions to have the most significant influence on the engineering behaviour. Hence the soil name conveys valuable information on engineering performance. The systematic engineering description of soils has evolved with the relatively young science of soil mechanics. In contrast to this, a basis for naming rocks was already well established within the science of geology when the need to develop a method for the systematic engineering description of rock arose. Rock names currently used in such descriptions are therefore, by tradition, based on geological names. This can be a source of confusion among engineers with minimal geological training. The names reflect the genesis of the rock and are often based on such factors as grain size, texture, and mineral assemblage. The resultant names seldom bear any relation to engineering performance. For the igneous rocks which comprise only 25% of the Earth’s crust there are over 2000 names, reflecting subtle mineralogical differences, usually with little engineering significance. In contrast, sedimentary rocks have fewer names and yet they are more abundant and can exhibit a much broader range of engineering properties.

Geological classification of rock

Rocks are divided into three groups based on mode of formation. These groups include the following.

1. *Igneous rocks*. These are formed from the solidification of molten material.
2. *Sedimentary rocks*. These are formed from the accumulation of fragmental rock material and organic material or by chemical precipitation.
3. *Metamorphic rocks*. These are formed by alteration of existing rocks through the action of heat and pressure.

A series of handbooks produced by the Geological Society of London on the field description of igneous rocks (Thorpe and Brown 1985), sedimentary rocks (Tucker 1982) and metamorphic rocks (Fry 1984), provide some useful advice but are aimed primarily at field geologists. Within each genetic class, different classification systems are employed. In many cases a full petrographic analysis is required to classify a rock specimen in geological terms. Such classification systems are too elaborate for engineering application, and usually provide little or no information of engineering significance. For engineering use the classification systems have been simplified and the number of rock names kept to a minimum. In a full rock description a similar approach to describing soil samples is employed, making use of prefixes and suffixes to describe selected features of the hand specimen and to include estimates of certain engineering characteristics. Before discussing the scheme by which rocks are described, it is necessary to outline how rock types are identified.

The principal criteria used in classifying all types of rock material include:

1. mineral assemblage;
2. texture and fabric,
   texture refers to the mutual relationship between the constitutive crystals/grains; and
3. grain size.

Igneous rocks

Rocks in this broad family are characterized by a crystalline or more rarely, glassy texture with low porosity (usually <2%), unless the rock has been weathered. Generally the strongest rocks are found among this group.

Igneous rocks are formed from solidification of molten material (magma) which may originate in or below the Earth's crust. Magma may solidify within the crust (at depth or near the surface) giving rise to intrusive igneous rocks, or it may pour out on to the Earth's surface before solidifying completely, giving rise to extrusive igneous rocks. The initial chemical composition of the magma together with the rate at which the magma cools determine the mineral assemblage, texture and grain size of the resulting igneous rocks.

The grain size of igneous rock can range from very coarse (equivalent to gravel and cobbles in soil) to fine (equivalent to silt and clay in soil) and is related to the rate of cooling of the parent magma. Coarse-grained rocks are associated with slow cooling rates, and fine-grained rocks with more rapid rates of cooling. If the magma cools very rapidly then there is no time for crystals to develop and amorphous glassy rock is produced. Generally intrusive igneous rocks are crystalline with grain sizes ranging from very coarse- to medium-grained (cobbles to fine sand in soil terms). Extrusive rocks are usually fine-grained (silt to clay in soil terms), crystalline, glassy (or opalescent) or porous.

The principal criteria used in the petrological classification of igneous rocks are as follows.

- Mineral assemblage.
  The main rock forming minerals which occur in igneous rocks include quartz, feldspar, muscovite, biotite and mafic minerals. Classification is based on the relative proportions of quartz, feldspar and mafic minerals. Quartz and feldspar are generally light in colour, whereas the mafic minerals are generally dark. A simple classification of igneous rocks may be based on colour index (i.e. whether the rock is dark or light in colour).
- Grain size.

A simplified classification suitable for engineering use is shown in Table 2.11. In general, most igneous rocks will be placed in the acid or basic category. It is often difficult to identify intermediate igneous rocks in hand specimen or indeed in the mass. It should be pointed out that experienced igneous petrologists will often resort to a simple classification such as that shown in Table 2.11 when making a preliminary description of the rock as seen in the field. A more obscure geological name would normally only be applied after thin sections have been studied in the laboratory.

Table 2.11 Classification of igneous rocks

Field relations	texture	Grain size	Acid	Intermediate	Basic	Ultrabasic
			Light coloured rocks	Light/dark coloured rocks	Dark coloured rocks	Dark coloured rocks
Intrusive	Crystalline	Very coarse grained 60 mm	Rock consists of very large and often well developed crystals of quartz, feldspar, mica and frequently rare minerals. PEGMATITE			
Intrusive	Crystalline	Coarse grained 2 mm	At least 50% of the rock is coarse grained enough to allow individual minerals to be identified.			Rock is coarse grained and dark in colour (dull green to black) with a granular texture. It contains olivine and augite in abundance but no feldspars.
			Rock is light coloured with an equigranular texture (majority of grains approximately the same size) and contains >20% quartz with feldspar in abundance.	Rock may be medium to dark in colour with a more or less equigranular texture and contains <20% quartz with feldspar and hornblende in abundance.	Rock is dark coloured and often greenish with abundant plagioclase (about 60%) and augite together with some olivine. The rock usually feels dense.	
			GRANITE	DIORITE	GABBRO	PERIDOTITE
Intrusive	Crystalline	Medium grained 0.06 mm	At least 50% of the rock is medium grained. Crystal outlines are generally visible with the aid of a hand lens but individual minerals may be difficult to identify.			Rock is greyish green to black with a splintery fracture when broken and generally feels soapy or waxy to the touch. It is often criss-crossed by veins of fibrous minerals and/or banded.
			Rock is similar in appearance to granite but the crystals are generally much smaller.	Rock is similar in appearance to dionte but the crystals are generally much smaller.	Rock is dark coloured and often greenish with a granular texture. Individual minerals may be difficult to identify. The rock usually feels dense.	
			MICRO-GRANITE	MICRO-DIORITE	DOLERITE	SERPENTINITE
Extrusive	Crystalline/glassy	Fine grained	At least 50% of the rock is fine grained. Outlines of crystals are not usually visible even with the aid of a hand lens. All rocks in this category may be vesicular.			
			Rock is light coloured (often pale reddish brown or pinkish grey) and may be banded. RHYOLITE Rock is light coloured with a very low specific gravity and highly vesicular.	Rock is medium to dark in colour (shades of grey, purple, brown, or green) and frequently porphyritic	Rock is black when fresh and becomes red or green when weathered. The rock is often vesicular and/or amygdaloidal.	
			PUMICE	ANDESITE	BASALT	
Extrusive	Glassy	Glassy	Rock is glassy and contains few or no phenocrysts. It is often black in colour and has a characteristic vitreous lustre and conchoidal fracture. OBSIDIAN Rock is glassy and contains few or no phenocrysts. It may be black, brown or grey in colour with a characteristic dull or waxy lustre. PITCHSTONE			

Basalt is probably the most common extrusive igneous rock. It is generally characterized by structures such as flow banding, often with porous zones (pumice) and columnar jointing. Basalts extruded under water often exhibit pillow shaped structures (pillow lava). Of the intrusive rocks, granite and dolerite are perhaps the most ubiquitous.

Some igneous rocks exhibit large crystals embedded in a finer-grained matrix. Such rocks are termed porphyritic and the large crystals are termed phenocrysts. Extrusive igneous rocks often have numerous spherical or ellipsoidal voids (vesicles) scattered throughout or concentrated in layers. These are produced by the inclusion of gas bubbles within the magma as it cools. In some cases, these voids may be filled with minerals. Such mineral filled inclusions are termed amygdales.

An elementary discussion of igneous rocks is given by Blyth and deFreitas (1984), McLean and Gribble (1985) and Goodman (1993), and a more detailed discussion dealing with genesis and petrography is given by Hatch *et al.* (1972).

Sedimentary rocks

Most sedimentary rocks are cemented aggregates of transported fragments derived from pre-existing rocks. Typically these rocks will comprise rock fragments, grains of minerals resistant to weathering (mainly quartz) and minerals derived from the chemical decomposition of pre-existing rocks (clay minerals) bound together with chemical precipitates such as iron oxide and calcium carbonate. Other forms of sedimentary rock include accumulations of organic debris (typically shell fragments or plant remains), fragmental material derived from volcanic eruptions, and minerals that have been chemically precipitated (for example, rock salt, gypsum and some limestones).

The polygenic nature of sedimentary rocks has resulted in the development of a number of classification schemes. However two broad groups can be identified.

1. *Detrital (fragmental or clastic) sediments*:
   - clastic deposits: accumulations of rock or mineral fragments;
   - bioclastic deposits: accumulations of faunal debris (for example, shell, coral or bones); and
   - pyroclastic deposits: accumulations of fragmental material produced by volcanic eruption.
2. *Organic and chemical sediments*:
   - organic deposits: accumulations of dead plants and other biodegradable material; and
   - chemical deposits: accumulations of minerals chemically precipitated from surface water or groundwater.

The fragmental nature of the rocks in group (1) permits them to be classified primarily on the basis of grain size, predominant mineral composition (i.e. greater than 50%) and texture and fabric. Rocks belonging to group (2) are classified on the basis of composition alone. Table 2.12 shows how sedimentary rocks are classified for engineering purposes.

The detrital sediments when deposited may be regarded as soils. With time they are gradually transformed into rock through the action of consolidation, creep and cementation. This process termed diagenesis results in a progressive increase in strength and decrease in compressibility and permeability. The degree of diagenesis is highly variable and results in rocks with porosities ranging from less than 1% to greater than 50%. Sandstones may be weakly cemented such that individual grains may be easily removed by light abrasion or so well cemented that they are much stronger than concrete. Some of the younger mudrocks in the UK are only weakly cemented and are thus considered as engineering soils (for example, London Clay, Gault Clay, Weald Clay). The older and more strongly cemented mudrocks are generally stronger and are less likely to soften and slake when exposed to the elements. However many of these have been subjected to such high overburden stresses that the platey clay mineral particles have become aligned and impart a fissility to the rock resulting in a high degree of anisotropy. Such rocks are referred to as shale.

Table 2.12 Classification of sedimentary rocks

Group Usual structure	Grain size	Composition and texture	Detrital sediments Bedded	
			Quartz, rock fragments, feldspar, and other minerals.	At least 50% of the rock is composed of carbonate minerals (rocks usually react with dilute HCl).
Granular	Coarse-grained 2 mm	Rudaceous	Rock is composed of more or less rounded grains in a finer grained matrix: CONGLOMERATE Rock is composed of angular or sub-angular grains in a finer grained matrix: BRECCIA	CALCI-RUDITE
Granular	Medium-grained 0.06 mm	Arenaceous	Rock is composed of: (i) mainly mineral and rock fragments. SANDSTONE (ii) 95% quartz. The voids between the grains may be empty or filled with chemical cement. QUARTZ SANDSTONE (iii) 75% quartz and rock fragments and up to 25% feldspar (grains commonly angular). The voids may be empty or filled with chemical cement. ARKOSE (iv) 75% quartz and rock fragments together with 15% + fine detrital material. ARGILLACEOUS SANDSTONE	CALC-ARENITE
Smooth	Fine- grained 0.002 mm	Argillaceous	Rock is composed of at least 50% fine-grained particles and feels slightly rough to the touch. SILTSTONE	CALCI-SILTITE
Smooth	Fine- grained 0.002 mm	Argillaceous	Rock is homogeneous and fine-grained. Feels slightly rough to smooth to the touch. MUDSTONE Rock has same appearance and feels as mudstone but reacts with dilute HCl. CALCAREOUS MUDSTONE	CHALK (Bioclastic)
Smooth	Very fine-grained	Argillaceous	Rock is composed of at least 50% very fine-grained particles and feels smooth to the touch. CLAYSTONE Rock is finely laminated and or fissile. It may be fine or very fine grained SHALE	CALCI-LUTITE

Some sandstones (particularly quartzite) may appear to be crystalline as a result of being relatively fine grained and well cemented or as a result of recrystallization. In the absence of any evidence of bedding or other sedimentary features it would be easy to mistaken them for igneous rocks. In terms of the engineering behaviour of the rock material such a mistake is not serious. However the relationship between an intrusive igneous rock and the host rock is quite different from that of a sandstone unit within a sequence of other sedimentary rocks and hence such an error may have serious consequences.

Rocks containing at least 50% clay minerals are termed claystone if homogeneous or shale if laminated and fissile. These fine-grained rocks will generally have a smooth surface texture.

Detrital sediments containing mainly carbonate minerals may be subdivided on the basis of grain size. They may have granular or smooth textures depending on the grain size. Those with granular textures are termed calci-rudite, calc-arenite and calci-siltite depending on grain size. Very fine-grained varieties include chalk and calci-lutite.

Table 2.12 *continue*

Group Usual structure			Pyroclastic sediments Bedded	Chemical and organic sediments	
	Grain size	Composition and texture		Bedded	Massive/Bedded
			At least 50% of the grains are of fine-grained volcanic material. Rocks often composed of angular mineral or igneous rock fragments in a fine-grained matrix.	Crystalline carbonate rocks depositional texture not recognizable. Fabric is non-elastic.	Depositional textures often not recognizable.
Granular	Coarse-grained 2 mm	Rudaceous	Rock is composed of: (i) Rounded grains in a fine-grained matrix: AGGLOMERATE (ii) Angular grains in a fine-grained matrix: VOLCANIC BRECCIA	Rock is crystalline and composed of carbonate (>90%) reacts violently with HCl. LIMESTONE Rock is crystalline and may show a yellowish colouration and/or the presence of voids. Reacts mildly with cold dilute HCl. Reaction increases by heating the HCl. DOLOMITE LIMESTONE Rock is crystalline and composed of magnesium carbonate (>90%). When small chip of rock is immersed in dilute HCl, these is no immediate reaction, but a slow formation of CO_2 beads on surface of chip; reaction slowly accelerates. Rate of reaction in increased by heating HCl. DOLOMITE	Rock is crystalline, salty to taste and may be scratched with the finger nail. HALITE (rock salt) Rock is crystalline and may be scratched with the finger nail. Grains turn into a chalky white substance when burnt for a few minutes. GYPSUM Rock is crystalline: colourless to white, frequently with a bluish tinge. It is harder than gypsum and has three orthogonal cleavages. ANHYDRITE Rock is black or brownish black and has a low specific gravity (1.8—1.9). It may have a vitreous lustre and conchoidal fracture and/or breaks into pieces that are roughly cuboidal. COAL
	Medium-grained 0.06 mm	Arenaceous	Rock is composed of mainly sand sized angular mineral and rock fragments in a fine-grained matrix. TUFF		
Smooth	Fine- grained 0.002 mm	Argillaceous	Rock is composed of silt sized fragments in a fine- to very fine-grained matrix. Matrix and fragments may not always be distinguished in the hand specimen. FINE-GRAINED TUFF		Rock is black or various shades of grey and breaks with a characteristic conchoidal fracture affording sharp cutting edges. The rock cannot be scratched with a penknife. FLINT Rock has similar appearance and hardness as flint but breaks with a more or less flat fracture. CHERT
	Very fine-grained		VERY FINE-GRAINED TUFF		

Sedimentary rocks containing at least 50% volcanic material although detrital in nature are generally treated separately. These are referred to as pyroclastic sediments and are subdivided on the basis of grain size. It is sometimes difficult to recognize pyroclastic rocks without the aid of a thin section. Medium-grained pyroclastic rocks (tuffs) are generally characterized by angular grains (medium to coarse sand size) which resemble crystals in a fine-grained matrix.

The chemical and organic sediments are subdivided on the basis of mineral or organic content and texture. Rocks formed by chemical precipitation generally have a crystalline texture. Most chemically precipitated rocks are water soluble and weaker than igneous or metamorphic rocks.

Carbonate rocks (greater than 50% carbonate content) with a crystalline texture are termed limestone or dolomite according to their magnesium content. Both the crystalline and fragmental varieties of carbonate rocks can be identified by their reaction with hydrochloric acid (HCl).

An elementary discussion of sedimentary rocks is given by Blyth and deFreitas (1984), McLean and Gribble (1985) and Goodman (1993). For a more detailed discussion, see Greensmith (1978) and Tucker (1982).

Metamorphic rocks

Metamorphic rocks are derived from pre-existing rocks of all types in response to marked changes in temperature or stress or both. An increase in temperature or pressure can cause the formation of new minerals and the partial or complete recrystallization of the parent rock with the development of new textures. Three broad types of metamorphism can be distinguished.

1. *Dynamic metamorphism*. This type of metamorphism generates intense stresses locally, which tend to deform, fracture and pulverize the rock.
2. *Regional metamorphism*. This type of metamorphism affects an extensive area through an increase in pressure and temperature.
3. *Contact metamorphism*. This type of metamorphism results from the heating of the host rock in the vicinity of a body of intruded igneous magma.

In all of these groups it is possible to distinguish various intensities of metamorphism (termed metamorphic grade) based on mineral assemblages. The metamorphic minerals produced, however, depend to a large extent on the chemical composition of the original rock and are often difficult to identify in the hand specimen. Thus mineral composition is not an essential factor in a simple classification of metamorphic rocks. In many cases metamorphic rocks are deformed during recrystallization resulting in the development of characteristic and often conplex fabrics and textures. The most conspicuous fabric exhibited in the hand spec1en or field exposure is a layering (fabric) and preferred orientation of mineral gneiss within each layer (texture). This type of fabric with its characteristic texture is termed foliation or schistosity, and is most common in regional metamorphic rocks.

The complexity of metamorphic rocks is such that there no generally agreed descriptive classification, nor are there agreed definitions of such common metamorphic rock types as schist, gneiss and amphibolite. This is confusing for the geologist and even more so for the engineer.

Many metamorphic rocks retain sufficient of their primary sedimentary or igneous features to be given sedimentary or igneous names. If it is necessary to emphasize that a particular rock has undergone metamorphism, this may be done by adding the prefix '*meta-*' before the appropriate igneous or sedimentary rock name, for example, metabasalt, metaquartzite.

In many areas, however, primary igneous and sedimentary features have been completely destroyed by metamorphism. In others it is not certain whether the boundaries between different compositional types of metamorphic rock represent sedimentary bedding or not. In such cases a metamorphic rock name must be used. Fabric may be used together with grain size to produce a simple aid to naming such rocks as shown in Table 2.13.

An elementary discussion of metamorphic rocks is given by Blyth and deFreitas (1984), McLean and

Gribble (1985) and Goodman (1993), and a more detailed discussion dealing with genesis and petrography is given by Mason (1978).

Table 2.13 Classification of metamorphic rocks

Fabric Grain size	Foliated	Massive
Coarse-grained	Rock appears to be a complex intermix of metamorphic schists and gneisses and granular igneous rock. Foliations tend to be irregular and are best seen in field exposure: MIGMATITE	Rock contains randomly orientated mineral grains (fine- to coarse-grained). Foliation, if present, is poorly developed. This rock type is essentially a product of thermal metamorphism associated with igneous intrusions and is generally stronger than the parent rock: HORNFELS
	Rock contains abundant quartz and/or feldspar. Often the rock consists of alternating layers of light coloured quartz and/or feldspar with layers of dark coloured biotite and hornblende. Foliation is often best seen in field exposures: GNEISS	Rock contains more than 50% calcite (reacts violently with dilute HCl), is generally light in colour with a granular texture: MARBLE
	Rock consists mainly of large platey crystals of mica, showing a distinct subparallel or parallel preferred orientation. Foliation is well developed and often undulose: SCHIST	If the major constituent is dolomite instead of calcite (dolomite does not react immediately with dilute HCl), then the rock is termed a: DOLOMITIC MARBLE
2 mm		
Medium-grained	Rock consists of medium- to fine grained platey, prismatic or needle-like minerals with a preferred orientation. Foliation often slightly nodulose due to isolated larger crystals which give rise to a spotted appearance: PHYLLITE	Rock is medium to coarse-grained with a granular texture and is often banded. This rock type is associated with regional metamorphism: GRANULITE
0.06 mm		
Fine-grained	Rock consists of very fine grains (individual grains cannot be recognized in hand specimen) with a preferred orientation such that the rock splits easily into thin plates: SLATE	Rock consists mainly of quartz (95%) grains which are generally randomly orientated giving rise to a granular texture: QUARTZITE (METAQUARTZITE)

DESCRIPTION OF ROCK MATERIAL

In the UK various methods have been proposed for the description of intact rock. Table 2.14 compares the principal methods.

Table 2.14 Comparison of the principal methods of description of intact rock

Description	Anon (1970)	Anon (1972)	BS 5930 (1981)
Colour	•	•	•
Grain size	•	•	•
Texture and structure	•	•	•
Weathered state	•	•	•
Alteration state	•	•	
Cementation state	•		
Minor lithological characteristics	•	•	
Mineral type	•		
ROCK NAME	•	•	•
Estimated strength	•	•	•
Other characteristics and properties		•	•

Over the years the number of terms addressed specifically in the description of rock material has reduced in an effort to simplify the description process and to make it more concise. However many geologists and engineers still make reference to such factors as cementation state, minor lithological characteristics and mineral type where relevant in their rock descriptions.

The following scheme for systematic rock material description is commonly used in practice:

(a) colour;
(b) grain size;
(c) texture fabric and structure;
(d) weathered state and alteration state where relevant;
(e) minor lithological characteristics, including cementation state where relevant;
(f) ROCK NAME (in capitals);
(g) estimated strength of the rock material; and
(h) other terms indicating special engineering characteristics.

An example of this system in use is shown in Table 2.15.

Table 2.15 Example of systematic rock material description

	(i)	(ii)	(iii)
(a)	Light pinkish grey	Light yellowish brown	Light pinkish white
(b)	Coarse-grained	Fine-grained	Medium-grained
(c)	Porphyritic, massive	Thickly bedded	Foliated
(d)	Slightly weathered Slightly kaolinized	Fresh	Fresh
(e)		Weakly cemented Ferruginous	With bands of dark coloured biotite with preferred orientation
(f)	GRANITE	QUARTZ SANDSTONE	GNEISS
(g)	Very strong	Weak	Very strong
(h)	Impermeable except along joints	Porous	

The word order used in both soil and rock description should be consistent and highlight the engineering behaviour of the material. The schemes were developed at different times and by groups of people with different backgrounds. The scheme for describing soil was developed first alongside the science of soil mechanics. It represents a scheme developed by engineers for engineers. The scheme for describing rock in a systematic manner for engineering purposes, however, was developed much later but did not follow the science of rock mechanics. The resulting scheme is biased somewhat towards geology. in mitigation, this probably stems from the difficulties involved in describing a material that can take on so many different forms within an existing framework of geological terms that need to be retained for ease of communication between engineers and geologists. Apart from the potential communication problems there is a strong argument for not using geological names in rock description and classification. indeed Duncan (1969) proposed a scheme based on texture, structure, composition (calcareous or non-calcareous), colour and grain size.

As a result of the different paths taken in the development of the two schemes they are not consistent in the word order used. Table 2.16 shows a comparison between the two schemes illustrated in this book.

Classical models of soil mechanics involve the concepts of initial porosity and its subsequent modification by stress history. The engineering description of soil embodies these concepts and in addition recognizes the undrained strength of low permeability soils. The first term in the description (*consistency and relative density*) is providing information on strength (for cohesive soils) or initial porosity (for cohesionless soils). The second term (*fabric or fissuring*) provides some information on the likely engineering performance of the soil in the mass. The descriptive terms used here should highlight inhomogeneity, anisotropy and discontinuities such as fissures. The third term (*colour*) is rarely of great importance other than as a means of correlation.

Table 2.16 Comparison between soil and rock descriptions

Soil	Rock
Consistency or relative density	Colour
Fabric or fissuring	Grain size
Colour	Texture, fabric and structure
Subsidiary constituents	Weathered state and alteration state
Angularity or grading of principal soil type	Minor lithological characteristics
PRINCIPAL SOIL TYPE	ROCK NAME
More detailed comments on constituents or fabric	Estimated strength of the rock material
	Other terms indicating special engineering characteristics

In many cases, the soils will contain a mixture of particle sizes or materials which will influence their mechanical properties. Hence it is necessary to attempt to separate the subsidiary components of a soil from the principal component within the description. The name used for the soil in the description relates to the principal soil type and, in general, is based on particle size or the presence of organic material. The soil name thus provides valuable qualitative information concerning the physical and mechanical properties of the material. For example, the term 'sand' is indicative of relatively high permeability (provided the subsidiary component is not clay or silt) and relatively low compressibility, whereas the term 'clay' indicates relatively low permeability and relatively high compressibility. In granular soils (for example, sand or gravel) the soil name may be further refined by describing the shape of the particles (angularity). The object of each term within the soil description serves a specific purpose in attempting to define the engineering performance of the material as far as is possible without subjecting it to laboratory or *in situ* tests. Indeed, in some countries the engineering description of soils are used directly in geotechnical design and no provision is made for mechanical testing. The word order is necessary to provide a systematic framework for description. In summary, the components of the engineering description of soil may be considered as providing the following information:

- strength;
- variability and mass behaviour (e.g. anisotropy); and
- composition (indicative of permeability and compressibility).

For historical reasons the word order used in rock description is different from that used in the description of soil. Not only is this confusing, but the order used fails to highlight adequately the engineering properties in the same manner as for soil description.

Until recently, soil mechanics has considered strength to be of paramount importance and hence the descriptive terms relating to this property appear first in the word order. In rock engineering, rock material strength is of lesser importance than that of the rock mass. However, when describing intact rock the strength of the rock is possibly the most important parameter. In rock description the term relating to strength appears after the rock name. The first term in the word order is reserved for the least important parameter: colour.

It will be appreciated from the discussion on the geological classification of rocks that in many cases the rock name is derived from the texture, composition and grain size. It seems strange that texture and grain size feature so highly within the word order. The rock name may not be so indicative of engineering properties as a soil name is, but it generally is strongly indicative of texture and grain size. In certain cases some qualification of texture and/or grain size may be necessary in the description.

For example a sandstone may be composed predominantly of fine-grained sand particles and hence this should be stated in the description.

Fabric and structure will often play an important role in determining the engineering behaviour of the rock and thus should appear near the top of the word order.

The weathered state of a rock may be difficult or impossible to identify in a hand specimen. However the effects of weathering on the rock material will picked up in the other descriptors such as strength and colour. In many cases the most noticeable effect of weathering at the rock material scale will be discolouration and/or weakening adjacent to discontinuities. The problems involved in the description of weathering are discussed in detail later.

In the authors' opinion the word order should be changed such that it matches as far as possible that used for soil description and highlights better the more important engineering characteristics. The proposed new word order is shown below:

(a) estimated strength;
(b) fabric and structure;
(c) colour;
(d) lithological characteristics including where relevant:
  - texture,
  - grain size,
  - weathered state if known with certainty,
  - alteration state (e.g. kaolinized, hematized),
  - cementation state (e.g. weakly or strongly cemented),
  - type of cement (e.g. calcareous, ferruginous, siliceous),
  - subsidiary minerals (e.g. mica in sandstones),
  - porosity (e.g. porous or highly porous),
  - fossil content (e.g. fossiliferous or shelley);

(e) ROCK NAME;
(f) evidence of weathering (e.g. discolouration adjacent to discontinuities, including degree of penetration or loss of cement); and
(g) other terms indicating special engineering characteristics.

Here is an example of the above word order in use:

Moderately weak, bioturbated, light reddish brown, weakly cemented, calcareous fine SANDSTONE highly friable adjacent to discontinuities to a depth of 10mm.

The terms used in the conventional description of rock material are described below.

Colour

Colour is often the most noticeable feature of a rock but is possibly the most difficult to describe accurately and hence unaided assessments can be most misleading. It is normally associated with mineral composition of the constituent particles or the cementing material (in the case of sedimentary rocks) and hence should not be underrated. The colour of rock should be assessed objectively applying similar precautions to those mentioned in assessing the colour of soils. Wherever possible, colours should be compared with a standard chart, such as the rock colour chart produced by the Geological Society of America (1963) or the Munsell Soil Colour Chart (obtainable from Tintometer Ltd, Waterloo Road, Salisbury, England). Where standard charts are not available the simplified scheme proposed by the 1972 Working Party Report (Geological Society of London 1972) should be used (Table 2.17). For example, a rock colour might be described as dark greenish grey'.

Table 2.17 Rock colour

1	2	3
Light	Pinkish	Pink
Dark	Reddish	Red
	Yellowish	Yellow
	Brownish	Brown
	Olive	Olive
	Greenish	Green
	Bluish	Blue
		White
	Greyish	Grey
		Black

Grain size

Many of the common rock types are classified on the basis of grain size and hence the rock name often has an inherent grain size implication. It is recommended, however, that a descriptive grain size term should be included before the rock name. The same broad grain size ranges that are used for describing soils should be employed for rocks, as shown in Table 2.18.

Table 2.18 Grain size

Term	Particle size (mm)	Equivalent soil grade
Very coarse-grained	>60	Boulders and cobbles
Coarse-grained	2—60	Gravel
Medium-grained	0.06—2	Sand
Fine-grained	0.002—0.06	Silt
Very fine-grained	<0.002	Clay

For grains <0.06mm, grain boundaries are indistinct or below the threshold of visibility of the unaided eye.

Although it is recommended that the broad grain size bands be used (see Table 2.18), it would be more useful if the grain size subdivided further, particularly in the medium grain size (i.e. sand) range. For example, the rock name SANDSTONE implies a medium grain size but this could be qualified more by calling it a FINE SANDSTONE if the dominant grain size was in the fine sand range (0.06—0.2mm). In the above example the grain size qualifier is placed next to the rock name to avoid confusion. This approach is implied in the rock classification table (table 9) in BS 5930:1981. Of course, it can be argued that the inclusion of the grain size in the recommended form aids checking whether the correct rock name has been used.

Texture, fabric and structure

Texture may be defined as the geometrical aspect of the constituent particles or crystals together with the mutual relationship between them. In sedimentary rocks, texture refers to the size, shape and arrangement of the component mineral grains and in igneous and metamorphic rocks it deals with the crystallinity, granularity and the geometric relationships between the constituent minerals. The textural terms used by geologists are often complex and are frequently based on examination of thin sections under a microscope.

The most common textural terms are crystalline, glassy, granular or smooth. In most cases these may be sufficient for engineering use. However, these textures like grain size form the basis on which the rock has been named. The rock name therefore will suggest the texture. However variations in texture may be described using these terms.

The more complicated textural terms can be employed to a limited extent to provide a shorthand for descriptions. For example, an igneous rock may contain coarse- or medium-grained crystals in a finer-grained crystalline matrix. Such a texture could be described as porphyritic. A similar texture in a metamorphic rock would be termed porphyoblastic.

Rock fabric refers to the spatial arrangement and orientation of grains within the rock. In sedimentary rocks, fabric essentially deals with grain to grain relations, grain orientation, cementation and porosity. Fabric in igneous and other crystalline rocks refers to the pattern produced by the shapes and orientations of the crystalline and non-crystalline components of the rock.

In some cases rock fabric may not be recognized without the aid of a microscope. Examples of rocks fabric terms are homogeneous, schistose and lineated.

Structure refers to the larger scale inter-relationship of texture and fabric and is therefore often more noticeable. Common structural terms include foliated, massive, flow-banded, veined and homogeneous. In bedded sedimentary rocks, the individual beds may exhibit laminated, cross-laminated, graded, slump or bioturbated structures. The surfaces of bedding planes may be ripple-marked, sun-cracked or sole-marked.

BS 5930:1981 recommends that the descriptive terms shown in Table 2.19 be used for planar structures, such as bedding and lamination in sedimentary rocks. The terms and definitions used are based on the Engineering Group Working Party Report on the Preparation of Maps and Plans in Terms of Engineering Geology (1972).

Table 2.19 Spacing of planar structures

Term	Spacing
Very thickly bedded	>2m
Thickly bedded	600mm—2m
Medium bedded	200 mm—600 mm
Thinly bedded	60 mm—200 mm
Very thinly bedded	20mm—60mm
Laminated (sedimentary) Closely (metamorphic and igneous)	6rnm—20mm
Thinly laminated (sedimentary) Very closely (metamorphic and igneous)	<6mm

For sedimentary rocks, structures such as bedding may be described as thick beds or thickly bedded (for example thickly bedded limestone).

Weathered state

Rock material displays a wide variation in physical and mechanical properties. Some part of this variation is attributable to the different origins of the three principal groups of rocks, igneous, sedimentary and metamorphic. However, any rock which is brought into the near surface environment will be subject to structural, textural and mineralogical changes resulting from weathering which will affect its engineering properties. In general, rock material may lose strength, become more compressible and its permeability may increase or decrease. It is this near surface environment that yields the most variation in physical and mechanical properties of both rock material and the rock mass. It is also the environment in which most engineering works take place.

Weathering is that process of alteration and breakdown of rock occurring under the direct influence of the hydrosphere and the atmosphere, at or near the Earth's surface. This process results from adjustment of the rock to the stress, chemical and biological conditions of the near surface environment and comprises physical disintegration (*physical weathering*) and chemical decomposition (*chemical weathering*). Where there has been little or no transport of altered or loosened material a complete weathering profile may be present in which a residual soil grades downwards through weathered rock into unaltered ‘fresh’ rock.

The processes of physical disintegration and chemical decomposition generally act together in weathering rock. However, weathering is strongly influenced by climate (rainfall and mean temperature), often with the result that one process is predominant. For example, in hot desert regions physical disintegration will be the dominant process, whereas chemical decomposition will dominate in a humid tropical region. However, it should be pointed out that the progress of chemical decomposition usually relies on fractures formed partly as a result of physical disintegration. Similarly, fractures may develop in response to changes in volume and weakening from chemical weathering.

The extremes of weathering are unaltered fresh’ rock and residual soil. Intermediate states of weathering are difficult, if not impossible, to identify if the dominant weathering mechanism and the appearance of the end members are not known, particularly in weak rocks. The task is further complicated if the rock mass cannot be observed. In many cases engineers and geologists attempt to describe the weathered state of rock samples obtained from drillholes. A drillhole does not provide a sufficient volume of the rock mass to permit an accurate assessment of the state of weathering. Indeed, the weathered state of the rock obtained from drillholes is based almost entirely upon the condition of the rock material. In cases where physical weathering processes dominate, the rock material will appear relatively fresh even when the fracture state of the rock mass may indicate a high degree of weathering of the rock mass.

The main processes of chemical weathering all depend on the presence of water and may result in the alteration or dissolution of the component minerals grains. In the case of sedimentary rocks, the cement which binds the grains together is also prone to chemical attack. Typically, the chemical decomposition of the rock material starts at discontinuity walls and works inwards towards the centre of the intact blocks. This is often associated with discolouration penetrating the rock from the discontinuity walls. In cases were cement is removed by solution, the rock may be friable adjacent to discontinuities, and the zone of discolouration may be absent. The degree to which the discolouration or the removal of cement has penetrated the rock will indicate the degree of weathering. Of course, in cases where this zone has penetrated to the centre of blocks it may be difficult to determine whether the observed features are a product of weathering, or simply associated with the way in which the rock was formed (i.e. the rock is really fresh). Such problematic cases can only be resolved by making observations of the rock mass to define the overall weathering profile.

The effect of only slight or moderate chemical decomposition will be to influence the shear strength and compressibility of the discontinuities with little effect on the intact rock. Since in most rock masses the discontinuities control the engineering performance, the recognition of the early stages of chemical decomposition is clearly important. When the volume of chemically decomposed rock exceeds that of the fresh rock in intact blocks the rock material properties will be affected. It is likely that when the rock material is in such a highly weathered condition, the discontinuities will not have such a significant effect on the performance of the rock mass as would be the case if the rock material were fresh. This is particularly true with respect to compressibility.

In soluble rocks, chemical weathering may be recognized by the opening of discontinuity apertures and the presence of voids. Only the insoluble material is left behind. If the rock contains little or no insoluble material nothing is left behind except a void. Typically, the dissolution is associated with the passage of water through the discontinuities and hence the process results in an increase in aperture and a reduction in the degree of contact between adjacent discontinuity walls. This will affect the shear strength and compressibility characteristics of the rock mass as well as increasing the mass permeability.

Physical weathering of rock will generally cause the formation of new fractures, together with the opening of existing discontinuities. If the effects of chemical weathering are minimal the rock material will remain relatively fresh. In such cases the weathering may only be recognized from discontinuity spacing and aperture measurements. These measurements form an essential feature of rock mass descriptions. It will be the pattern of variation in spacing and aperture that will indicate the degree and stages of physical weathering. The decrease in discontinuity spacing and the general loosening of intact blocks of rock associated with this weathering process will have a significant influence on the performance of the rock mass.

From the above discussion it can be seen that the main indicators of weathering are as follows.

Chemical weathering

DECOMPOSITION

- Discolouration penetrating into the rock material from discontinuity walls.
- Formation of a zone of noticeably weaker rock (grains easily removed in the case of igneous rocks and granular sedimentary rocks or rock has become softened in the case of some mudrocks) penetrating inwards from discontinuities.

SOLUTION

- Removal of cement adjacent to discontinuity walls (in cemented sedimentary rocks).
- Widening of discontinuity apertures often with evidence of channelling.
- The presence of voids either associated with discontinuities or within the intact rock.

Mechanical weathering

- Formation of new fractures resulting in a reduction in discontinuity spacing and intact block size.
- Widening of discontinuity apertures.
- Loosening of the fracture block system.

The effects of chemical decomposition can generally be identified from intact blocks of rock seen in isolation from the rock mass. The degree of weathering may be determined from either by the amount that the zone of decomposition penetrates the rock material or from the ratio of the volume of rock to that of residual soil. In rock subject to solution or physical weathering the evidence of weathering and hence the determination of the degree of weathering can only be assessed by examination of the rock mass.

Both chemical and physical weathering processes will ultimately produce a residual soil in which the original texture, fabric and structure of the rock is destroyed. When such materials are observed they will be described as soils. They will only be recognized as the ultimate product of weathering through the establishment of a weathering profile within the rock mass.

Weathering normally takes place in a systematic fashion, such that highly weathered rock or residual soil may be found at or near the ground surface and this grades into fresh rock at depth, giving rise to a weathering profile. This simple pattern, however, may not always occur in reality due to local variations in rock type and geological structure. It is possible for weathered rock to pass laterally into unweathered rock and for discrete zones of weathered rock to exist below fresh rock. The depth of weathering is considerably variable ranging from centimetres to over 100 m.

In the schemes for the systematic description of rock material currently in use there is an expectation for the degree of weathering to be described. It is clear from the above discussion that when describing rock material from a hand specimen or from a stick of core it may be extremely difficult or impossible to make a statement about the degree of weathering. Certain readily identifiable features of weathering such as discolouration can and should be incorporated into the rock description. Other features of weathering such as loss of strength will be incorporated within the overall description without necessarily being attributed to weathering. When the descriptions are brought together in order to study the rock mass the pattern of such features will aid in the identification of a weathering profile.

Attempts have been made at developing classification schemes which allow the degree of weathering to be defined for different lithologies (Anon 1970, Anon 1977, BS 5930 1981). The early schemes (Anon 1970, for instance) were based on the chemical weathering of granitic rocks and represented a hybrid material grade and mass zonal scheme. In 1977, the Working Party of the Engineering Group of the Geological Society on the Description of Rock Masses (Geological Society of London 1977) clearly separated the description of weathering on a rock material scale and on a rock mass scale. This scheme, like the earlier ones, placed great emphasis on the weathering profiles developed on granitic rocks in tropical and sub-tropical environments. Little guidance was given for the description of rock material weathering. BS 5930:1981 provided recommendations for the description of weathering of rock material. The British Standard proposed that weathered rock material may be described or graded using four terms: *decomposed*, *disintegrated*, *fresh* and *discoloured*, but provided no guidance for determining and describing the degree of weathering. Attempts to use these schemes in the description of rock material have met with difficulty. It is the opinion of the authors that any reference to degree of weathering should be omitted from the description unless it is known with some certainty on the basis of experience and knowledge of the typical weathering profile for that rock type. For rocks weathering in conditions where physical disintegration dominates, it is unlikely that the degree of weathering may be determined from examination of rock material alone.

Clear evidence of weathering should be included in the description of rock material. For example, in cases where discontinuity walls are discoloured or weakened these features, together with the distance they have penetrated, should be included.

The Engineering Group of the Geological Society has commissioned a Working Party to study the description and classification of weathered rocks for engineering purposes. The Report of this Working Party (1995) provides a scheme for describing the state of weathering for uniform rock materials which are moderately strong or stronger in the fresh state and which show a clear gradation in engineering properties during weathering. The proposed classification scheme requires the use of appropriate index tests such as the point load test and slaking tests.

The most logical approach to the problem of classifying the degree of weathering is to describe the rock material without attempting to provide a statement on how weathered it may be, apart from commenting on the presence of discolouration, decomposition, voids and softening. Once sufficient descriptive data on the rock material and the rock mass has been acquired to establish the mechanisms and stages of weathering present, a site specific weathering classification can be easily developed to provide a consistent means of describing both the rock material and, more importantly, the rock mass.

Alteration

Alteration refers to those changes in the chemical or mineralogical composition of a rock produced by the action of hydrothermal or other fluids. A common example of this phenomenon in granite rocks is the alteration of feldspars to form kaolinite. This is termed kaolinization. Other common forms of alteration are tourmalinization, mineralization, decalcification, and dolomitization. It is difficult to distinguish between the effects of weathering and alteration in some cases. In general, weathering effects die out at depth whereas alteration effects may be significant at great depth.

The engineering characteristics of the rock material and the rock mass can be drastically changed by alteration. For example, kaolinized granite is usually considerably weaker than unaltered granite.

Dolomitization in limestone is associated with a volume change which results in the formation of cavities.

In most cases, the descriptive terms used for weathering may be used to describe alteration. It should, however, be made clear in the description that the rock material has been subject to alteration in order to make the distinction between weathering and alteration.

Minor lithological characteristics

Minor lithological characteristics refer to the cementation state and cement type together with subordinate particle size and dominant mineral composition in the case of sedimentary rocks. In the case of igneous and metamorphic rocks, it refers to dominant or unusual mineral types. This section of the rock description may be used for noting unusual or interesting lithological features that are thought to be relevant to the engineering behaviour of the rock. Simple terms should be used where possible and defined if there is any likelihood of ambiguity. Terms should be quantified wherever possible.

In cemented rocks, descriptive terms should be used to describe the state of cementation. The scheme shown in Table 2.20 is recommended.

Table 2.20 Rock cementation

Term	Definition
Indurated	Broken only with sharp pick blow, even when soaked; makes hammer ring
Strongly cemented	Cannot be abraded with thumb or broken with hands
Weakly cemented*	Pick removes material in lumps which can be abraded with thumb and broken with hands
Compact*	Requires pick for excavation; 50mm peg hard to drive more than 50—100mm

* These materials may be treated as soil.

The cementation of sedimentary rocks takes place in two ways, first, by the enlargement of mineral grains by deposition of the same mineral on each grain surface in crystallographic continuity with the parent mineral grain, and secondly by the deposition of mineral matter in the pore spaces between grains. The descriptive terms for the common types of cement are shown in Table 2.21.

Table 2.21 Common types of cementing minerals

Composition of cement	Common form of cement	Term	Identification characteristics
Silica	Quartz Chalcedony	Siliceous	Rock normally hard and does not react with dilute HCl
Iron oxide	Limonite Haematite	Ferruginous	Mineral grains often stained brown or yellowish brown
Calcium carbonate	Calcite	Calcareous	Cement will react with dilute HCl

Other common terms used to describe minor lithological characteristics include clayey, marly, silty sandy, cherty shaly, clastic, bioclastic and metamorphosed.

Rock name

Rock names should be technically correct but simple enough for general and field uses. The scheme outlined in Tables 2.11, 2.12 and 2.13 is recommended. These are based on Dearman's scheme of petrographic description (Dearman 1974) which has been adopted (with some light alterations) by the Geological Society of London's Engineering Group Working Party in its report The Description of Rock Masses for Engineering Purposes (1977). Where necessary, a full petrographic analysis can be carried out at a later stage to enable a more petrographically correct rock name to be given.

Estimated strength of rock material

It is only absolutely essential to know the strength of rock material when describing massive rocks with little or no discontinuities, since in rocks which have discontinuities the behaviour of the rock mass is largely governed by these and not the rock material. It is useful, however, to have an assessment of rock material strength in the rock description particularly for assessing the shear strength of discontinuities (Barton 1973).

When describing rock cores or rock exposures, it is normally sufficient to estimate the strength. A scheme for estimating rock material strength based on the modified scheme of Piteau (1970) is recommended by the 1977 Working Party Report (Geological Society of London 1977). This scheme is shown in Table 2.22.

Table 2.22 Rock material strength

Term	Unconfined compressive strength MN/m^2 (MPa)	Field estimation of hardness
Very strong	>100	Very hard rock — more than one blow of geological hammer required to break specimen
Strong	50—100	Hard rock — hand held specimen can be broken with single blow of geological hammer
Moderate strong	12.5—50	Soft rock — 5mm indentations with sharp end of pick
Moderately weak	5.0— 12.5	Too hard to cut by hand into a triaxial specimen
Weak	1.25—5.0	Very soft rock — material crumbles under firm blows with the sharp end of a geological pick
Very weak rock or hard soil	0.60—1.25	Brittle or tough, may be broken in the hand with difficulty
Very stiff	0.30_0.60*	Soil can be indented by the finger nail
Stiff	0.15_0.30*	Soil cannot be moulded in fingers
Firm	0.08_0.15*	Soil can be moulded only by strong pressure of fingers
Soft	0.04_0.08*	Soil easily moulded with fingers
Very soft	<0.04*	Soil exudes between fingers when squeezed in the hand

* The unconfined compressive strengths for soils given above are double the undrained shear strengths.

Where possible, strength should be estimated using a simple field test such as the Schmidt hammer rebound test (Hucka 1965; Deere and Miller 1966; Hendron 1968; Aufmuth 1974; Dearman 1974) or the Point Load Test (Franklin *et al.* 1971; Broch and Franklin 1972; Bieniawski 1973, 1975; Aufmuth 1974; ISRM 1985). Of these two field tests, the point load test is the more reliable.

DESCRIPTION OF DISCONTINUITIES

Most rock masses are fractured, and in many cases these fractures form a distinct pattern of parallel or sub-parallel sets. These fractures, which are known collectively as discontinuities, are recognized by the fact that they have little or no tensile strength. In most cases it is the discontinuities that control the engineering performance of a rock mass and not the rock material. The degree to which the discontinuities control performance is related to the relative scale of the engineering works and the spacing of the discontinuities in three dimensions. For example, in the case of a shallow foundation on rock, if the discontinuity spacing is significantly greater than the dimensions of the foundation, the performance of the foundation is likely to be relatively unaffected by the discontinuities. If however, the discontinuities have a spacing similar to or less than the foundation dimensions, then the discontinuities will play the dominant role. Another factor which will influence the performance of the rock mass is the orientation of discontinuities relative to the direction of the applied stresses. In the foundation example above, the applied stress is vertical and hence any horizontal or near horizontal discontinuities will have the greatest influence on the settlement characteristics of the foundation. The orientation of discontinuities also play an important role in controlling the stability of rock slopes.

Just as intact rocks are characterized by generic rock names, so discontinuities may be characterized by their generic type. Identification of the generic type is not always easy, even for the trained geologist. However, such a characterization may prove useful in predicting the extent and importance of a particular set of discontinuities. There are many classifications of discontinuities as to form, size and origin. This is indicative of the fact that there is no single classification system that is used in practice (Chernyshev and Dearman 1991). However two principal types of discontinuity may be readily identified:

1. discontinuities characterized by shear displacement; and
2. discontinuities characterized by very little or no shear displacement.

The most common discontinuities of type 1 are faults and shears. Faults may have relative displacements ranging from over a kilometre to less than a metre. They tend to occur in sets, often clustered within a fault zone, but may be widely spaced. Shears exhibit much smaller displacements than faults. Both faults and shears often have surfaces marked by slickensides or contain crushed rock, clay or sandy infill (gouge).

The most common types of discontinuity are characterized by very little or no shear displacement (type 2) and are generally referred to as joints. Most fractures of this type have formed in response to shear and tensile stresses associated with tectonism and hence may be related to geological structures such as folds. However, in igneous rocks fractures result from cooling and in all rock types fractures may be produced by stress relief as overburden is removed by erosion.

Discontinuities often occur in sub-parallel sets within a rock mass. It is these sets and systems that will generally control the engineering performance of the rock mass at shallow depths. These discontinuities generally include joints, cleavages, bedding planes, laminations and some tension cracks. The initial examination of a rock mass may not reveal all of the sets present, and in some cases it may not be possible to identify any sets at all. However, the systematic nature of these discontinuities permits statistical analysis (discussed later) allowing sets if present to be identified. Some discontinuities are unique and hence do not always occur in sets'. In some cases, sets may be found but are very limited in extent. Such discontinuities have to be considered individually. They include faults, shear planes, veins and some tension cracks. Although they may only affect a relatively small proportion of the rock mass they can nevertheless control engineering performance locally.

Since discontinuities play such a significant role in controlling the engineering performance of rock masses, it is essential that they are described carefully and systematically. Those parameters that can be used in some type of 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, and shear strength will be used in determining the most likely mechanisms of failure. This information together with joint water pressure and the shear strength of critical discontinuities will permit a preliminary limit equilibrium analysis to be carried out (Hoek and Bray 1981). For the purposes of a preliminary investigation these parameters can probably be estimated with reasonable accuracy from careful description of the discontinuities. Features such as roughness, wall strength, degree of weathering, type of infilling material and signs of water seepage will provide important additional data for this engineering problem.

For the case of a shallow foundation on rock, orientation, spacing and aperture of discontinuities may be used in estimating the compressibility of the rock mass. In such a case attention should be paid to the orientation of discontinuities relative to the direction of the applied foundation load. Features such as roughness, wall strength and compressibility, degree of contact, degree of weathering and the type of infilling material will provide meaningful additional data for this engineering problem (Matthews 1993).

It will be noted that in the above examples there is a high degree of overlap in terms of those features that are regarded as important. Piteau (1970, 1973) lists the discontinuity properties that have the greatest influence at the design stage as follows:

- orientation;
- size;
- frequency;
- surface geometry;
- genetic type; and
- infill material.

Suggested methods for the description of discontinuities in rock masses are given by the Engineering Group of the Geological Society Working Party Report on the description of rock masses (1977) and the International Society for Rock Mechanics Commission on Standardization of Laboratory and Field Tests (ISRM 1978). The two schemes are compared in Table 2.23.

Table 2.23 Comparison of methods for description of discontinuities

Working Party Report (1977)	ISRM (1978)
Number of discontinuity sets	Orientation
Location and orientation	Spacing
Spacing	Persistence
Aperture	Roughness
Persistence	Wall strength
Infilling	Aperture
Nature of surfaces	Filling
Additional information	Seepage
Weathered and altered state	Number of discontinuity sets Block size

It will be seen from Table 2.23 that the two schemes for the systematic description of discontinuities are very similar. The major differences are in the order in which the features are described and some of the terms used. For example the Working Party Report (1977) suggests that the weathered or altered state of the discontinuities should be described. Essentially this means ‘to what extent has the discontinuity walls undergone deterioration by chemical weathering processes?’ Such processes are likely to bring about a change in strength and compressibility of the discontinuity walls. Instead of attempting to describe weathering the ISRM scheme focuses on the effects of weathering by suggesting that the wall strength be assessed. The ISRM scheme specifically asks for the seepage characteristics of fractures to be noted where possible. This important discontinuity property is included in the Working Party Report under the heading of additional information. The only feature not common to both schemes is the description of block size which forms part of the ISRM scheme.

Both the Working Party Report (1977) and ISRM scheme provide a good general introduction to the qualitative aspects of discontinuity measurement. However, these publications are limited by the fact that they do not incorporate data processing techniques, developed in the 1980s for the elimination of sampling bias and the quantification of discontinuity characteristics. A comprehensive treatment of these aspects of discontinuity description may be found in Priest (1993).

Orientation

The orientation of discontinuities in a rock mass is of paramount importance to design in rock engineering.

The orientation of a discontinuity in space is described by dip direction (azimuth, three digits) measured clockwise in degrees from true north and by the dip of the line of steepest declination in the plane of the discontinuity measured in degrees from the horizontal (two digits), (for example: dip direction/dip (025°/52°). These measurements are normally made by means of a magnetic compass and clinometer device fitted with a spirit level (ISRM 1978). The majority of discontinuity surfaces are irregular, resulting in a significant amount of scatter of measurements being made over a small area. To reduce this scatter it is recommended that a 200mm diameter aluminium measuring plate be placed on the discontinuity surface before any measurement is made. In many cases, there may not be enough of the discontinuity surface exposed to allow the use of such a plate. If the exposure cannot be enlarged then a smaller plate must be used. A suitable combined compass and clinometer (geological compass) to which measuring plates can be attached is shown in Hoek and Bray (1981). Combined compass clinometers are available from a number of sources and range in price from about £30 to over £100. The most common type of geological compass is the Silva compass (type 15T). Some of the more expensive compass clinometers suffer from problems with the damping of the compass needle. Too much damping can lead to errors in dip direction measurements, whereas too little can make such measurements very time consuming. The process of making discontinuity orientation measurements is made more efficient by the use of a digital compass in combination with a digital clinometer. An electronic geological compass has been developed by F.W. Breithaupt and Son (Germany) which incorporates these features enabling the orientation of discontinuities to be measured and stored simply by placing the lid of the device on the discontinuity. The device has a resolution of 10 in both dip direction and dip, and is capable of storing up to 4000 measurements together with comments. These data may be transferred to a personal computer in the office for processing.

Many compasses have the capacity to correct for differences between magnetic north and true north. It is recommended that this adjustment is always set to zero; corrections can be made later during processing or plotting (Priest 1993). It should also be noted that compass needles balanced for magnetic inclination in the northern hemisphere will be severely out of balance in the southern hemisphere. Furthermore, electronic compasses set up for use in the northern hemisphere should not be used in the southern hemisphere.

Through careful use of the conventional geological compass and practice it is possible achieve a resolution of less than 30 seconds in dip and dip direction on readily accessible discontinuities (Priest 1993). However, Ewan and West (1981) conclude that different operators measuring the orientation of the same feature have a maximum error of ± 10° for dip direction and ±5° for dip angle.

Spacing

Discontinuity spacing is a fundamental measure of the degree of fracturing of a rock mass and hence it forms one of the principal parameters in the engineering classification of rock masses. In particular, for tunnelling this property has been used in the classification for support requirements (Barton *et al.*, 1974; Bieniawski 1976) and for foundation settlement predictions on rock (Ward *et al.* 1968). The spacing of adjacent discontinuities largely controls the size of individual blocks of intact rock. In exceptional cases, a close spacing may change the mode of failure of the rock mass from translational to circular. In such cases where the joints are extremely closely spaced the rock mass will tend to behave like a granular soil and joint orientation is likely to be of little consequence.

Discontinuity spacing may be considered as the distance between one discontinuity and another. More specifically ISRM (1978) defines discontinuity spacing as the perpendicular distance between adjacent discontinuities. It is easier when collecting spacing data in the field to adopt the former more general definition. For example, a random sample of discontinuity spacing values may be obtained from a linear scanline survey (described later). Such a survey provides a list of the distances along the scanline to the points where it is intersected by the discontinuities which have been sampled. Subtraction of consecutive intersection distances provides the discontinuity spacing data. Perpendicular discontinuity spacing data may be determined during data processing in the office. However it is more meaningful if such spacings are determined for discontinuities of the same type (e.g. the same discontinuity set).

Priest (1993) defines three different types of discontinuity spacings.

1. *Total spacing*: The spacing between a pair of immediately adjacent discontinuities, measured along a line of general, but specified, location and orientation.
2. *Set spacing*: The spacing between a pair of immediately adjacent discontinuities from a particular discontinuity set, measured along a line of any specified location and orientation.
3. *Normal set spacing*: The set spacing when measured along a line that is parallel to the mean normal to the set.

The mean and range of spacings between discontinuities for each set should be measured and recorded. Ideally these measurements should be made along three mutually perpendicular axes in order to allow for sampling bias. Where discontinuity sets are readily identifiable in the field the normal set spacing of each set may be recorded in terms of the maximum, minimum and modal (most frequent) or mean spacing. A comprehensive treatment of the statistical analysis of discontinuity spacing and frequency is given by Priest (1993).

Descriptive terms for discontinuity spacing given by the Engineering Group Working Party Report on the Description of Rock Masses (Geological Society of London 1977), ISRM (1978) and BS 5930:1981. These are compared in Fig. 2.3. Typically, these descriptive terms will be applied in the field to normal set spacings.

Discontinuity spacing

	< 6mm	6mm – 20mm	20mm – 60mm	60mm – 200mm	200mm – 600mm	600mm – 2000mm	2000mm – 6000mm	> 6000mm
1	Very narrow	Narrow	Moderately narrow	Moderately wide	Wide	Very wide	Extremely wide	
2	Extremely close		Very close	Close	Moderate	Wide	Very wide	Extremely wide
3	Extremely close		Very close	Close	Medium	Wide	Very wide	

1) Geological Society (1977)
2) ISRM (1978)
3) BS 5930 : 1981

Fig. 2.3 Discontinuity spacing classification schemes: (1) Geological Society (1977): (2) ISRM (1978); (3) BS 5930: 1981.

It will be seen from Fig. 2.3 that the descriptive terms used in each document are similar but refer to different spacing classes. The Geological Society classification gives more emphasis to the more closely spaced fractures, whereas the ISRM classification places most emphasis on the widely spaced fractures. The British standard follows closely that recommended by the ISRM but omits the extremely wide category.

Clearly whichever classification one chooses to use it is important to state the source of classification or give the definitions of the descriptive terms used.

Persistence

Persistence refers to the discontinuity trace length as observed in an exposure. It is one of the most important factors in discontinuity description, but unfortunately it is one of the most difficult to quantify. One of the common problems that arises is the measurement of the persistence of major joints which are continuous beyond the confines of the rock exposure. It is recommended that the maximum trace length should be measured, and comment made on the data sheet to indicate whether the total trace length is visible and whether the discontinuity terminates in solid rock or against another discontinuity. Clearly, persistence is very much scale dependent and any measurements of persistence should be accompanied by the dimensions of the exposure from which the measurements were made.

It is helpful when collecting discontinuity persistence data in the field to set up a simple classification scheme based on trace length and type of termination. The trace length used in such a classification will vary from exposure to exposure according to the extent of the exposed rock in each case. Therefore it will be necessary either to define the categories each time or express the trace lengths as a percentage of the maximum possible trace length.

Matherson (1983) considers persistence to be a fundamental feature in quantifying the relative importance of discontinuities in a rock mass. BS 5930:1981 does not place sufficient emphasis on the observation and measurement persistence.

Wall roughness

The wall roughness of a discontinuity is a potentially important component of its shear strength, particularly in the case of undisplaced and interlocked features. In terms of shear strength, the importance of wall roughness as aperture, or infilling thickness or the degree of displacement increases. In cases where adjacent walls are not fully interlocked or mated the wall roughness will directly influence the degree of contact which in turn effect the compressibility of the discontinuity.

In general, the roughness of a discontinuity can be characterized by the following.

1. *Waviness*. First order wall asperities which appear as undulations of the plane and would be unlikely to shear off during movement. This will affect the initial direction of shear displacement relative to the mean discontinuity plane.
2. *Roughness*. Second order asperities of the plane which, because they are sufficiently small, may be sheared off during movement. If the wall strength is sufficiently high to prevent damage these second order asperities will result in dilatant shear behaviour. In general this unevenness affects the shear strength that would normally be measured in a laboratory or medium scale *in situ* shear test.
3. *Condition of the walls*. Description of rock material forming discontinuity faces. Special attention should be given to weak zones in walls produced by weathering or alteration.

Waviness may be measured by means of a standard tape or rule placed on the exposed discontinuity surface in a direction normal to the trend of the waves. The orientation of the tape, together with the mean wave length and maximum amplitude should be recorded. In some cases it may be necessary to assess the waviness in three dimensions in which case a compass and disc clinometer are recommended.

Roughness may be assessed by profiling the discontinuity surface. Short profiles (<150mm) can be measured using a profiling tool which is obtainable in most DIY stores. Longer profiles may be measured using a 2 m rule as described by ISRM (1978).

A number of quantitative techniques for measuring waviness and roughness are described in detail by ISRM (1978). No guidance on the measurement of wall roughness is given in BS 5930:1981.

Quantitative techniques for assessing roughness can be time consuming, and for preliminary rock mass surveys a qualitative assessment making use of simple descriptive terms should be employed. ISRM (1978) recommends a nine point visual classification shown in Table 2.24 which is based on two scales of observation: small scale (several centimetres); and intermediate scale (several metres).

Table 2.24 Roughness categories

Category	Degree of roughness
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.

It should be pointed out that such a scheme for describing roughness is meaningful only when the direction of irregularities in the surface is in the least favourable direction to resist sliding. It is also necessary to specify the trend of the lineation on the surface of the discontinuity in relation to the direction of shearing. Furthermore, it is recommended that at each site where this scheme is used, typical examples of each category should be identified and photographed to maintain uniformity of assessment and hence make the scheme more objective.

Wall strength

Wall strength refers to the equivalent compression strength of the adjacent walls of a discontinuity. This may be lower than the intact strength of the rock owing to weathering or alteration of the walls. The relatively thin 'skin' of wall rock that affects shear strength and compressibility can be tested by means of simple index tests. The apparent uniaxial compressive strength can be estimated from Schmidt hammer tests (Barton 1973) and from scratch and geological hammer tests, since the latter have been roughly calibrated against a large body of data. It is recommended that such tests be carried out on freshly broken rock surfaces such that the estimated wall strength may be directly compared with that of the intact rock. It is likely that the intact strength may be measured in the laboratory as part of the investigation and this will provide a means of calibrating these somewhat crude field measurements. The descriptive terms used for intact strength discussed earlier may be applied to the description of wall strength.

Wall strength may also be assessed in terms of weathering grade. ISRM (1978) recommends a set of descriptive terms for both the discontinuities and the rock mass as a whole. The same scheme has been adopted by BS 5930:1981 for the general description of weathering (or alteration) of the rock material and the rock mass.

Aperture

Aperture is the perpendicular distance separating the adjacent rock walls of an open discontinuity, in which the intervening space is air or water filled. Discontinuities that have been filled (for example, with clay) also come under this category if the filling material has been washed out locally.

Large apertures may result from shear displacement of discontinuities having a high degree of roughness and waviness, from tensile opening resulting from stress relief, from outwash 'and dissolution. Steep or vertical discontinuities that have opened in tension as a result of valley formation or glacial retreat may have extremely wide apertures measurable in tens of centimetres.

In most sub-surface rock masses, apertures may be closed (i.e. <0.5mm). Unless discontinuities are exceptionally smooth and planar it will not be of great significance to shear strength that a 'closed' feature is 0.1mm wide or 1.0 mm wide. Such a range of widths however may have a greater significance with respect to the compressibility of the rock mass. Where compressibility is concerned it is important to describe the degree of contact across adjacent rock walls in addition to the aperture observations.

Large apertures may be measured with a tape of suitable length. The measurement of small apertures may require a feeler gauge. Details of measurement techniques may be found in ISRM (1978). The descriptive terms recommended by ISRM (1978) are given in Table 2.25.

Table 2.25 Discontinuity apertures

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

Filling

Filling refers to material that separates the adjacent rock walls of a discontinuity and that is usually weaker than the parent rock. Typical filling materials are sand, silt, clay, breccia, gouge and mylonite. Filling may include thin mineral coatings and healed discontinuities. Mineral coatings such as chlorite can result in a significant reduction in shearing resistance of discontinuities (Hencher and Richards 1989). In general, if the filling is weaker and more compressible than the parent rock its presence may have a significant effect on the engineering performance of the rock mass. The drainage characteristics of the filling material will not only affect the hydraulic conductivity of the rock mass but also the long- and short-term mechanical behaviour of the discontinuities since the infill may behave as a soil.

ISRM (1978) suggests that the principal factors affecting the physical behaviour of infilled discontinuities are as follows:

1. mineralogy of filling material;
2. grading or particle size;
3. overconsolidation ratio (OCR);
4. water content and permeability;
5. previous shear displacement;

6. wall roughness;
7. width of infill; and
8. fracturing or crushing of wall rock.

If the thickness of the infill exceeds the maximum amplitude of the roughness the properties of the infill will control the mechanical behaviour of the discontinuity. Clearly the wall roughness and the thickness or width of infill must be recorded in the field. An engineering description of the infill material should be made in the field and suitable samples taken for laboratory tests. The infill should be carefully inspected in the field to see whether there is any evidence of previous movement (for example, slickensides) since this is likely to reduce the shearing resistance of the fracture significantly.

Seepage

Water seepage through rock masses results mainly from flow through discontinuities ('secondary permeability') unless the rock material is sufficiently permeable such that it accounts for a significant proportion of the flow. Generally it should be noted whether a discontinuity is dry, damp or wet or has water flowing continuously from it. In the latter case the rate of flow should be estimated. Of course such observations are dependent upon the position of the water table and the prevailing weather conditions. It is important to note whether flow is associated with a particular set of discontinuities. ISRM (1978) gives a set of descriptive terms that may be applied to seepage observations for filled and unfilled discontinuities.

Number of sets

The appearance of the rock mass together with its mechanical behaviour will be strongly influenced by the number of sets of discontinuities that intersect one another. The appearance of the rock mass is affected since the number of sets tends to control the degree of overbreak in excavations. The number of sets also affects the degree to which the rock mass can deform without failure of intact rock. In tunnelling, three or more sets will generally result in a three-dimensional block structure.

A number of sets may be identified by direct observation of the exposure. However the total number of sets present in the rock mass is normally determined from a statistical analysis of the discontinuity orientation data (Matherson 1983; Priest 1985; 1993).

The number of joint sets comprising the intersecting joint system. The rock mass may be further subdivided by individual discontinuities such as faults.

Block size

Block size is an important indicator of rock mass behaviour. Rock masses comprising relatively large blocks tend to be less deformable than those with small blocks. In the case of underground excavations such rock masses generally develop favourable arching and interlocking. In the case of slopes, a small block size may result in ravelling or circular failure brought about by the rock mass behaving like a granular soil.

The block size is determined from the discontinuity spacing, number of sets and persistence. The number of sets and the orientation of discontinuities will determine the shape of the resulting blocks. However, since natural fractures are seldom consistently parallel regular geometric shapes such as cubes, rhombohedrons and tetrahedrons rarely occur.

BS 5930:1981 recommends the descriptive terms for block size and shape given in Table 2.26. A more quantitative approach to block size description is given by ISRM (1978).

Table 2.26 Block size and shape

First term (size)	Maximum dimension
Very large	>2 m
Large	600 mm—2 m
Medium	200 mm—600 mm
Small	60 mm—200 mm
Very small	<60 mm
Second term (shape)	**Nature of block**
Blocky	Equidimensional
Tabular	One dimension considerably smaller than the other two
Columnar	One dimension considerably larger than the other two

Other descriptive terms which give an impression of the block size and shape include:

Massive	Few fractures or very wide spacing
Irregular	Wide variations of block size and shape
Crushed	Heavily jointed to give medium gravel size lumps of rock.

METHODS OF COLLECTING DISCONTINUITY DATA

The method used in collecting discontinuity data will depend largely upon the engineering application and the degree of access to the rock mass. In some cases, for example tunnelling, discontinuity data are required at some depth below the ground surface. Surface exposures may be available but may not be representative of the rock mass at the depth of interest, owing to weathering agencies (for example, stress relief causing reduction in joint spacing and an increasing in aperture). Access to the rock mass at the depth of interest can only be achieved through the use of trial adits, shafts or drillholes. Trial adits and shafts are expensive and hence in the majority of cases the preferred method of access is by drillholes. In other cases, for example shallow foundations and rock slope design, the necessary rock mass information may be obtained from surface exposures if available. Drillhole information may be used to supplement the data obtained from surface exposures. Where surface exposures are not available, or are considered to be unrepresentative of the rock, mass drillholes alone may be the only source of data. Table 2.27 shows how the quality of discontinuity data is affected by the type of access to the rock mass and the type of survey method used.

It may be seen from Table 2.27 that access to the rock mass via a drillhole suffers from a number of disadvantages. First, a drillhole permits only a small volume of the rock mass to be viewed such that the persistence of discontinuities cannot be adequately assessed. Secondly the orientation of the core must be known before any fracture orientation measurements can be made. Also, drillholes are prone to directional biasing of discontinuity data unless they are drilled with different orientations. For example, if only vertical drillholes are employed any vertical or near vertical sets of discontinuities may be missed altogether or a false impression may be given with respect to the frequency of these fractures. The only way to overcome this problem is to drill inclined holes at a number of different orientations. Terzaghi (1965) and Priest (1993) discuss methods of dealing with directional biasing and Priest (1985) discusses methods of analysing orientated core.

It is impossible to measure aperture from drillhole core since it is inevitable that the sticks of core will have moved relative to one another during and after sampling. The only way of measuring aperture in this case is by inspection of the drillhole wall. This is achieved using a borehole impression packer or a borehole television camera. The borehole impression packer has been used with success in hard rocks such as granite. In weak rocks however there is a tendency for the borehole wall to be eroded by the cuttings as they are brought to the surface. This is particularly so in the chalk where small fragments of flint can be very effective in eroding the drillhole wall, making the interpretation of the impression packer data very difficult.

Table 2.27 Quality of information from different types of discontinuity survey and access to the rock mass (based on Geological Society of London Working Party Report on the Description of Rock Masses (1977))

Type of information	Direct measurement (surface exposure, trial adit or shaft)	Surface photography	Drillhole core	Orientated drillhole core	Drillhole camera	Drillhole impression packer	Geophysics acoustic methods
Location	Good	Good	Good	Good	Good	Good	Medium
Type of discontinuity	Good	Medium	Good	Good	Good	Poor	Poor
Description of rock material	Good	Poor	Good	Good	Poor	None	None
Orientation: dip	Good	Medium	Medium/Poor	Good	Poor	Good	Poor
Orientation: dip direction	Good	Medium	Poor	Medium	Medium	Medium	Poor
Spacing	Good	Good	Medium	Medium	Medium	Medium	Poor
Persistence	Good	Good	Poor	Poor	Poor	Poor	Poor
Wall roughness: waviness	Good	Medium/Poor	Poor	Poor	Poor	Poor	Poor
Wall roughness: roughness	Good	Medium/Poor	Medium	Medium	Poor	Poor	Poor
Wall strength	Good	None	Medium	Medium	None	None	None
Aperture	Good	Poor	Poor	Poor	Medium	Medium	Medium/Poor
Infill: nature	Good	Poor	Medium	Medium	Poor	Poor	None
Infill: thickness	Good	Poor	Medium/Poor	Medium/Poor	Medium	Poor	Poor
Seepage	Good	Medium	None	None	Medium	None	Poor
Number of sets	Good	Good/Medium	Poor	Medium	Medium	Medium	None
Block size	Good	Good/Medium	Poor	Medium	Medium	Medium	Poor

Ranking: Good feature measured reliability;
Medium feature measured but not easily and often with poor reliability;
Poor feature difficult to measure, often measurement is inferred;
None impossible to identify feature.

Discontinuity infill may be washed out or contaminated by the drilling fluid such that it becomes difficult to assess its thickness or properties adequately. Mineral coatings on joint walls may be observed in core samples. However, it is impossible to assess the degree of coverage from such a small sample.

Adits, shafts and surface exposures can offer a much larger expanse of rock for examination and permit direct observation and measurement of discontinuities. However, they can be just as prone to directional biasing as the drillhole if only a single orientation of adit or exposed face is available. These forms of access clearly have distinct advantages over the drillhole, but the trial adit or shaft may prove too expensive and the surface exposure may be unsuitable due to inadequate size. Despite the obvious disadvantages of the drillhole, the fracture state of the rock mass may be determined in sufficient detail to permit classification of the rock mass. Where surface exposures have revealed one or more discontinuity sets drillholes and drillhole core may be used effectively to check whether these persist at depth.

DISCONTINUITY SURVEYS

The rock mass contains a considerable amount of geometrical information which must be collected, filtered and interpreted. Clearly an irregular highly fractured rock face presents a somewhat daunting challenge to anyone who wishes to quantify the rock structure or discontinuity network in an unbiased manner. It is important, therefore, to ensure that measurement systems are based upon objective but flexible sampling strategies linked to rigorous data analysis based upon the principles of geometrical probability and statistics (Priest 1993). Typically between 1000 and 2000 discontinuities should be sampled to provide an adequate characterization of a site (Priest and Hudson 1976). This number is generally made up from samples between 150 and 350 discontinuities taken at between 5 and 15 sample locations chosen to represent the main zones based on geological structure and lithology. In some cases the extent of the site or the exposed rock makes such large numbers of measurements impractical or impossible. In such cases the minimum sample of 200 discontinuities should be taken.

The method of collecting discontinuity data will vary according to the type of access to the rock mass. The two broad sampling strategies that can be adopted involve either the logging of drillhole core or the examination of an exposed rock face.

When using drillholes, a detailed fracture log of the core is required together with an inspection of the drillhole wall. In the case of exposed rock faces above or below ground the most widely used sampling methods include scanline sampling (ISRM 1978; Priest and Hudson 1981; Priest 1993) and window sampling (PahI 1981; Priest 1993).

Fracture logging of drillhole core

A common problem in logging fractures in core samples or in a rock face is identification of artificial fractures resulting from the drilling process or by the creation of the face (blasting and stress relief). These fractures are normally excluded from the log, unless a conscious decision is made to the contrary which should be clearly stated on the log. A degree of judgement is therefore required. Artificial and natural fractures can often be distinguished from each other by observing the freshness, brightness, staining and erosion of the fracture surface. For example, in the chalk natural fractures often exhibit manganese spots or dendritic patterns and relatively smooth surfaces, whereas artificial fractures are clean and rough.

Every natural fracture which cuts the core should be described in the following manner.

1. The position of the fracture in the core sample should be recorded. A pictorial log of the fractures cutting the drillhole may be made from this information.
2. The angle the fracture makes with the core axis should be noted. Where the orientation of the core is known the dip and dip direction of the fracture should be determined.
3. The roughness of the fracture surfaces, should be noted.
4. If any infill is present, its thickness and nature should be described.
5. The presence of any mineral coatings should be noted.
6. Where possible or practical the compressive strength of the fracture surface should be determined using a Schmidt hammer. A qualitative assessment of the wall strength may be made by indenting the wall and the side of the core using the point of a knife, pick or other sharp implement.

The average spacing of discontinuity sets identified from the fracture log may be determined from the recorded positions of the relevant fractures. The general fracture state of the rock mass may be assessed from the determination of the total and solid core recovery, the fracture index, Rock Quality Designation (RQD) and Lithological Quality Designation (LQD). These parameters are defined in the section on Logging Rock Cores.

Scanline sampling

Although many different techniques have been described for sampling discontinuities in rock exposures (Muller 1959; Pacher 1959; Da Silveria *et al.* 1966; Knill 1971) the line or scanline approach is preferred (Piteau 1970; Broadbent and Rippere 1970) on the basis that it is indiscriminate (all discontinuities whether large or small should be recorded) and provides more detail on discontinuity spacing (Priest and Hudson 1976, 1981) and attitude than other methods. However there is currently no universally accepted standard for scanline sampling.

In practice, a scanline survey is carried out by fixing a measuring tape to the rock face by short lengths of wire attached to masonry nails hammered into the rock. The nails should be spaced at approximately 3 m intervals along the tape which must be kept as taut and as straight as possible. The face orientation and the scanline orientation should be recorded along with other information such as the location, date, and the name of the surveyor. Where practicable the face and scanline, including a scale and appropriate label, should be photographed before commencing the sampling process. In cases where the face is irregular it will be necessary to take photographs from several viewpoints. A simple way to provide a scale is to attach clearly visible markers at 1 m intervals along the length of the scanline. Care should be taken to minimize distortion of the face on the photographs.

Once the scanline is established the surveyor works systematically along the tape recording the position and condition of every discontinuity that intersects it. The features that are commonly recorded include the following.

1. *Intersection distance*. This is the distance in metres (rounded to the nearest cm) along the scanline to the intersection point with the discontinuity. Where the face is irregular it will be necessary to project the plane of fractures not in contact with the tape on to the tape such that the position of such fractures can be accurately recorded. In highly irregular faces this method can lead to significant errors in the determination of joint spacing. Ideally a clean, approximately planar rock face should be selected for scanline sampling.
2. *Orientation*. This is the dip direction and dip of the discontinuities.
3. *Semi-trace length*. This is the distance from the intersection point on the scanline to the end of the discontinuity trace. The distance may be measured directly, estimated by eye or scaled from a photograph of the rock face, when it becomes available. There will be two semi-trace lengths associated with each discontinuity: one above and one below for a horizontal scanline; one to the left and one to the right for an inclined or vertical scanline.
4. *Termination*. It is helpful to record the nature of the termination of each semi- trace. The scheme recommended by ISRM (1978) has proved to be adequate:
   ‘I’ or 1 Discontinuity trace terminates in intact rock material
   ‘A’ or 2 Discontinuity trace terminates at another discontinuity
   ‘O’ or 3 Termination obscured or trace extends beyond the limits of the exposure.
5. *Roughness*. A profile of the discontinuity surface roughness may be made in the manner described earlier or the Joint Roughness Coefficient (JRC) (Barton 1973) may be estimated visually.
6. *Curvature*. This refers to surface irregularities with a wavelength greater than about 100 mm. This can be determined by measuring offsets at 100mm intervals along a straight base line, then digitizing and quantifying the resulting profile (ISRM 1978). This can be assessed visually using the classification scheme given in Table 2.28.

Other features such as type of discontinuity, nature of infill, aperture, water flow, slickensides are generally reported in a comments column on the logging sheets.

Further scanlines should be set up on a second rock face, approximately at right angles to the first, to minimize the orientation sampling bias. Errors may arise from sampling a single line since those discontinuity sets which have an orientation similar to that of the face and those whose traces are nearly parallel to the scanline are likely to be missed in the survey as a result of directional or orientation sampling bias. Corrections have been devised to compensate for directional bias (Terzaghi

1965; Robertson 1970) but these will not aid the identification of joints sets which intersect the scanline at low angles (<10°).

Table 2.28 Roughness categories

Category	Degree of roughness
1	Polished
2	Slickensided
3	Smooth
4	Rough
5	Defined ridges
6	Small steps
7	Very rough

The length of the scanline should be at least fifty times the mean discontinuity spacing (Priest and Hudson 1976) in order to estimate the frequency of discontinuities to a reasonable degree of precision. Priest (1993) recommends that the scanline should contain between 150 and 350 discontinuities, of which about 50% should have at least one end visible. These criteria have implications on the minimum size of exposed rock face that can be representatively sampled. Often a compromise must be sought between these criteria owing to size of face or restrictions in the access to parts of the face.

West (1979) investigated the reproducibility of measuring joint frequency in the Lower Chalk. The position along a measuring tape of all joints intersected by the scanline were recorded by six observers. Very small fractures (less than about 0.1 m in length), shattered rock exhibiting fine crazing and rubble or screen zones were disregarded. Different observers produced different joint frequency diagrams, indicating the subjective nature of the method of measurement used. The element of judgement required in deciding which fractures to ignore is considered to be the primary cause of the differences in the records. Ewan *et al.* (1981) carried out a similar study in limestone, sandstone and mudstone exposed in the Kielder Aqueduct Tunnels. They found that the number of joints recorded could vary by up to a factor of four for different observers measuring the same scanline. However, in the case of this study, the 10 m scanline lengths used were typically less than the minimum recommended by Priest and Hudson (1976) (i.e. 50 times the mean joint spacing) which may have contributed to the variation in joint count between observers. Clearly it is necessary to reduce the subjective element of joint measurement by measuring all fractures that cross the scanline. However, this may prove to be too time consuming and indeed unrepresentative fractures such as those induced by the excavation method would be included. The subjectivity cannot therefore be eliminated, only minimized.

Window sampling

Window sampling provides an area based alternative to the linear sampling techniques outlined above which reduces the sampling biases for discontinuity orientation and size. The preliminaries and measurement techniques are essentially the same as for scanline sampling except that all discontinuity traces which across a defined area of the rock face are measured.

The sampling window may be defined by setting up a rectangle of measuring tapes pinned to the rock face. In order to minimize sampling bias effects it is recommended that the window should be as large as possible, with each side of a length such that it intersects between 30 and 100 discontinuities (Priest 1993). Where possible, two windows of similar size should be set up on mutually perpendicular faces.

In general, areal sampling provides a poor framework within which to collect orientation, frequency and surface geometry data for individual discontinuities. The window is likely to contain a large number of relatively small discontinuities, making it difficult to keep track of which discontinuities have been measured. The process of window sampling is generally more laborious than scanline sampling when attempting to apply the same rigorous sampling regime.

Face sampling

At the preliminary stage of an investigation it is often necessary simply to establish the number of discontinuity sets, the average orientation of each set and the relative importance of each set. Matherson (1983) suggests that for simple rock slope stability assessments at the preliminary site investigation phase it is sufficient to observe persistence, aperture and infilling in addition to orientation. In such cases it may be expedient to adopt a sampling strategy that is less rigorous than those outlined above.

Face sampling involves recording the orientation, persistence, roughness, aperture, filling and seepage of a representative number of discontinuities in the exposed face. A suitable geological compass should be used for the orientation measurements and a suitable classification or coding system should be used for the other parameters. The size of the sample must be large enough to ensure statistical reliability. A minimum of 200 measurements per face is recommended by the Engineering Group Working Party Report, *The Description of Rock Masses for Engineering Purposes* (Geological Society of London 1977) whereas a minimum of 150 measurements is recommended by ISRM (1978). Such large sample sizes may preclude the measurement of all the parameters listed above for each fracture. An evaluation of the importance of each discontinuity however should be made.

Face sampling is generally non-systematic and tends to concentrate on discontinuity orientation which is adequate for preliminary investigations, where the information should be kept simple. Discontinuity spacing cannot be readily determined unless sets are recognizable in the face or scanline sampling is carried out. The data from the face survey may be easily analysed by hand and can be used to provide a rapid assessment of stability of rock cuts. It should be pointed out that the data from a face survey alone are insufficient for rigorous stability analysis.

The grouping of data collected from a very large exposure or from a number of different locations may obscure discontinuity patterns. Ideally the area studied should be divided into units, domains or structural regions (Piteau 1973). Data from each face should be assessed separately. If it can be established that each face shows a similar discontinuity pattern it may then be possible to group the data from a number of faces. If different rock types are present in the survey area the discontinuities observed in each type should be placed in separate groups. Collection of data in domains allows individual or grouped assessment. This is likely to be of major importance where alignment, design or rock type change along a proposed route.

PRESENTATION OF DISCONTINUITY DATA

Discontinuity data should be presented in a form that allows ease of assimilation and is amenable to rapid assessment. Discontinuities may be shown on maps, scale drawings of exposures or block diagrams which may be used to indicate the spatial distribution and interrelationships of these features. Such methods, although very useful, do not allow the quantitative assessment of orientation and spacing which are perhaps the most important aspects of discontinuities within a rock mass. In many cases, it is not possible to recognize all the discontinuity sets or assess the dominant orientation of some discontinuity sets in the field. Such factors may only be assessed by statistical analysis of discontinuity orientation data.

Discontinuity orientation data are easier to visualize and analyse if presented graphically. The simplest form is a rose diagram, which shows the frequency of fractures as a function of their dip direction, but says nothing about their dip magnitude. It is therefore difficult to identify joint sets using this approach. The most commonly used method of presenting orientation data is the hemispherical projection. This method allows the distribution of dip and dip direction to be examined simultaneously and provides a rapid visual assessment of the data as well as being readily amenable to statistical analysis. Although this method is used extensively by geologists, it is little understood by engineers, since it bears no recognizable relationship to more conventional engineering drawing methods. The basis of the method and its classic geological applications are described by Phillips (1971). Rock engineering applications are described in detail by Goodman (1976), Hoek and Brown (1980), Priest (1980, 1985, 1993), Hoek and Bray (1981) and Matherson (1983).

Hemispherical projection methods

The principle of hemispherical projection methods is that the orientation of a line in three-dimensional space is uniquely represented by a point within a two-dimensional area. Hemispherical projections may be divided into the following two main types.

1. *The equal-angle projection*. This projection accurately preserves the angular relationships between features. The data are plotted on an equatorial equal- angle net (Fig. 2.4a).
2. *The equal-area projection*. This projection preserves the spatial distribution of features. The data are plotted on an equatorial equal-area net (Fig. 2.4b).

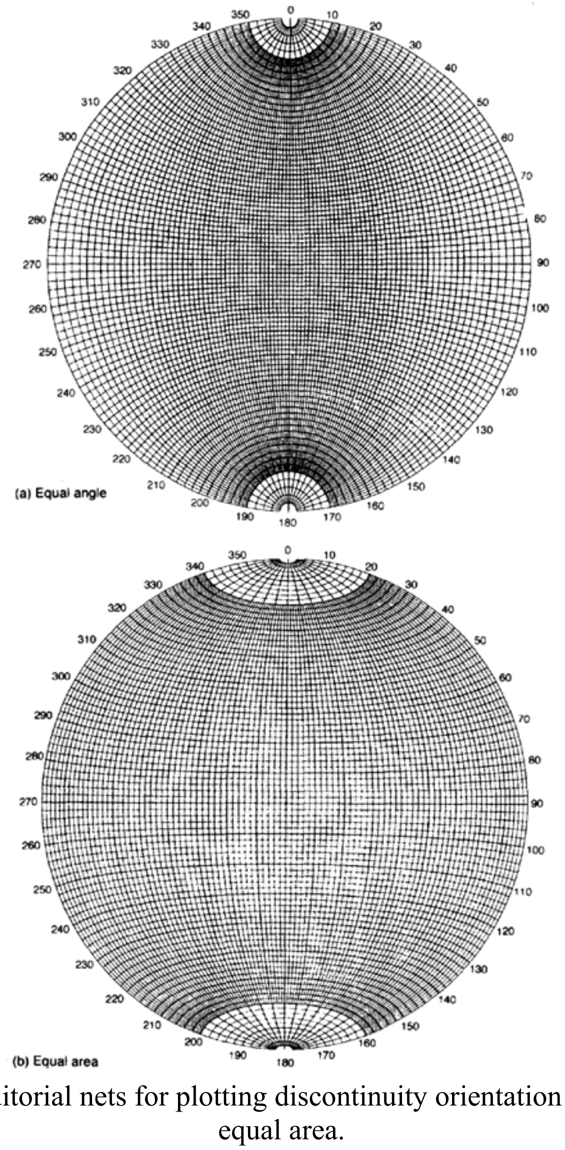


Fig. 2.4 Examples of equitorial nets for plotting discontinuity orientation data: (a) equal angle; (b) equal area.

It is important, when presenting orientation data for statistical analysis, that both the angular relationships and the spatial distribution of the discontinuities are accurately represented. Clearly this is not possible using a single hemispherical projection. Furthermore, not all hemispherical projections are suitable for statistical analysis. The equal-area projection permits the assessment of statistical distributions whilst still permitting planes and lines to be plotted, but with reduced accuracy, and hence is used for presenting discontinuity data in rock engineering. The principles of the equal- area hemispherical projection are shown in Fig. 2.5. This type of projection, like other types of hemispherical projection, is based upon a reference sphere that is free to move in space, but not free to rotate. Thus, all the discontinuities within an exposure can be represented in the same sphere in terms of orientation, independent of their position in space. Each discontinuity will cut the sphere in a similar manner to that shown in Fig. 2.5. In the equal-area projection the trace of the discontinuity on the lower hemisphere can be projected on to a planar surface directly below. The projected trace appears as a great circle. The distance from the centre of the projection to the great circle along the direction of dip is related to the dip of the discontinuity by the expression:

$$AX = 2R\cos\frac{90+\theta}{2} \tag{2.1}$$

where R radius of the reference sphere, and θ dip of discontinuity. When the plane is horizontal (i.e. θ = 0°), AX= 2Rcos45° = R√2. This means that the radius of the resultant projection is larger, by a factor √2, than the radius of the reference hemisphere. The point X is, therefore transferred to point X', a distance OX' from the centre of the plane of projection, by letting OX` = AX/√2. Hence:

$$OX' = R\sqrt{2}\cos\frac{90+\theta}{2} \tag{2.2}$$

Priest (1985) described the process of equal-area projection as like 'peeling the skin' off the lower reference hemisphere, flattening it out and then shrinking it to a circle of radius R.

Discontinuities may be represented as great circles projected from either the lower or upper hemisphere giving rise to lower and upper hemisphere projections respectively. Lower hemisphere projections are most commonly used for assessing discontinuity data. In practice, the great circle representing a discontinuity is plotted directly on to a Lambert or Schmidt net. Examples of these nets are given in Hoek and Bray (1981) and Priest (1985). If 200 measurements of dip and dip direction are made for each locality as recommended, the data are not going to be clearly presented as 200 great circles.

Furthermore, the assessment of statistical distribution is not possible using great circles. Every plane, however, can be represented on the hemispherical projection by a discrete point which is the projection of a line that intersects the plane of the discontinuity at right angles (Fig. 2.5). This line is termed a pole and is unique for each plane. Projections of poles will always be offset from the corresponding great circle by the radius of the reference hemisphere, along the diagonal representing the direction of dip.

An example of a hemispherical projection of poles representing discontinuities intersecting a number of scanlines is shown in Fig. 2.6. The poles shown in Fig. 2.6 have been classified according to persistence using symbols of different sizes. A cluster of large circles would indicate a set of persistent joints in Fig. 2.6. A single persistent discontinuity, however, can be just as important in rock engineering terms as a cluster of less persistent discontinuities. It is therefore useful to indicate persistence of fractures on the hemispherical projection. The manner in which this has been done in Fig. 2.6 does not permit an accurate statistical analysis of the data directly from this plot. For statistical analysis of poles they must be plotted as points.

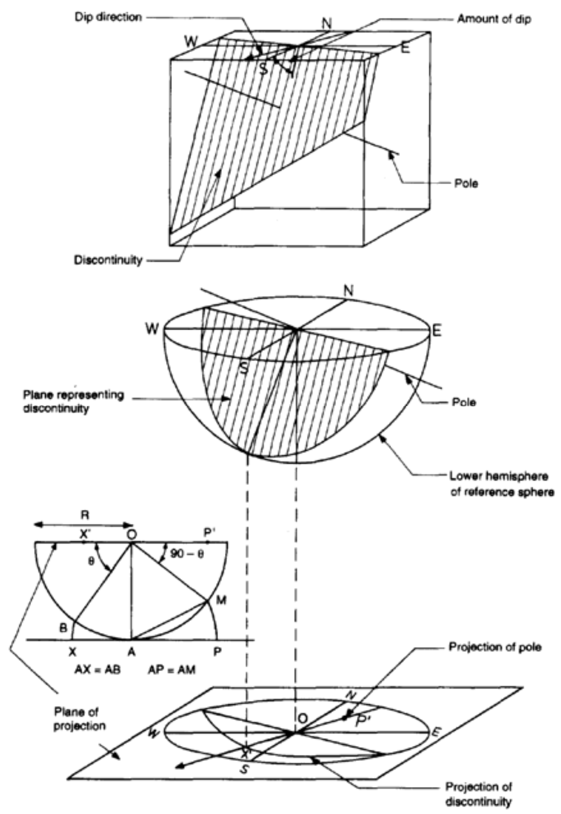


Fig. 2.5 Principles of equal area stereographic projection.

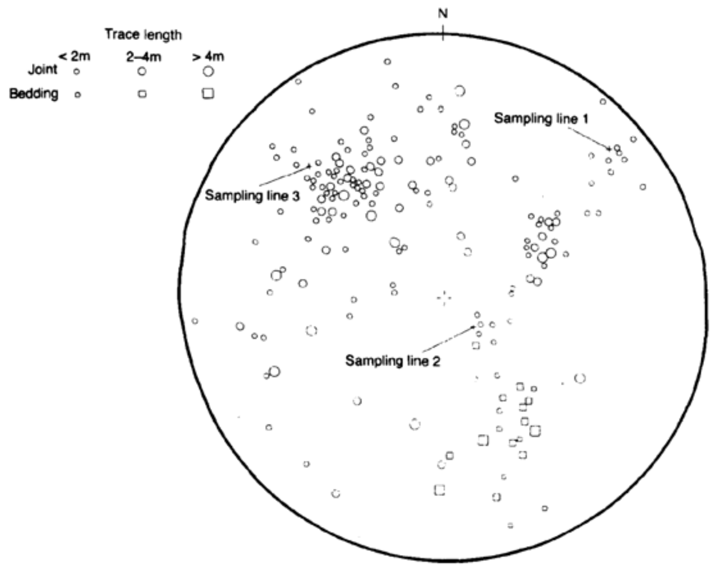


Fig. 2.6 Example of a lower-hemisphere plot of discontinuity normals (poles) classified according to persistence (after Priest 1985).

Statistical analysis of orientation data is carried out to identify sets of parallel or nearly parallel discontinuities. If the rock mass is dominated by a systematic planar fracture pattern then the predominant orientation of each discontinuity set may be identified with relative ease by observing the clusters of poles on the lower hemisphere projection. However in many cases the interpretation of the data is complicated by the following factors:

1. the discontinuities are not planar;

2. the rock mass is cut by randomly orientated discontinuities in addition to those that occur in sets; and
3. the degree of parallelism within a given set may be relatively low.

These factors will result in poles being less clustered. In such cases discontinuity sets, if present in the rock mass. may only be identified from a statistical analysis. This involves placing a sampling window over the data, to generate a matrix of moving average values, representing the variation in the concentration of poles over the projection. The moving average values may be contoured at some appropriate interval to aid interpretation. Figure 2.7 shows the contours of pole concentration determined in this way. Details of various sampling methods are given in Hoek and Bray (1981), Matherson (1983) and Priest (1993). The most commonly used technique is the counting circle (Hoek and Bray 1981; Priest, 1993). This makes use of a circle with a diameter such that the area of the counting circle represents 1% of the plane of projection. The circle is placed on the plane of projection and the number of poles falling within its perimeter are counted and recorded on the plane of projection at the position of the centre of the counting circle as a percentage of the total population of poles. The counting circle is moved over the plane of projection either in a random manner (floating circle) or using a rectangular grid. When the counting circle is close to the edge of the net, any part of the circle that extends beyond the perimeter must re-enter at a diametrically opposite point. Thus the contours of pole concentrations crossing the perimeter of the net should be symmetrical about the diameter. The counting circle method can be most time consuming. A more rapid system of sampling is provided by using a Dimitrijevic Counting Net (Dimitrijevic and Petrovc 1965). The use of this net is described by Matherson (1983). There are a number of computer programs available to perform the statistical analysis. These should be used with caution since it is not always clear what sampling method is being used.

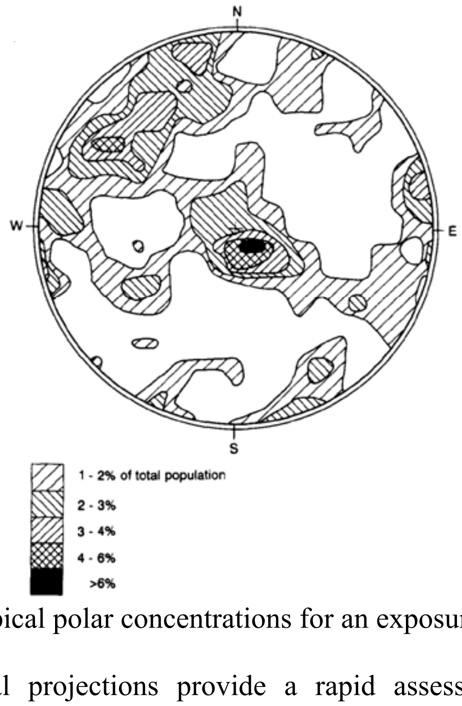


Fig. 2.7 Typical polar concentrations for an exposure of chalk.

Contoured lower hemispherical projections provide a rapid assessment of rock slope stability assuming the shear strength of the discontinuities are purely frictional (Markland 1972; Hoek and Bray 1981). Such assessments do not provide a factor of safety, they simply allow potential failure mechanisms to be identified. Hoek and Brown (1980) and Priest (1985) describe the use of hemispherical projections in the assessment of the stability of underground excavations.

Histograms

Discontinuity spacing data are best presented in the form of histograms. Histograms may be produced for individual sets of discontinuities or for all discontinuities intersecting a scanline. If the discontinuities in a particular set exhibit a regular spacing they will give rise to a normal distribution and a mean spacing may be easily determined. However, in many cases fractures are clustered or randomly spaced giving rise to a negative exponential distribution (Priest and Hudson 1976). Examples of joint frequency distributions measured from scanlines in sandstone and mudstone are given in Fig. 2.8. the histograms show a close agreement with the negative exponential distribution expressed as:

$$f(x) = \lambda e^{-\lambda x} \tag{2.3}$$

where λ = the mean discontinuity frequency per metre. By fitting a negative exponential distribution to the spacing data the mean spacing may be determined from 1/ λ.

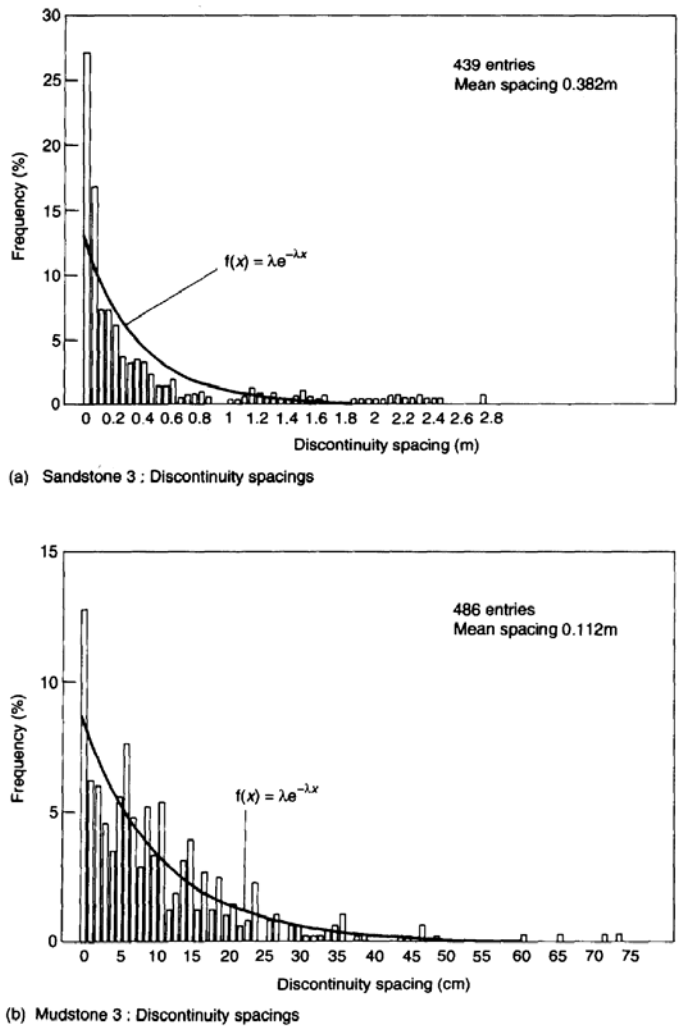


Fig. 2.8 Histograms of discontinuity spacings (after Yenn 1992).

Priest and Hudson (1976) established the following relationship between Rock Quality Designation (RQD) and the mean discontinuity frequency per metre (λ):

$$RQD = 100e^{-0.1\lambda}(0.1\lambda + 1) \tag{2.4}$$

where *RQD* is a parameter normally derived from drilicore (Deere 1964) and is commonly used in the classification of rock masses. The definition of RQD and its application to rock mass classification is discussed later.

DESCRIPTION OF ROCK MASSES

The rock material descriptions, the location of changes in lithology and the description of discontinuities can be brought together to form an overall description of the rock mass in engineering terms. The key elements of a rock mass description are as follows.

1. *Lithology*. This includes the rock types present and any variations in rock material properties within each lithological unit. A rock mass therefore may be divided up into zones on the basis of lithology or changes in intact material properties.
2. *Structure*. This includes large and small scale geological structures such as bedding, folding, faulting and intrusive bodies of igneous origin. A rock mass therefore may be divided up into zones on the basis of structure. It is likely that such a zonation will be similar to that based on lithology for certain structures.
3. *Weathering and alteration*. The processes of weathering or alteration are likely to bring about changes in the mechanical properties of the rock material and the rock mass within any given lithological unit. Weathering is likely to affect rocks near the ground surface, although it should be remembered that in certain cases the depth to which weathering extends may be more than l00m. Alteration may affect rocks at any depth. The rock mass may be divided into zones based on the degree of weathering or alteration.
4. *Discontinuities*. The discontinuities cutting the rock mass may be associated with a number of processes such as deposition, cooling, tectonism and weathering. The pattern of discontinuities commonly varies from place to place within a rock mass as a result the interaction of one or more of these processes and the rock material. The rock mass can therefore be zoned on the basis of discontinuity pattern (orientation, spacing and persistence will be dominant factors contributing to discontinuity patterns). Other attributes of discontinuities such as wall roughness and aperture may also be employed in this exercise.
5. *Engineering application*. Any engineering grade classification is likely to be performed for a particular engineering application. Different applications place emphasis on different attributes of the rock material and the discontinuities.

A rock mass description will involve dividing the rock mass into units of similar expected engineering behaviour. The parameters used in this exercise will be taken from (1), (2), (3) and (4) above and controlled by the engineering application. An example of such a method of rock mass description is given in Fig. 2.9. Thus, features are arranged into groups on the basis of their relationship with the application. Such a grouping forms the basis of engineering grade classification. An example of such a engineering grade classification is given for the chalk at Mundford, Norfolk by Ward *et al*. (1968). The application in this case was the assessment of rock mass compressibility under foundation loading. Ward *et al.* assumed that the compressibility of the rock mass depended primarily upon the following factors:

1. the presence or absence of structure;
2. the spacing of discontinuities;
3. the orientation of discontinuities;
4. the aperture of discontinuities; and
5. the hardness of intact chalk.

The classification of the Mundford chalk was based on these factors. The definition of the divisions used in this classification is shown in Table 2.29.

In areas not affected by intense tectonism the chalk is generally characterized by sub-horizontal and sub-vertical sets of discontinuities. In such cases it will be the subhorizontal discontinuities that will most influence the compressibility of the rock mass when subject to applied vertical stress changes. Factors (1), (2) and (4) are generally controlled by weathering processes in the chalk. At the Mundford site, a bed of low porosity high strength chalk (a hardground) was present within the rock mass. The intact chalk within this hardground had different mechanical properties to the chalk above and below it, and hence it was considered to be of importance in controlling the mass compressibility when within the zone of influence of the loaded area. Thus lithology played a part in this classification.

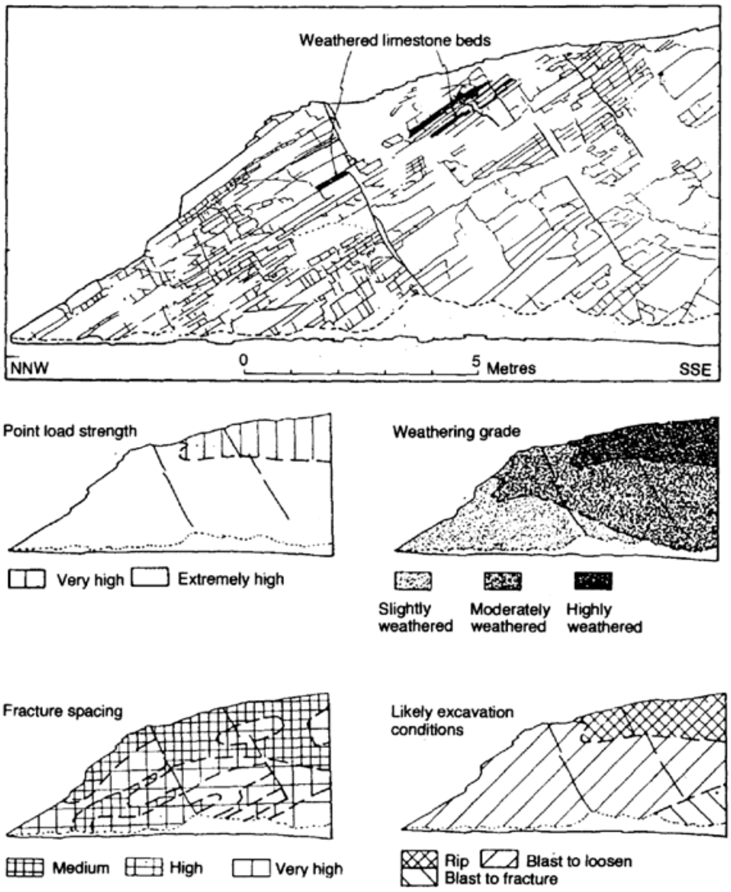


Fig. 2.9 Geotechnical properties and engineering appraisal of a quarry face comprising chert and limestone (after Fookes *et al.* (1971)).

Whilst structure, discontinuity spacing and aperture were defined in a way that could be determined with relative ease in the field, the other parameters were not. The lack of definition of some components may stem from the fact that these, like many engineering grade classifications, are site specific. This classification and its subsequent extensions have been used by geotechnical engineers indiscriminately over much of the chalk outcrop in the UK.

In order to make such a classification more applicable to the whole of the chalk outcrop a greater understanding of the mechanisms which control the mass compressibility of this material is needed. In an attempt to achieve a more generally applicable assessment of chalk mass compressibility based on visual assessment, Matthews (1993) proposed a more simplified classification that reflects our current state of knowledge. This classification is shown in Table 2.30.

Rock mass classifications are commonly used in rock engineering as an aid to design. Most of these classifications such as the geomechanics classification (Rock Mass Rating System (RMR), Bieniawski (1973)) and the Q-System (Barton *et al.* 1974) were developed primarily for underground excavation engineering. Some of these classifications such as the RMR system have been extended for use in rock foundation and slope engineering.

Table 2.29 Visual classification of the chalk (after Matthews *et al.* (1990))

Grade	Original description (Ward *et al.* (1968), I-V, Wakeling (1970), VI)	SPT N	Identification factors: Normally used in practice — Structure	Jointing/ particle sizes	Not normally considered — Hardness/strength	Weathering
VI	Extremely soft structureless chalk, containing small lumps of intact chalk.	<8	Bedding and jointing absent	Behaviour dominated by chalk fines	Extremely soft	
V	Structureless melange. Unweathered and 8—15 partially weathered angular chalk blocks and fragments set in a matrix of deeply weathered remoulded chalk. Bedding and jointing are absent.	8-15		Behaviour dominated by intact lumps		Deeply weathered
IV	Friable to rubbly chalk. Unweathered or partially weathered chalk with bedding and jointing present. Joints and small fractures closely spaced, ranging from 10mm apart to about 60mm apart. Joints commonly open up to 20mm and infilled with weathered debris and small unweathered chalk fragments.	15-20	Bedding and jointing present	Joints: 10-60mm spacing <20mm aperture with infill debris	Friable to rubbly	Unweathered or partially weathered
III	Rubbly to blocky chalk. Unweathered medium to hard chalk with joints 60mm to 200mm apart. Joints open up to 3 mm, sometimes with secondary staining and fragmentary infillings	20-25		Joints: 60-200mm spacing <3mm aperture possible infill	Rubbly to blocky	Unweathered , sometimes with secondary staining on joints
II	Medium hard chalk with widely spaced closed joints. Joints more than 200 mm apart. When dug out for examination purposes this material does not pull away along the joint faces but fractures irregularly.	25-35		Joints: >200mm spacing 0mm aperture	Medium hard	Unweathered
I	Hard, brittle chalk with widely spaced closed joints. Details as for Grade II but here the chalk is harder.	>35		As II	Hard	Unweathered

Table 2.30 Assessment of mass compressibility of chalk based on visual assessment (after Matthews (1993))

Grade	Description	Compressibility characteristics
A	Structured chalk: discontinuities more than 200mm apart, and closed.	Very low compressibility. E_i=1000-10000 MPa q_y>l000kPa Intact dry density significantly affects compressibility.
B(i)	Structured chalk: discontinuities closer than 200 mm apart, and open. Fracture-block system: tight.	Relatively low compressibility. E_i=500-1500MPa q_y=150-420kPa normally greater than 200kPa E_y≈50MPa Intact dry density is likely to have a limited effect on compressibility.
B(ii)	Structured chalk: discontinuities closer than 200mm apart, and open. Fracture-block system: loose.	Relatively low compressibility. E_i=300-500MPa q_y=150-420kPa, normally greater than 200kPa. E_y≈50MPa Intact dry density has little effect on compressibility.
C	Structureless chalk: a melange of fines and intact chalk lumps, with no regular orientation of bedding or jointing.	Relatively high compressibility. E_i=100-300MPa q_y unknown, but probably less than 200 kPa. Intact dry density has no effect on compressibility. Compressibility behaviour is likely to be effected by the relative proportions of fines and intact lumps of chalk

The most popular rock mass classifications used in rock engineering are the RMR and the Q-Systems. The factors used in these classifications are given in Table 2.31.

Table 2.31 Parameters used in the RMR system and the Q-System of rock mass classification

RMR System	Q-System
Uniaxial strength of intact rock	Rock Quality Designation (RQD)
Rock Quality Designation (RQD)	Number of joints sets
Spacing of discontinuities	Roughness of the most unfavourable discontinuity
Condition of discontinuities (wall roughness, aperture, wall strength, etc.)	Degree of alteration or filling along the weakest joint
Groundwater conditions	Water inflow
Orientation of discontinuities	Stress condition

Importance ratings are allocated to the different value ranges of the parameters, the higher rating indicating better rock mass conditions. The importance ratings are derived from experience and case histories. After the importance ratings for the parameters have been established they are combined to give an overall rating for the rock mass. This rating is most commonly used in assessing the support requirements for underground excavations (RMR and Q) or for estimating the rock mass compressibility for foundation design (RMR). The use of these rock mass classification systems together with others not mentioned here are discussed by Bieniawski (1989).

RECORDS OF BOREHOLES

Obtaining soil and rock samples f good quality requires much care and attention by the driller. As a result, this exercise can be expensive. This is particularly so in the case of rock. It is necessary therefore to justify this expense by maximizing the data gathered by this operation and presenting it in such a manner that it can be readily understood and interpreted. The final borehole logs should be based on the visual examination and description of the samples, the laboratory test results, and the driller's daily report forms. If, as is often the case, a well-trained drilling technician cannot be assigned to each drilling rig, any investigation cannot be better than the drillers responsible for the execution of the work. Even though the quality of the drilling and sampling may be good, if the driller is unable to observe and record adequately the results of his work, the final site investigation will be poor. Good driller's records are therefore often the key to good site investigation.

The driller's record should include the following information:

1. *contractual details*: contractor, client, location, and supervisor's name;
2. *borehole location*: borehole number, ground level at borehole, and inclination and bearing;
3. *drilling equipment*: type of drilling rig used, diameter of borehole, and details of casing. In the case of rotary drilling the type of bit and corebarrel should be given together with details of the type of flush used;
4. *progress*: date of start and finish of borehole, level at the end of each day's boring, and driller's name. In the case of rotary drilling, if the type of bit is changed for any reason the level at which it is changed should be recorded. If a bit needs to be replaced due to wear the level at which the new bit is started should be recorded;
5. *geotechnical data*: soil or rock description, with depth below ground surface, thickness of each soil or rock type, and level (relative to datum) of each change of soil or rock conditions;
6. *groundwater data*: levels at which water is encountered, levels at which water stabilizes in the borehole, rate of inflow, levels at which loss of return occurs, water and casing levels when taking water samples, and depth from which water samples taken;

7. *samples*: level of top and bottom of drive, diameter, type (e.g. open-drive, piston, double tube swivel type corebarrel with plastic liner (e.g. Core Line), etc.), reference number, and length of recovery;
8. in situ *testing*: level at which *in situ* test performed, type of test, and result of test.

In practice this record is usually achieved in two stages. The driller produces a hand-written borehole log at the end of each day's drilling, based on his records made as boring was in progress. This record is given to the engineer who carries out a preliminary engineering record. This record of the borehole is likely to be far from perfect, because at this stage the amount of laboratory testing has not been determined and therefore samples cannot be extracted and split for description, but it is produced to allow testing to be scheduled and additional boring to be varied.

Two factors must combine to produce a good engineering borehole or drillhole record: accurate recording of sampling and soil or rock changes at the time of boring, and good soil and rock description. In order to produce good records of stratum changes the driller or drilling technician must not only be capable of producing a good description of soil and rock, but he must also be aware of the importance of features such as fissuring and fabric in soils and discontinuities in rock. This requires a level of training which is rare at present. In the authors' experience it is unusual for the design engineer to communicate the important features of his proposed structure to the site investigation engineer, and frequently the engineer in charge of the site investigation makes no attempt to discuss his detailed requirements with the drilling foreman.

The final borehole log must be laid out in a simple but informative manner. It must be remembered that if the layout is made over-complicated, important information may be missed by the engineer when attempting to read the log. Examples of borehole and drillhole logs are given in Figs 2.10 and 2.11. In soft ground boring (Fig. 2.10), in addition to the sample descriptions emphasis is given to the location and type of sampling and *in situ* testing. In rotary coring (Fig. 2.11), however, emphasis is placed upon the state of recovery of the core and the fracture state. Guidance on the logging of rock core may be found in the Geological Society of London's Engineering Group Working Party report on the 'Logging of Rock Cores for Engineering Purposes' (Geological Society of London 1970). A revision Working Party set up by the Engineering Group (Geological Society of London 1977) to examine the need for revising the proposals of the 1970 Working Party found that these have in general won acceptance in the UK, although many contractors had made minor modifications to suit individual circumstances. Most of the proposals of the 1970 Working Party are now embodied in the Code of Practice (BS 5930:1981).

It is common practice when logging rock core to take colour photographs of each core run laid out in the corebox. It is important that a colour scale and scale rule be included in the photograph to aid interpretation and comparison with the descriptions given in the log.

STATE OF RECOVERY OF CORE

The state of rock cores recovered is largely a function of the drilling method, and the amount of care employed by the driller during a core run and extraction of core from the corebarrel. Hence these factors must be considered when assessing core recovery and fracture state. The nature and amount of core recovered from good careful drilling can provide a valuable indication of the *in situ* condition and probable engineering behaviour of the rock mass. In any core recovered there will be fractures of natural and artificial origin. It is important that natural fracturing is distinguished from artificial fracturing on the log. Artificially induced fractures should not be ignored since they may assist in the assessment of rock excavation.

Location

Record of Borehole No. 4

(SHEET 1 of 2)

Client

Type of boring SHELL AND AUGER

Job No. Ground level 78.20m O.D. Diameter / 150mm Casing / 150mm

Daily Progress	Ground water levels	Depth of casing	Samples: Depth	Samples: No.	Samples: Type	Strata: Depth	Strata: Reduced level	Description of strata	Geology
23.5.80								TOPSOIL	
			0.50	1	D	0.35	77.85	Stiff light brown CLAY with angular flint and chert gravel becomes darker with depth	
			1.00	2	D				
			1.50 - 1.95	3	U(21)				
						1.80	76.40	Firm light brown sandy CLAY with gravel and sandy lenses	
			2.50	4	D				
	2.80		2.75	5	B				
	3.80		3.00	7	BC(9)	3.00	75.20	Medium dense coarse with fine to medium clayey silty SAND with some gravel lenses	UPPER SAND
			2.80	8	W				
			4.50	10	BC(11)				
			4.65	11	D	4.60	73.60	Soft brown CLAY	
						4.80	73.40	Very stiff grey CLAY with chalk fragments. Weathered brown between 4.80 and 5.00m and 6.75 to 7.00m	BOULDER CLAY
			5.00	12	D				
			6.00 - 6.45	13	U(31)				
			6.75	14	D				
			7.00	15	D	7.00	71.20	Very dense fine to coarse sub-angular flint and chert GRAVEL with medium to coarse sand	LOWER GRAVEL
			7.50	17	BC(100+)				
			8.25	18	D				
			9.00 - 9.75	20	BC(100+)				
23.5.80	7.25	9.50							
24.5.80	NIL	9.50	9.75	21	D				

Key
U.... undisturbed 102mm diameter sample
D.... disturbed jar sample
B.... disturbed bulk sample
W.... water sample
S( ). standard penetration test
C( ). cone penetration test
(33).. number of blows ('N' value)
▼.. groundwater encountered

Remarks

Losing water in lower gravels

Continued/....

GROUND ENGINEERING LIMITED

GE 3035

Fig. 2.10 Engineering borehole record in soft ground (courtesy of Ground Engineering Ltd).

The core recovered can be divided into five categories:

1. solid core greater than 0.1 m in length;
2. solid core less than 0.lm in length;
3. fragmental material not recovered as core;
4. additional material which may have been lost from the previous core: and
5. reduced length and/or diameter of core due to erosion of soft or friable material.

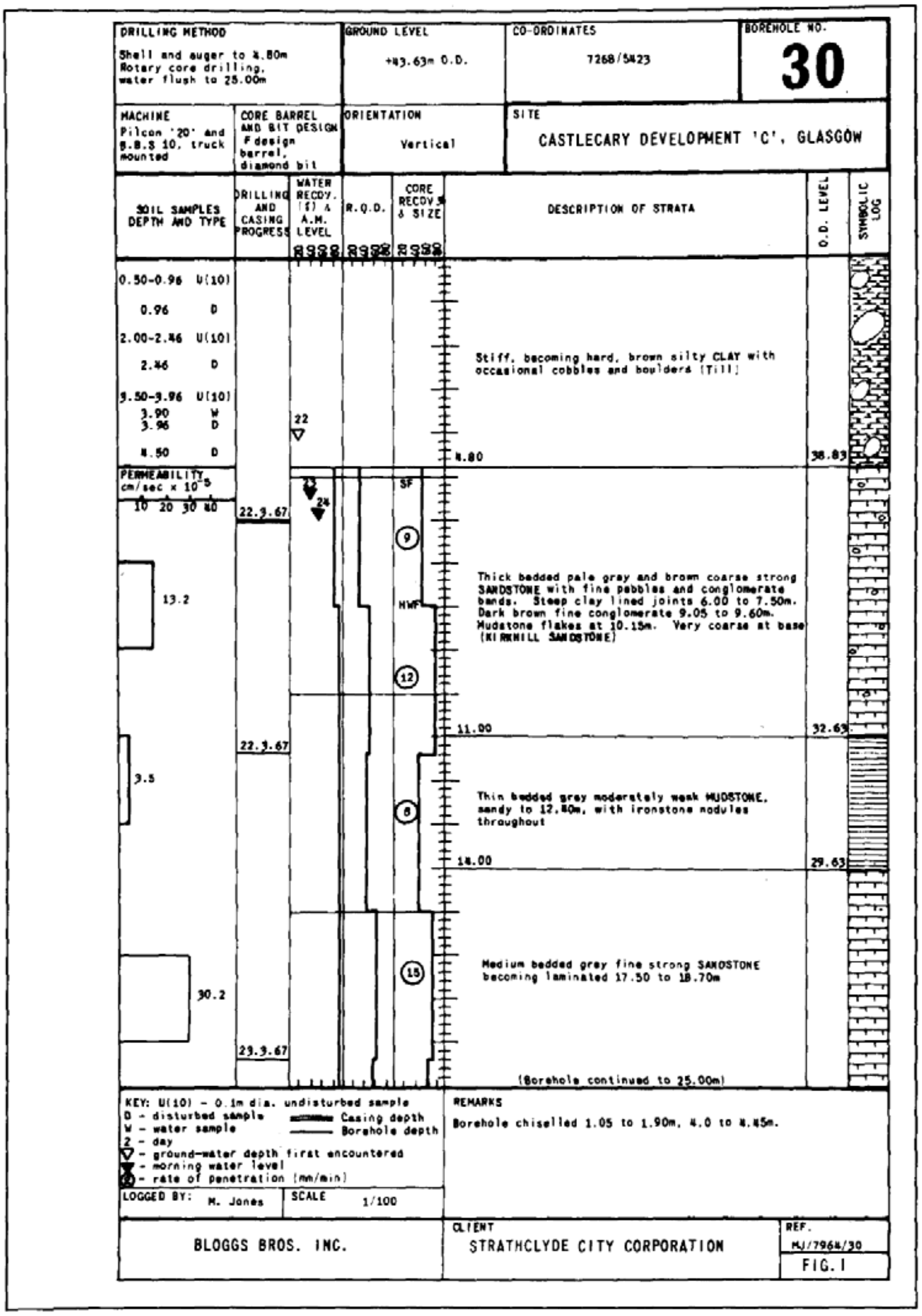


Fig. 2.11 Engineers' drillhole record (Geological Society of London 1970).

The quality of rock recovered may be classified in terms of total or solid core recovery or in terms of a quality index such as rock quality designation (RQD), fracture index or stability index, provided only natural fractures are considered. The definitions of these terms are given in Table 2.32. The determination of the more commonly used parameters are shown schematically in Fig. 2.12. Solid core recovery (SCR), RQD, fracture index and stability index may be used as criteria for a quantitative description of the fracture state of the cores. The simplest of these is solid core recovery and is always shown along with total core recovery in a graphical form in the borehole log. The stability index is the most complicated method of assessing rock quality and hence is rarely used in practice. Core recovery (total and/or solid), RQD and fracture index are normally shown in the borehole log in a graphical form with some indication of changes in corebarrel size (Fig. 2.11). The fracture state of the core recovered may be assessed using these parameters together with the fracture log discussed earlier.

Table 2.32 Methods of classifying the quality of rock cores

Classification	Definition	Category of core considered	Remarks
Total core recovery	Percentage of the rock recovered during a single coring 'run'	(1), (2), (3), (5), i.e. all the core placed in the core box	Gives indication of material that has been washed into suspension or the presence of natural voids
Solid core recovery	Percentage of full diameter core recovered during a single coring 'run'	(1) and (2)	Gives indication of fracture state
Rock quality designation (RQD) (Deere 1964)	Percentage of constant diameter solid core greater than 0.lm in length recovered during a single coring 'run'	(1)	Can give indication of fracture state but does not take changes in core diameter into account. The diameter of the core should preferably not be less than 55mm (NWX or NWM size)
Fracture index	Number of fractures per metre. This is generally calculated for each core run	(1) and (2)	Can give indication of fracture spacing
Stability index (Ege 1968)	Index no. = 0.1 x core loss (length drilled-total recovery) x 10^{-2}+no. of fractures per 0.3m (1 ft) + 0.1 x broken core (core <7.5 cm in length) + weathering (graded 1-4 from fresh to completely weathered) + hardness (graded 1-4 from very hard to incompetent)	(1), (2), (3), (5)	Can give indication of fracture state but does not take changes in core diameter into account

Since it was first introduced by Deere and his co-workers in 1967, RQD has become increasingly used. Indeed it has become an integral part of the rock mass classification systems used in underground excavation engineering and in rock foundations (Bieniawski 1989). For 50mm diameter cores, where a height to diameter ratio of 2:1 was used for strength and stiffness tests, RQD provides a useful indication of the number of times such a test result could occur in each core run, assuming the lithology was consistent. As such, the percentage of core over 100mm in length had a real value, especially in assessing the bearing capacity of a rock mass. However both the British Standard (BS 5930:1981) and the ISRM (ISRM 1981) recommend the RQD should be based on an axial measurement along the centre of the core rather than the solid core length. When there are discontinuities present that are inclined such that they make a relatively small angle with the core axis (for example, steeply dipping joints intersecting a vertically orientated core), the RQD can have a higher value than the SCR and have no relationship to the performance of that rock mass to any tested sample (Fig. 2.13).

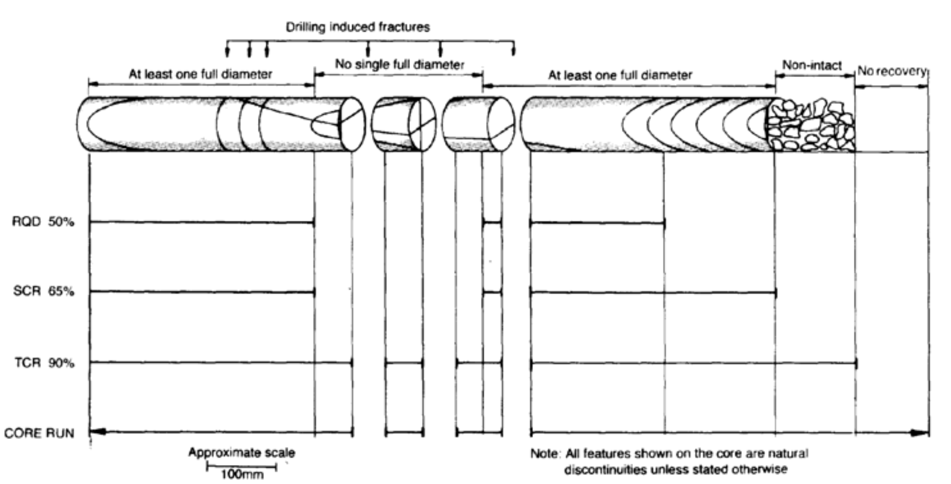


Fig. 2.12 Schematic illustration of fracture logging terms (after Norbury *et al.* (1986)).

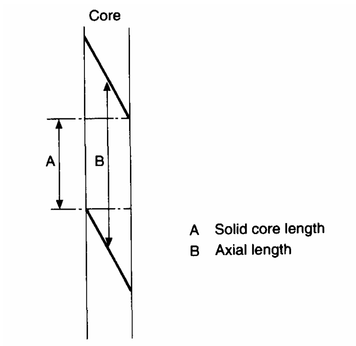


Fig. 2.13 Measured lengths related to an inclined discontinuities.

Hawkins (1986) suggests that a new rock quality designation value be introduced based on minimum core lengths of 300mm instead of 100 mm. This new value would be referred to as RQD_{300}. The reasoning behind this proposal is that 300 mm approximates to the maximum thickness that can be ripped in certain rock types. A more sound argument for changing the base length for RQD measurements relates to the comments made above concerning the value of RQD in relation to the number of test specimens that can be obtained from a single core run. Since it is now common practice to use P and S size corebarrels, particularly in weak rocks, solid core lengths of greater than 230 mm or 281mm respectively are required to meet the 2.5:1 length to diameter ratio recommended by ISRM for strength and stiffness tests (ISRM 1981). The disadvantage of using a different base length however is that RQD_{100} is required for the conventional rock mass classification systems such as the RMR or the Q-System (Bieniawski 1989).

The measurement of RQD does not take into account any changes in lithology within the core run. Changes in lithology are often associated with a change in the fracture pattern owing to the different mechanical properties of each rock type. There is a natural tendency for engineers to assume that a high RQD value comes from a stronger rock. In a sequence of interbedded limestones and mudstones, for example, the limestone units may dominate in contributing to a relatively high RQD value, thus masking the fracture state of the less competent mudstone units. Such an example is shown in Fig. 2.14. In the example shown, the mudstone units contribute nothing to the RQD value of 55% for each run since they are characterized by a horizontal fracture spacing of 90 mm. Hawkins (1986) suggests that this type of problem may be avoided if RQD values are based on the thickness of lithological units rather than core run length. The lithological quality designation (LQD) could be shown on core logs alongside the conventional RQD values. Fig. 2.14 shows the relationship between LQD and RQD for two adjacent core runs. Clearly LQD is of particular value when dealing with interbedded rocks of contrasting mechanical properties. In cases where the thickness of each rock unit is equal to or greater than the core run length the value of the LQD is diminished since the conventional RQD measurements are likely to reflect the changes in fracture state associated with lithology.

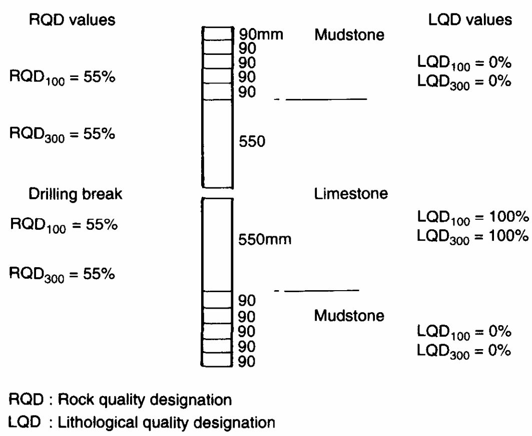


Fig. 2.14 Relationship of RQD and LQD on two adjacent core runs (after Hawkins 1986).

RECORDS OF TRIAL PITS AND SHAFTS

Trial pits are the cheapest method of obtaining samples and a continuous record of ground conditions in soils and soft rocks for shallow depths. Shafts may be used to investigate greater depths in cohesive soils and rocks, but generally provide less discontinuity data since it is more difficult to clean the debris caused by digging from the circular face of a shaft than it is from the flat faces of a trial pit. Furthermore, shafts are often more expensive to dig than trial pits because of the specialized plant involved. Trial pits are ideally suited for investigating superficial deposits which often exhibit a high degree of variability both laterally and vertically. They are also particularly valuable in examining fill material for voids, loosely compacted layers or deleterious material and for investigating slope failures. Trial pits allow hand cut samples of soil to be taken, thus minimizing the effects of mechanical disturbance. Where pits are dug in landslips it is often possible to take undisturbed samples of the major slip planes.

When recording information from trial pits, particular attention should be paid to the orientation of the faces of the pit, the orientation of undisturbed samples taken from it, and the orientation of discontinuities (fissures, joints, cleavage, bedding planes, slip planes, and faults) encountered within it. The logging of trial pits or shafts involves making a full engineering description of all the materials and discontinuities found in the faces and floor of the pit in the manner outlined earlier. Trial pits are generally logged by describing the materials encountered along a vertical line in one or more faces of the pit. If the soil or rock is highly variable, all the faces should be described in detail and a scale drawing made of each face. In some cases, it may be necessary to describe the materials in the floor of the pit. Discontinuities where present should be described in all the faces of the pit. In situations where more than one face is examined in detail, a plan of the pit should be drawn and the respective faces labelled and orientations noted on the plan in order to identify the various pit face logs. It is often useful to take photographs of each face of the pit for future reference. The Geological Society of London's Working Party Report on the Preparation of Maps and Plans in Terms of Engineering Geology (1972) gives some guidance on the presentation of trial pit logs.

In order to log the faces of a trial pit or to take samples it will be necessary for personnel to work at the bottom of the pit. This can be extremely hazardous since the sides of the pit may collapse with little or no warning. In some cases, pits are dug with one face battered back in order to allow rapid escape. Support should be provided for the sides of pits which are deeper than 1.2 m; methods of timbering are discussed by Tomlinson (1980).