Geological Masses

3.1 Mass Fabric

The word '*mass*' is already defined in Chap. 1 as '*the volume of ground that will be influenced by or will influence the engineering work*'. All rock and many soil masses have discontinuities and their presence in the rock or soil mass is of the utmost importance to all engineering works in rock or soil. Much research has been undertaken into the characteristics and behaviour of discontinuities. Whole conferences, such as that on 'Rock Joints' held in 1990 (Barton and Stephenson 1990) have examined the subject in great detail. However, there is a major gap between acquiring such knowledge by research and the application of that knowledge to practice. *The problem is that the investigator must forecast the character of the discontinuities in the mass, and thence their likely behaviour, from the evidence available from outcrops or boreholes. Adequate data is seldom available.*

3.1.1 Discontinuities

The properties of discontinuities of greatest importance to a volume of ground are: shear strength, influencing, for example, the stability of slopes, underground excavations, and foundations, and stiffness, influencing the deformation and permeability of the ground.

Types of Discontinuities

Discontinuities must be classified in relation to their type. Two basic types of discontinuities may be distinguished. (1) *Integral discontinuities* – which are discontinuities that have yet to be opened by movement or weathering; they have tensile strength and, hence, a true cohesion. Intact bedding planes, foliation planes, and strongly cemented joints are integral discontinuities. (2) *Mechanical discontinuities* – discontinuities which have been opened as a response to stress or weathering; they have little or no tensile strength but do generate shear strength. They may be divided into:

- *Bedding, schistosity or foliation planes* – these are formed by changes of material or mineral arrangement in the rock.
- *Joints* – which result from strains of tectonic or diagenetic origin and often fall into well-defined sets whose members are oriented essentially parallel to each other.

- *Fractures* – which result from strain due to man-made stresses (blasting etc.) or geomorphological strains (land sliding, creep etc.). They do not necessarily fall into well-defined sets.
- *Faults and shears* – which result from tectonic, geomorphological and man-made strains, with shear movement on either side of a shear surface.

Bedding, schistosity and foliation planes, joints and fractures are normally a regular occurring feature in the mass at approximately regular distances, with similar orientation, and with more or less similar characteristics (Fig. 3.1). These are normally grouped in a "*set*" or family, e.g. bedding plane family and joint set, etc. Both faults and shears may fall into well-defined sets but may occur on a scale greater than that of most engineering works.

3.1.2 Shear Strength

The main concern regarding discontinuities in engineering geology is their resistance to shear stress. This is described by Coulomb's law, which is:

$$\tau = c + \sigma_n \tan\varphi \qquad (3.1)$$

where τ = shear strength (N m^{-2}), c = cohesion (N m^{-2}), σ_n = normal stress on the sliding plane, and φ = angle of sliding resistance.

The cohesion may be caused by 'true cohesion', e.g. there is tensile strength between the two discontinuity surfaces, or may be 'apparent cohesion' caused by irregularities

Fig. 3.1. Bedding planes as mechanical discontinuity planes causing failure of a slope in limestone

along the discontinuity surfaces. The surfaces of mechanical discontinuities may be smooth and planar or exhibit varying degrees of roughness. For a particular rock the greater the roughness of the surface the greater the resistance of the discontinuity to shearing. If normal stresses are relatively low then shear movement along a rough surfaced discontinuity must be accompanied by vertical dilation (Fig. 3.2). Patton (1966) analysed the situation by proposing that the *angle* of shearing resistance of a surface containing asperities inclined at angle i, is ϕ_b, the basic friction angle, plus i, so that the shear strength of the discontinuity with asperities is:

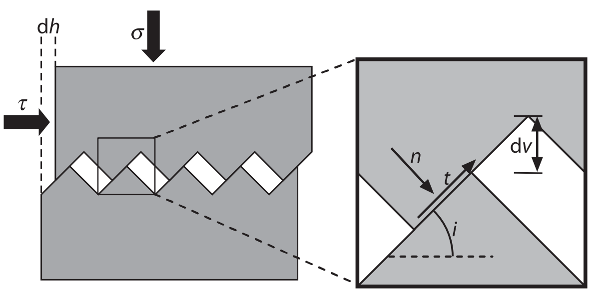
$$\tau = \sigma_n \tan(\phi_b + i) \tag{3.2}$$

One way to determine ϕ_b is to use a shearbox test in which vertical and horizontal displacements are measured. $dv/dh = \tan i$ and with Eq. 3.2, ϕ_b can be determined. The ϕ_b has been measured in the laboratory by many researchers to give values ranging from about 25° to 35°, the lower values generally pertaining to sedimentary rocks and the higher to igneous rocks. If $\phi_b + i = 90°$ sliding is not possible and breaking of the asperities has to occur before shear displacement is possible. The asperities fail also if normal and shear stresses are high relative to the strength of the asperities. Asperity failure of a discontinuity will give an '*apparent cohesion*' (Fig. 3.3). The residual angle of friction, ϕ_r is the angle of friction obtained after a discontinuity has been displaced. ϕ_r may equal ϕ_b, however it may not if asperities have broken, because rolling particles of debris influence the measured friction in a displaced discontinuity and the material remaining on the discontinuity wall after breaking of the asperities may also be of better quality, as it is probably less weathered than the outside of the asperities.

Scale of Roughness and Anisotropy

Roughness may be seen on different scales. Various authors have described roughness scales and descriptions for roughness (Fig. 3.4) show the relations as suggested by Hack (1998). Up to mm scale the roughness is normally formed by grains or crystals in the rock, as, say, might be found in shrinkage joints in granite. On the 0.01 to 1 m scale, roughness is normally due to depositional features in sedimentary rock or foliation undulations in metamorphic rock. At greater scales discontinuity surface undulations may

Fig. 3.2. Dilation (*dv*) following sliding on asperities under horizontal applied stress (τ) and vertical applied stress (σ)



n is the normal and *t* the shear stress on the contact plane, resulting from the applied stresses σ and τ

Fig. 3.3. The behaviour of rough and smooth discontinuities with varied shear and normal stresses

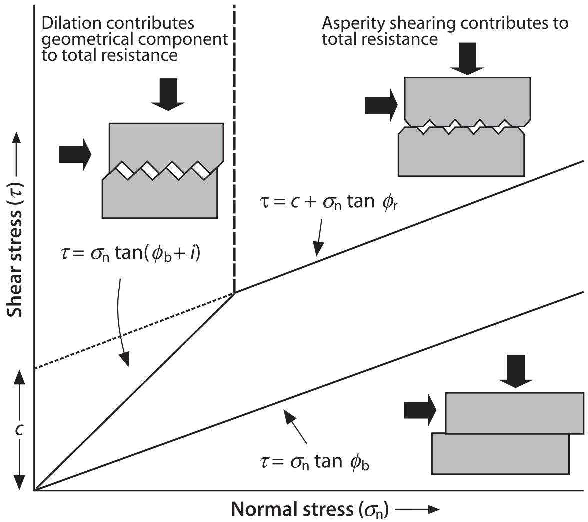
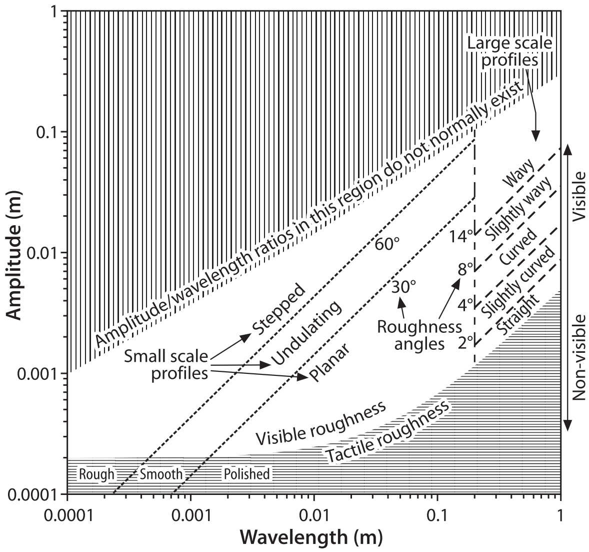


Fig. 3.4. Amplitude versus wavelength for discontinuity roughness (after Hack 1998). For small amplitudes and wavelengths, the roughness is of a triangular form whereas with larger amplitudes and wavelengths the roughness changes to a more sinusoidal form. Lustre is not included in the boundary Non-visible to Visible roughness. The boundaries in the graph are *dashed*, as these are not exact



be brought about by folding and it rests with the judgement of the observer to decide whether such dip variations may be considered as large scale roughness or to mark domain limits for geotechnical units. It must be remembered that profiles are two-dimensional while roughness is a three dimensional property. On the scale of grain size, this may not be important except in the case of the lineation features known as slickensides (Fig. 3.5); in the direction of lineation these are smooth but, normal to it, they are rough. At larger scales features such as ripple marks may be similarly distinctly anisotropic.

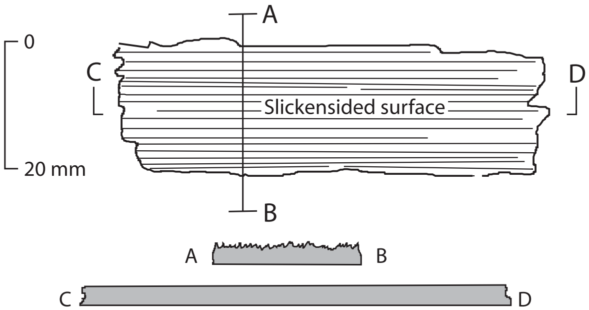


Fig. 3.5. Roughness dependent on direction for slickensided surfaces

Fitting versus Non-Fitting Roughness

Figure 3.2 shows a regular saw tooth profile as roughness profile. In reality, roughness profiles of discontinuities are not so regular and are three-dimensional. In general, only one perfect matching fit exists between the walls of the discontinuities. If the walls are displaced relative to each other, the fit will become less. If the displacement is large enough the roughness profiles become completely non-fitting. The effect on shear strength is that the contribution of the roughness angle i gradually diminishes with larger displacement. The effect has been described by Rengers; the so-called 'Rengers envelop' (Fig. 3.6). This effect makes it very important that the description of a discontinuity includes whether the discontinuity walls are fitting or displaced. In the later case an estimate should be given as to how much the i-angle has been reduced.

Discontinuity Wall Strength

A full evaluation of discontinuity shear strength requires not only assessment of roughness but also of wall strength. Accordingly, it is necessary to measure wall rock strength. This is almost impossible by conventional laboratory testing because of the extremely small size of the samples but recourse may be had to impact testers applied to the discontinuity surface. The Schmidt hammer gives too strong a blow to be of use (for the N-value reflects both near surface and deeper rock properties) but the Equotip tester (Verwaal and Mulder 1993) gives a much lighter blow which rebound reflects rock properties at shallow depth. Its use to assess wall strength has been described by Hack et al. (1993) (Fig. 3.7). If it is necessary to measure wall strength, it also follows that for some rocks that are particularly susceptible to weathering, estimates should be made of the *rate* of strength reduction by weathering of the discontinuities in any new construction.

Aperture and Discontinuity Infill

Openness of discontinuities in outcrops may have arisen from slope relaxation, near surface movement or plant root wedging and thus (except perhaps in the case of solution cavities in limestone) may in no way reflect internal conditions. If discontinuities are open, they may be either wholly or partially filled by material from either weath-

Fig. 3.6. Rengers envelope (after Fecker and Rengers 1971)

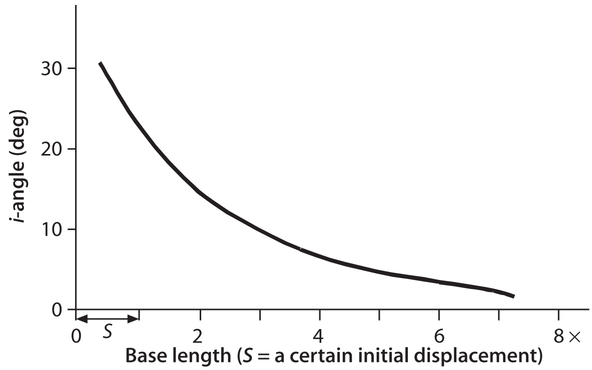
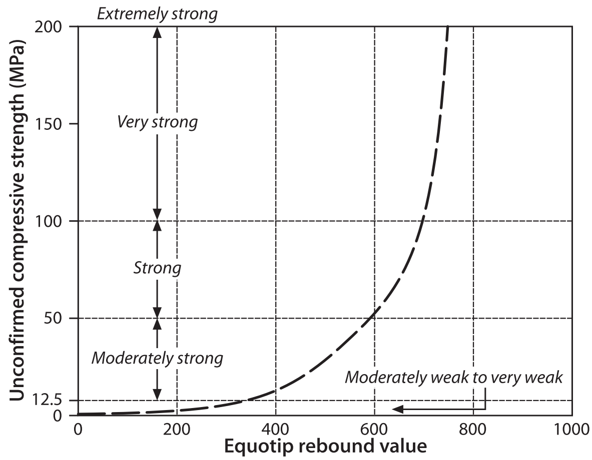


Fig. 3.7. Equotip rebound values vs. unconfined compressive strength (after Verwaal and Mulder 1993)



ering in situ or from outside. Thus bedding planes opened by weathering may be filled with clay or limestone solution cavities filled by washed in debris. Such infilling will clearly affect the shear strength of the discontinuity. If the infill is weak, the discontinuity shear strength may be less than that of the discontinuity with walls in contact. If the infill results from mineralisation as, for example, quartz or calcite, the infill may be stronger than, and closely bonded to, the rock; the discontinuity may then be described as 'healed'. Table 3.1 (BS EN ISO 14689-1) gives descriptive terms for discontinuities that have a continuous aperture or an infill of continuous thickness in all directions. However, discontinuities often do not have a continuous aperture or infill thickness. For such discontinuities, Table 3.1 looses its merits and other description criteria should be used, such as, descriptions based on the ratio of volume enclosed by the discontinuity to the total rock mass volume.

Field Estimate of Discontinuity Friction Angle

Determining the shear strength of a discontinuity is one of the most difficult tasks while it is probably the most important property to measure. A simple and often ad-

Table 3.1. Descriptive terminology for aperture

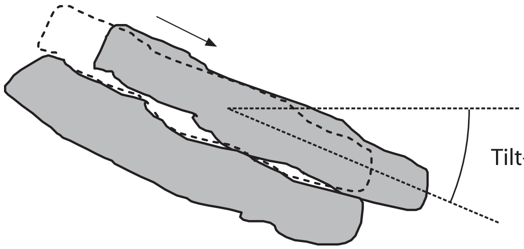
Class	Descriptor	Numeric value (mm)
Closed	Very tight	< 0.1
	Tight	0.1 – 0.25
	Partly open	0.25 – 0.5
Gapped	Open	0.5 – 2.5
	Moderately wide	2.5 – 10
	Wide	10 – 100
Open	Very wide	100 – 1 000
	Extremely wide	> 1 000

equate assessment can be made with the tilt test. Two pieces of rock including the discontinuity are tilted while the angle of the discontinuity with the horizontal is measured (Fig. 3.8) with, for example, the inclinometer included in a geological compass. The angle measured at the moment the top block moves is the '*tilt-angle*'. If no infill is present and the two blocks had fitting discontinuity surfaces, the tilt angle equals the small-scale roughness angle plus the angle of friction of the surface material. If the discontinuity roughness is completely non-fitting, the tilt-angle equals the material friction only. Undisturbed infill material will seldom be present in this test, but if it is, the tilt-angle includes the influence of the undisturbed infill. If it is not possible to obtain a sample with undisturbed infill material, it is sometimes possible to scrape infill material from another still in situ discontinuity and place this between the blocks of the tilt test sample. The thickness of the so-formed infill layer should be the same as in situ. The measured tilt-angle is then including the influence of remoulded infill material. Cohesion is not measured in the tilt test separately. If present, either real or apparent, it will be included in the tilt-angle. Steps on discontinuity planes causing hanging of the blocks, for example, result in a very high tilt-angle (which may be up to 90°). Whether this is realistic for the friction of the in situ discontinuity has to be judged on the strength of the cohesion or asperities in relation to the stresses in the in situ rock mass. Stresses in the rock mass will generally be far higher than those used during the tilt test and may cause either shearing of real cohesion or shearing of asperities along the in situ discontinuity. Clearly, the tilt-angle is only representative for small-scale roughness and low normal stresses as tilting meter-scale rock blocks is normally not possible for the average engineering geologist!

A more sophisticated methodology is to use classification systems to estimate friction or shear strength along a discontinuity. An example is the relation given in Eq. 3.3 (Barton 1971):

$$\tau = \sigma_n \tan\left[JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right) + \varphi_\mathrm{r}\right] \tag{3.3}$$

Fig. 3.8. Tilt test



JRC is the joint roughness coefficient (a number, low for smooth planar surfaces rising with increasing roughness, estimated by visual comparison of the discontinuity surface to standard roughness graphs), *JCS* is the joint wall condition strength, σ_n is the normal stress on the discontinuity, and ϕ_r is the residual friction angle. If no other test is available, it is possible to use the tilt-angle of a non-fitting discontinuity without infill material as residual friction angle.

Another example of a classification system for estimating discontinuity shear strength is the 'sliding criterion'. Based on back analyses of slope stability a sliding criterion was developed to easily estimate the shear strength of a discontinuity (Hack and Price 1995; Hack et al. 2002). The discontinuity is characterised following Table 3.2. The roughness is characterised by visually estimating large (Fig. 3.9) and small-scale (Fig. 3.10) roughness by comparing to standard profiles and by establishing tactile roughness, infill material, and presence of karst. The different factors for the different characteristics are multiplied and divided by an empirically established factor. This results in the so-called '*sliding angle*':

$$\varphi_{\text{sliding angle}}(\text{degrees}) = \frac{Rl \times Rs \times Im \times Ka}{0.0113} \tag{3.4}$$

The '*sliding angle*' is comparable to the tilt test idea but on a larger scale. The sliding angle gives the maximum angle under which a block on a slope is stable. The 'sliding criterion' has been developed on slopes between 2 and 25 m high. The 'sliding criterion' applies for stresses that would occur in such slopes, hence, in the order of maximum 0.6 MPa.

3.1.3 Persistence (Continuity)

Discontinuities may be persistent for long distances, for example, bedding planes, they may be persistent for a certain length and end in intact rock, or they may abut against other discontinuities. The shear strength along the discontinuity is dependent on the persistence. Intact rock has to be broken before displacement can take place if the discontinuity ends in intact rock. Boundary blocks have to move before abutting discontinuities can be displaced. If possible, persistence (sometimes called continuity) of discontinuities should be measured but opportunities to take such measure-

Table 3.2. Discontinuity characterisation for 'sliding criterion' (after Hack and Price 1995)

Condition of discontinuities			Factor
Roughness, large scale (*Rl*) (visual area > 0.2 × 0.2 and < 1 × 1 m^2)	Wavy		1.00
	Slightly wavy		0.95
	Curved		0.85
	Slightly curved		0.80
	Straight		0.75
Roughness, small scale (*Rs*) (tactile and visual on an area of 20 × 20 cm^2)	Rough stepped/irregular		0.95
	Smooth stepped		0.90
	Polished stepped		0.85
	Rough undulating		0.80
	Smooth undulating		0.75
	Polished undulating		0.70
	Rough planar		0.65
	Smooth planar		0.60
	Polished planar		0.55
Infill material (*Im*)	Cemented/cemented infill		1.07
	no infill-surface staining		1.00
	Non softening and sheared material, e.g. free of clay, talc, etc.	Coarse	0.95
		Medium	0.90
		Fine	0.85
	Soft sheared material, e.g. clay, talc, etc.	Coarse	0.75
		Medium	0.65
		Fine	0.55
	Gouge < irregularities		0.42
	Gouge > irregularities		0.17
	Flowing material		0.05
Karst (*Ka*)	None		1.00
	Karst		0.92

ments are limited to outcrops. Persistence cannot be measured from cores. Persistence is usually quoted as a simple one-dimensional measurement; the terminology for its description is given in Table 3.3. However, in reality persistence exists in two dimensions. Thus in Fig. 3.11a joint *y* seen in an outcrop perpendicular to the direction of strike may have but small persistence across a bed (e.g. abutting) while, seen on the bedding plane, it may have long persistence in the direction of strike. In Fig. 3.11b, joint *y* has limited persistence in the direction of strike. The difference in two-dimensional persistence between joints *y* in Fig. 3.11a and b could have great significance in slope stability. Sandstone blocks could part along *y* and slide down slope over the shale in Fig. 3.11a, but in Fig. 3.11b parting and down slope block sliding would be inhibited by the limited persistence of joint *y*. It is suggested that persistence in banked (see Fig. 3.12) and bedded rocks be recorded as a proportion of bank thickness. Thus, for example, in Fig. 3.11a the persistence of *y* could be given as >20 *t* in strike direction, where *t* is bank thickness, and in Fig. 3.11b the persistence of *y* could be given as ¼ to 1½ *t* in strike direction.

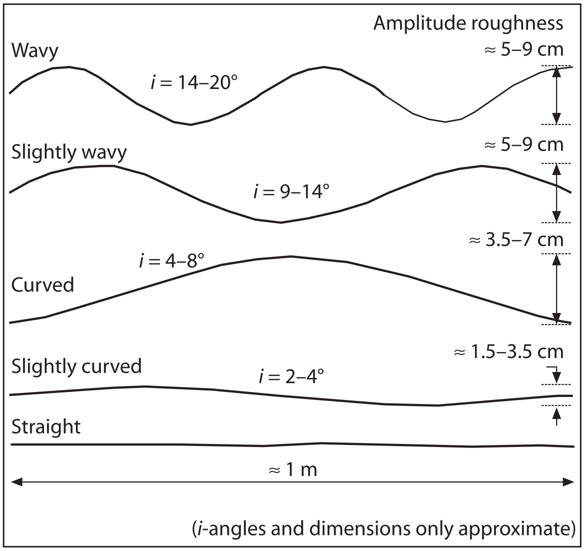


Fig. 3.9. Large scale roughness profiles (after Hack and Price 1995)

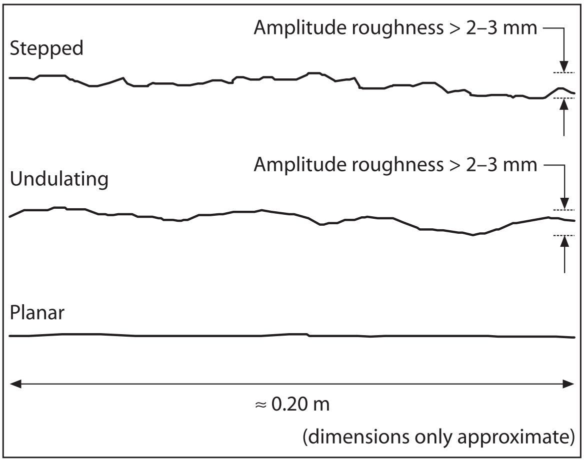


Fig. 3.10. Small scale roughness profiles (after Hack and Price 1995)

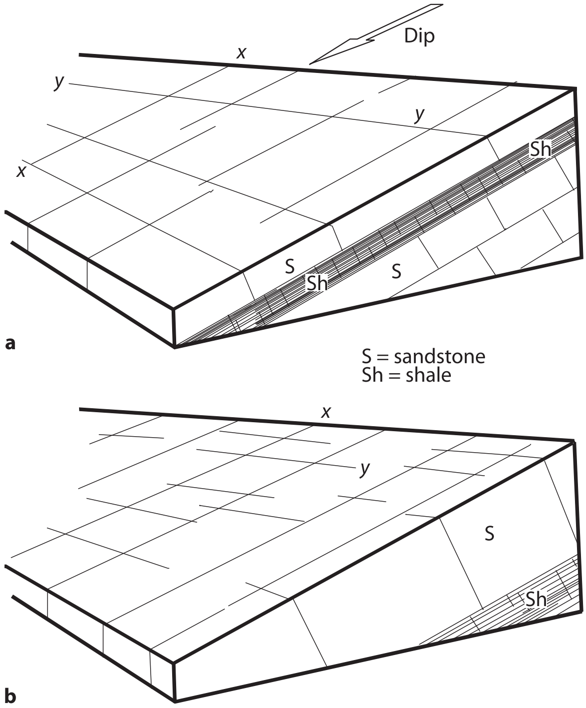
3.1.4 Orientation

The importance of discontinuities in any particular project depends partly on their orientation relative to directions of imposed stresses. The geologist records the orientation of discontinuities (which may be geological surfaces such as bedding planes giving an indication of geological structure) using a geological compass/clinometer usually placed on the surface being measured at some readily accessible outcrop. In the much more detailed surface mapping required for engineering geological purposes dip and strike measurements of discontinuities have

Table 3.3. Terms for the description of one-dimensional persistence (after BS EN ISO 14689-1)

Term	Numerical value (m)
Very low	< 1
Low	1 – 3
Medium	3 – 10
High	10 – 20
Very high	> 20

Fig. 3.11. Persistence



to be taken regardless of whether they are easy to reach or not. For work in tunnels, the compass/clinometer must have built-in illumination. Clinometers should have the capacity to sight-in the inclination of a discontinuity that cannot be reached for a contact measurement, say, for example, in a tunnel roof. Discontinuity orientation may also be measured from terrestrial stereo-photographs or laser scans (Slob et al. 2002).

There are various ways of expressing orientation. For engineering geological purposes, the best way is to record fully dip and strike data, leaving no possibility for misinterpretation of the record. Check the method you propose to use with another person – do they understand it!

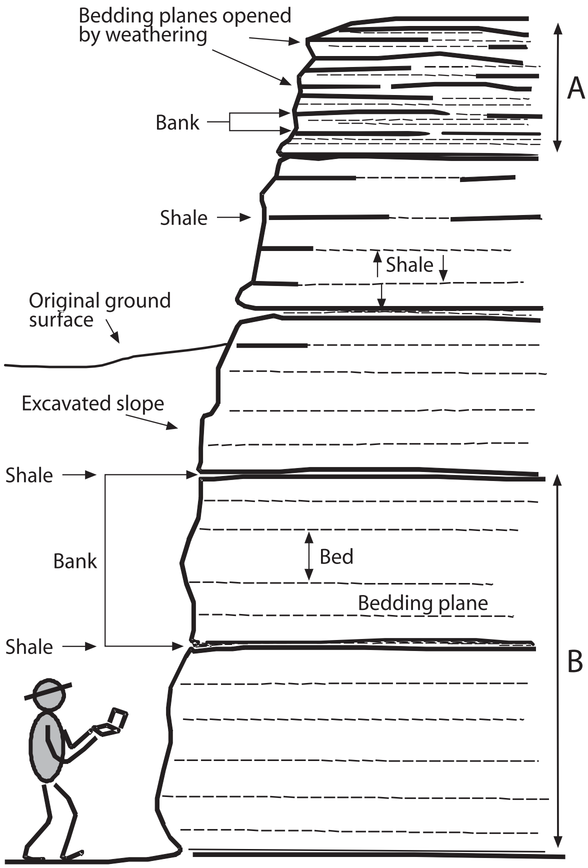


Fig. 3.12. Banks and beds

3.1.5 Spacing

It is quite clear that in describing a ground mass mention must be made of the spacing of discontinuities, e.g. the perpendicular distance between two discontinuities from the same set. This spacing is relevant to problems of slope stability, tunnel stability, excavation, groundwater flow and foundation bearing capacity. The terminology, which is standard for the description of discontinuity spacing, is given in Table 3.4. However, the use of this terminology does pose some problems when applied to bedded rocks. Assume the rock slope illustrated in Fig. 3.12 is in sandstone. The upper part of the slope is old, the lower part newly excavated, but because some, mostly higher, bedding planes have been opened by weathering and some assemblies of beds are separated by thin shaley units, the spacing of the bedding planes does not reflect the thickness of the geotechnical layers which would be of significance

Table 3.4. Terminology for bedding and discontinuity spacing (after BS EN ISO 14689-1)

Integral discontinuities	Spacing	Mechanical discontinuities
Very thickly bedded	> 6 m	Extremely widely spaced
Very thickly bedded	2 – 6 m	Very widely spaced
Thickly bedded	0.6 – 2 m	Widely spaced
Medium bedded	0.2 – 0.6 m	Medium spaced
Thinly bedded	60 mm – 0.2 m	Close spaced
Very thinly bedded	20 –60 mm	Very close spaced
Thickly laminated	6 –20 mm	Extremely close spaced
Thinly laminated	< 6 mm	Extremely close spaced

Note: For metamorphic rocks 'bedded' becomes 'foliated', 'thickly laminated' becomes 'closely foliated' and 'thinly laminated' becomes 'very closely foliated'. According to this standard bedding (integral discontinuities) is described using the numbers 2 and 63; e.g. Thickly laminated, 6.3–20 mm, and so on. A practical compromise is shown here.

Table 3.5. Rock block dimensions and description (after BS EN ISO 14689-1)

First term	Maximum dimension
Very large	> 2 m
Large	0.6 – 2 m
Medium	0.2 – 0.6 m
Small	60 – 200 mm
Very small	< 60 mm
Second term	**Shape of block**
Blocky or cubic	Equi-dimensional
Tabular	Thickness much less than length or width
Columnar	Height much greater than cross section

in, say, slope stability. To overcome this problem a further descriptive terms is needed. The author suggests the use of the term 'bank' to indicate a layer of geotechnical significance. Thus the sandstone in zone A in the slope would be described as 'thinly banked thinly bedded' while that in zone B would be 'thickly banked medium bedded'.

The shape and size of rock blocks depends upon the spacing of the discontinuities bounding them. The terminology proposed by British Standards (1999) is given in Table 3.5 and is designed to indicate the size of rock blocks that may come out of a quarry or be encountered in a tunnel.

3.2 Weathering

Most civil engineering works occur close to the surface and the process of 'weathering' has affected most groundmasses at shallow depth. Because of this, weathering of both engineering soils and rocks is one of the most important problems with which the engineering geologist has to contend. Weathering implies decay and change in state from an original condition to a new condition as a result of external processes. A review of weathering processes has been given by various authors (Anon 1995; Price 1995).

A most obvious sign of weathering is brown staining by the oxidation of iron bearing minerals. While solution is an agency involved in most forms of chemical weathering, it is most prominent in limestones and rocks containing halite and gypsum. In limestones, such as chalk and calcarenites, near vertical solution pipes may form extending from the limestone/overburden surface to depths of perhaps as much as 40 m. These are usually wholly but sometimes partially infilled by overburden deposits which have flowed or been washed in from above. Crystalline limestones are often strong enough to support the development of underground cavern systems through which rivers may flow. The limestones closely adjacent to such systems may be entirely unaffected by the solution weathering and be as strong as the fresh material.

Weathering takes place in all environments but is most intense in hot, wet climates where weathering may be expected to extend to great depths. While weathering may reach great depths in limestones, and rocks containing halite and gypsum, it is slow to do so and the style of weathering may change if climatic conditions change. In the northern hemisphere, where large areas have been subjected to phases of glaciation with intervening warmer periods the weathered nature of the groundmass as presently seen may reflect these changes. Thus, in the most recently glaciated areas all weathered materials may have been carved away by ice and almost fresh rock exposed. Beyond the boundaries of glaciation, weathering may reflect periglacial conditions (Higginbottom and Fookes 1971). It is generally thought that, at the end of the last glaciation, sea level rose by about 100 m. This implies that in seas less than 100 m deep, sea bed materials have been exposed to sub-aerial weathering for a substantial period of time and their properties may reflect this. Indeed, if at any time in geological history, any material has been exposed above surface, it would have been subject to weathering. Thus in Western Australia, some of the laterites exposed (and perhaps covered by more recent deposits) are thought to be of Tertiary age (Geological Survey of Western Australia 1974).

3.2.1 Influence of Weathering on Rock Mass Properties

Weathering weakens rocks. Table 3.6 shows how rock material and rock mass properties can change with weathering. The table in which the weathering is given by grades (I = fresh to VI = residual soil) shows the considerable difference in material properties that are a consequence of weathering. An ordinary geological map showing the granodiorite or the dolerite in the table would not give any indication of the

Table 3.6. Examples of variations in engineering properties of dolerite and granodiorite as a consequence of material and mass weathering. Columns 2 to 9 inclusive refer to material properties and weathering; columns 10 and 11 refer to the mass. Note: not all rocks and rock masses may weather this way

Grade	Density γ (kN m^{-3})	Porosity n (%)	Unconfined compressive strength UCS (MPa)	Unconfined tensile strength UTS (MPa)	Static deformation modulus Es (GPa)	Seismic velocity		Schmidt hammer number H[c]	Rock mass friction (deg)	Rock mass cohesion (KPa)
						Longitual wave, Vp (m s^{-1})	Shear wave Vs (m s^{-1})			
Dolerite[a]										
I–II	28.04	0.4	160 – 180	42–48	16.5	4 000–5 000		64 (60–75)		
III	27.64	0.5	83 – 160	15–42	3.3	2 500–4 000		53 (50–60)		
IV	26.96	1	58 – 83	11–15		1 800–2 500		45 (35–50)		
V	26.18	3.2	24 – 58	2–11		1 400–1 800		25 (20–35)		
Granodiorite[b]										
I	25.6–26.96	2.6–0.4	111 – 165		31 – 34	3 749–4 968	2 520–2 883		47	17
II	25.7–26	3.3–5.9	60 – 97		14.5 – 15.3	1 737–2 377	1 545–1 840		46	16
III	25.1–25.7	1.5–2.3	33 – 48		9.4 – 11.5	1 545–1 840	1 082–1 139		38	14
IV	22.9–25.1	5.2–6.1	8 – 24		3.9 – 5.9	499–1 447			17	8
V	19.8	24	0.1		0.002 – 0.013				6	3
VI	14.7	44								

[a] Dolerite data from author's own files; dolerite once exposed at Stirling Castle, Scotland.
[b] Granodiorite data from Krank K.D. and Watters R.J. (1983), except rock mass friction and cohesion. Granodiorite rock mass friction and cohesion from slope back ana lysis in Granodiorite in the Falset area, Spain, from Hack H.R.G.K. (1998). Grade scales follow the classification given in Table 3.7.
[c] Guide to values that might be expected in granite using an N hammer.

weathered condition of the rock, yet clearly this has great influence on the likely engineering behaviour of that rock material and mass. Because of this, it is customary to describe the weathered condition of the rock in all engineering geological descriptions of material or mass.

3.2.2
Susceptibility to Weathering

The lifetimes of most structures for civil engineering and infrastructure are in the order of 50 to 100 years. To guarantee the safe and sound design for the whole lifetime it is important to know what the geotechnical properties of the soil or rock mass are going to be at the end of this time span, i.e. the susceptibility to weathering of the soil or rock mass. If, for example, a slope is excavated in a sandstone in which the cement between the grains consists of gypsum it can be expected that in a moderate climate the gypsum will dissolve and the sandstone rock mass changes into a soil mass of loose sand grains within a few years. A slope made in fresh granite is not expected to undergo any major changes within 100 years in a moderate climate. The influence of weathering on intact rock used as building or grave-stones has been studied and also rates for weathering have been established. For most soils and rocks it is also reasonably well known how they deteriorate over long (geological) periods. However, for 50 to 100 year time spans very little is known. Research has been done to the weathering rates in underground excavations and its influence on rock mass classification ratings (Laubscher 1990) (see Chap. 4). For the extremes given in the slope example above, it is generally not difficult to make an estimate of the changes of properties due to weathering. For many other soil or rock masses, for which it is not so clear, the engineer has to estimate the susceptibility to weathering based on other exposures of known excavation date. If these do not exist, engineering judgement has to be used.

3.2.3
Standard Weathering Description Systems

Weathering can be described following a relatively simple scheme, such as in Table 3.7 which is compiled from recommendations given in BS 5930 (1981) and the report of the Engineering Group Working Party on Core Logging (Anon 1970), or from more elaborate schemes of which BS5930 (1999) is an example (Fig. 3.13, Table 3.8). While the simple scheme is easy to use, it often does not fit a particular state of weathering for a particular material in a particular environment. The more elaborate schemes have more flexibility to fit all sorts of materials and environments, but have the potential of being over complex and not understood.

The factors entering into the description of weathering are the condition of the discontinuities (joints, bedding planes, foliations etc.) and the condition of the material between the discontinuities. Weathering begins on the discontinuities that transmit water. Increasing weathering affects more and more discontinuities and eventually starts to affect the rock material. Usually rock mass weathering is assessed from a number of boreholes, and natural or artificial outcrops. Boundaries between weathering grades can never be more than approximate.

Table 3.7. Standard terminology for description of weathering of rock cores, outcrops and material

Weathering description	Grade No.	Rock core grades[a]	Rock outcrop grades[b]	Rock material descriptive terms[b]
Fresh	I (A)	No visible sign of weathering.	No visible sign of rock material weathering, perhaps slight discolouration on major discontinuity surfaces.	Rock material weathering can be described by using terms such as:
Faintly weathered[c]	I (B)	Weathering limited to the surface of major discontinuities.		
Slightly weathered	II	Weathering penetrates through most discontinuities, but only slight weathering of rock material.	Discolouration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering.	Discoloured: The colour of the original fresh rock material is changed and is evidence of weathering.
Moderately weathered	III	Weathering extends throughout the rock mass but the rock material is not friable.	Less than half the rock material decomposed or disintegrated to a soil. Fresh or discoloured rock is present as a continuous framework or as corestones.	Decomposed: The rock is weathered to the condition of a soil in which the original material fabric is still intact, but some or all of the mineral grains are decomposed.
Highly weathered	IV	Weathering through discontinuities and material and the rock material is partly friable.	More than half the rock material decomposed or disintegrated to a soil. Fresh or discoloured rock is present as a discontinuous framework or as corestones	Disintegrated: The rock is weathered to the condition of a soil in which the original material fabric is still intact. The rock is friable but the mineral grains are not decomposed.
Completely weathered	V	Rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.	All rock material is decomposed and/or disintegrated to soil. The original rock mass structure is still largely intact.	The stages above may be qualified by using terms such as 'partially', 'slightly', 'wholly'.
Residual soil	VI	A soil material with the original texture, structure, and mineralogy of the rock completely destroyed.	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume but the soil has not been significantly transported.	

[a] After Geol. Soc. Enging. Group Working Party on 'The logging of cores for engineering purposes' (Anon. 1970). [b] After BS 5930 (1981). [c] Faintly weathered is seldom found in descriptions and may be considered more-or-less obsolete. Grade I only becomes I (A) if faintly weathered is used.

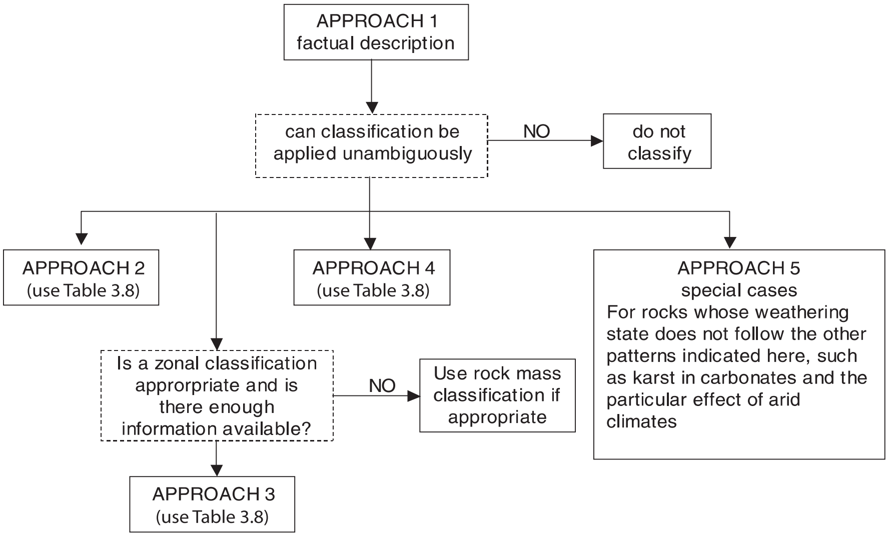


Fig. 3.13. Scheme for description of weathering (after BS5930 1999). See also Table 3.8. Note: BS EN ISO 14689-1 does not, at present, recommend Approaches 2 and 3

3.2.4 Weathering Description and Zonation

Weathering is a gradational feature and to deal with such features the usual approach is to impose boundary conditions within them so that they are divided into various grades defined by a range of characteristics. In reviewing the applicability of the grades of weathering proposed in the various systems mentioned above it is well to remember the purpose of describing weathering i.e. is to assess its significance with regard to engineering projects to be conducted in or upon the weathered rock mass. This significance depends upon two factors, first the change in engineering properties of the rock material and second, the volumetric regularity or irregularity of this reduction within the rock mass. Regarding the second, the writer distinguishes four basic types of mass weathering. These are:

1. *Uniform weathering*. A gradual decrease of weathering grade and intensity with depth, in thick strata of homogeneous lithology.
2. *Complex weathering*. An irregular weathering profile in layered lithologies that have different susceptibilities to weathering. It may mean that more weathered strata lie under less weathered strata, particularly if the strata dip from surface outcrop.
3. *Corestone weathering*. In many, mostly coarser grained, igneous rocks rounded '*corestones*' of almost fresh rock may be surrounded by very decomposed highly friable material, similar to a compact sand. The corestones become larger with depth.
4. *Solution weathering*. Carbonate and most salt rocks weather by dissolution. Joints and bedding planes become open and underground caverns may develop. In strong crystalline limestones, karstic conditions may result. In the weaker calcarenites,

Table 3.8. Description state of weathering (after BS5930 1999). This scheme no longer accords with BS EN ISO 14689-1 which is considered by some to be inferior to the old BS 5930 1999

Approach 2: Rock is moderately strong or stronger in fresh state (not in BS EN ISO 14689-1) *Uniform materials*		
Grade	**Classifier**	**Description**
I	Fresh	Unchanged from original state
II	Slightly weathered	Slight discolouration; slight weakening
III	Moderately weathered	Considerable weakened, penetrative discolouration; large pieces cannot be broken by hand
IV	Highly weathered	Large pieces can be broken by hand; does not readily disintegrate (slake) when dry sample immersed in water
V	Completely weathered	Considerably weakened; slakes in water; original texture apparent
VI	Residual soil	Soil derived by in-situ weathering but having lost retaining original texture and fabric
Approach 3: Heterogeneous masses (mixture of relatively strong and weak material) (not in BS EN ISO 14689-1) *Heterogeneous masses*		
Zone	**Description (2)**	**Typical characteristics**
1	100% grades I–III	Behaves as rock; apply rock mechanics principles to mass assessment and design
2	>90% grades III <10% grades IV–VI	Weak materials along discontinuities; shear strength stiffness and permeability affected
3	50 to 90% grades I–III 10 to 50% grades IV–VI	Rock framework still locked and controls strength and stiffness; matrix controls permeability
4	30 to 50% grades I–III 50 to 70% grades IV–VI	Rock framework contributes to strength; matrix or weathering product control stiffness and permeability
5	<30% grades I–III 70–100% grades IV–VI	Weak grades will control behaviour. Corestones may be significant for investigation and construction
6	100% grades IV–VI	May behave as soil although relict fabric may still be significant
Approach 4: Moderately weak or weaker in fresh state (permitted by BS EN ISO 14689-1) *Classification incorporates material and mass features*		
Class	**Classifier**	**Description**
A	Unweathered	Original strength, colour, fracture spacing
B	Partially weathered	Slightly reduced strength, slightly closer fracture spacing, weathering penetrating in from fractures, brown oxidation
C	Distinctly weathered	Further weakened, much closer fracture spacing, grey reduction
D	De-structured	Greatly weakened, mottled, lithorelicts in matrix becoming weakened and disordered, bedding disturbed
E	Residual or reworked	Matrix with occasional altered random or apparent lithorelicts, bedding destroyed. Classed as reworked when foreign inclusions are present as a result of transportation

calcisiltites and calcilutites (chalk), solution pipes, often infilled with materials from above, may have penetrated deeply into the strata.

While a complete volume of rock mass might be described using the terms given above within that mass the weathered nature must be described in greater detail in order to more closely target an engineering problem. This is perhaps best done with regard to the engineering significance of the mass weathering observed. The following terms are proposed:

i *Effectively unweathered*. The weathering of the rock mass is such that any engineering work may be constructed on or in it without regard to the weathered condition as found.
ii *Significantly weathered*. The weathering of the rock mass is such that some regard must be taken of the weathered condition of the rock mass in the design and construction of some particular engineering works. For example, weathering of discontinuities could imply an impairment of shear strength if the work involves the construction of rock slopes, but such weathering would have less effect on the design of foundations on the rock mass.
iii *Severely weathered*. The weathering of the rock mass is such that the weathered condition of the mass dominates the design and construction of any engineering work to be constructed on or in it. This implies both weathering of discontinuities and materials.
iv *Residual soil*. Sufficient of the rock material is decayed to the geotechnical condition of a soil to make the mass behave as a mass of soil. It may be important, because of their influence on, for example, slope stability, to distinguish between those residual soils that are structureless and those that have relict discontinuities.

The boundary between (*iii*) and (*iv*) is difficult to establish. A rock mass in condition (*iv*) may contain relict rock blocks which, if in continuous contact, may present the engineering behaviour of a rather weak rock mass. If the relict blocks are not in contact, the total mass may behave as an engineering soil. If the volume of soil material exceeds about 50% of the mass then the behaviour of the total mass should be that of a soil and be categorised as residual soil (*iv*); if the volume of soil is less than about 30% then behaviour should be that of a severely weathered rock mass (*iii*). It will be difficult to estimate such percentages in this transition zone between soil and rock mechanics and estimates are most likely to be subjective.

The terms given above could be used to describe zones of weathering within a rock mass. Such zones might be observed in outcrop and boundaries evaluated. The author has suggested a ratings system to aid establishing such boundaries (Price 1993). It also provides a numerical estimate of weathering that may form part of a rock mass classification system and has been so used by Laughton and Nelson (1996) in considering the rock mass parameters that are necessary to predict tunnel boring machine performance. The data required to zone a rock mass into categories (*i*) to (*iv*) may come from natural or artificial surface outcrops but is derived most commonly from boreholes. Thus, in core descriptions note must be made of the weathered condition of discontinuities and the extent of decay of rock materials.

In rock mechanics the condition of discontinuities is of major importance and it would seem logical to describe discontinuity weathering to provide a link to assessments of discontinuity strength. In both integral and mechanical discontinuities, three main conditions may be considered to exist. These are:

- *fresh:* no sign of weathering
- *surface stained*: no or little penetration of weathering into the wall rock (so that the discontinuity asperities have the strength of fresh rock)
- *surface weathered*: weathering penetrates to a depth greater than the roughness or undulations of the discontinuity surface (so that the discontinuity asperities have a strength less than that of fresh rock)

In the case of rock materials similar simple terms may be applied, namely:

- *fresh:* no sign of weathering
- *stained*: the material is weathered but without obvious loss in strength
- *decayed:* deeply stained and with obvious loss of strength, perhaps friable

In the case of solution weathering an additional term, "*absent by solution*", might be applied.

Achieving a weathering zonation of a rock mass investigated largely by boreholes is greatly aided if the style of weathering in outcrops of similar rock can be studied, even if these are not located close to the site. This may help the investigator appreciate the relationship of the linear data given by boreholes to the volumetric reality of the rock mass.

3.3 Ground Mass Description

A volume of ground may have some exposure that can be studied but in most cases it has to be investigated and described with the aid of core recovered from boreholes drilled for that purpose. The borehole core provides a description of the vertical profile but at some stage a general description of the volume is needed.

3.3.1 General Mass Description

Most ground masses on or in which engineering works are to be performed can be considered composed of a series of *geotechnical units* each distinguished by particular mass properties. The range of properties defining a unit is assigned by the investigator, commonly with regard to the particular type of engineering work to be performed. Thus a ground mass which is to be excavated to form a large road cutting might be divided into units relative to ease of excavation, to reuse of the excavated materials as embankment fill, or with regard to slope stability. The units related to one engineering process are not necessarily the same as those for another process for their boundaries would be defined by different parameters.

Whatever the units, the description of the mass must include descriptions of the units, their boundaries, their discontinuities, their water content and the level of groundwater within them. A common way of obtaining this information is to drill holes into the ground and describe the samples (cores) recovered from them. The process of describing such cores is known as core logging.

3.3.2 Core Logging

Poor or inappropriate core logging can lead to major civil engineering claims. Accordingly, accurate core logging is one of the most important activities for the engineering geologist. Cores taken as part of a site investigation for an engineering project should be logged by an engineering geologist rather than a geologist for the style, content and emphasis of the descriptions will depend on the recognition of the importance of the geological factors revealed in the cores to the particular engineering project. If the logging of a specific geology feature, such as structural geology, fossil content, etc., is required, a geologist specialised in the required feature should rather do this additionally.

Basic Requirements for Good Core Logging

The contractor and consultant must ensure that the cores are placed in good, well-labelled core boxes and are stored so that their condition as recovered from the borehole is maintained. They should not be allowed to dry out, be eroded by rain, frozen, etc. A well-lit large table in a clean, dry, covered store should be provided as a logging area. Help should be available to move core boxes.

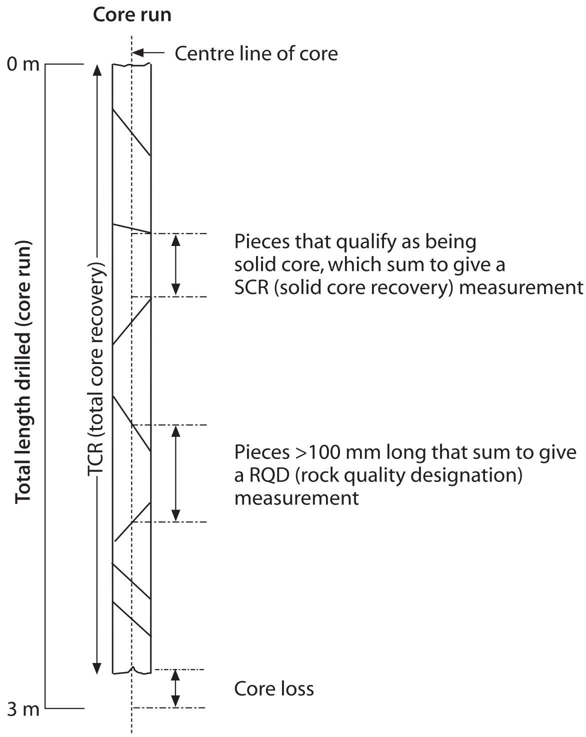
The Process of Core Logging

Ideally, all the cores should be logged in one exercise and the logger must be fully aware of and understand the nature of the engineering work so that the log gives the information necessary for the design of that work. Cores may be logged as they come out of the borehole for their description may influence the further development of the investigation but this provisional log should not be the final log.

For the final log the logger must look at all the soils and rocks and decide provisionally what they should be called and in what detail they should be described. A published system of description should be used and its selection justified. The way in which the cores are described will depend upon the geology, the engineering work, and the nature of the problem that the geology poses for that work. Photos of the core boxes against a scale should be made for later reference.

The cores should be sorted out so that they fit together as well as possible to allow *Total Core Recovery* (*TCR*) and, if required, *Solid Core Recovery* (*SCR*) and *Rock Quality Designation* (*RQD*) to be measured. *TCR* is the length of core recovered divided by the length of hole drilled (the core run) expressed as a percentage (Fig. 3.14). *SCR* is the total length of solid core cylinders as a percentage of the core run. Care has to be taken in defining what is meant by '*solid*' (Norbury et al. 1986). *RQD* is the total length of core sticks longer than 4 inches (now commonly converted to 100 mm) expressed

Fig. 3.14. Measurements that may be made on rock cores



as a percentage of the core run. According to Deere (1968) "*'The Rock Quality Designation' (RQD) is based on a modified core recovery procedure which, in turn, is based indirectly on the number of fractures and the amount of softening or alteration in the rock mass as observed in the rock cores in a borehole.*" Fractures induced by handling or the drilling process should not be counted (the pieces broken by such fractures should be fitted together and their total length measured) and the pieces counted should be 'hard and sound'. If all the sticks of core measured are less than 100 mm long, *RQD* is zero and if there are no breaks in the core *RQD* is 100%. This raises the possibility that if all sticks were 99 mm long, *RQD* would be 0% and if all were 101 mm long *RQD* would be 100%. Clearly measuring mechanical discontinuity spacing against a fixed baseline has some disadvantages. Some workers have measured *RQD* against several baseline lengths to give a guide to rock block size; Edmond and Graham (1977) did this to assess block size in a rock mass through which a tunnel was to be driven.

Some rock materials are composed of thinly interlaminated sandy and clayey bands. They may come out of the core barrel as solid sticks of rock but on drying out separate into thin discs, splitting along the clayey layers. *RQD* measured on the dried out cores would be very much lower than on those just recovered from the borehole. The logger is then faced with the difficult choice between including or ignoring the drying-out breaks. Inclusion gives a quality assessment of the rock material; exclusion gives an *RQD* related only to genetically, tectonically, or geomorphologically induced discontinuities. Such discing may also be a consequence of stress relief. Whether or not included in *RQD* measurements such discing would merit description and discussion in the report.

The uncertainties concerning the measurement and meaning of *RQD* suggest that *RQD* should be abandoned and the author does not recommend its use. It has, however, been incorporated in some Rock Mass Classification systems, and because of this will probably continued to be measured until these systems are modified or abandoned. The best reason to abandon *RQD* is to take advantage of the opportunities that computer technology offers. It should be appreciated that the *RQD* calculated from core applies only to the direction in which the hole from which the core is recovered, was drilled. An alternative to *RQD* is to record the number of fractures per given length of borehole. This may be done for metres depth, for each lithology or each engineering geological unit distinguished.

Recording

The logging process falls mostly into two parts; first describing the materials and second, describing the discontinuities. It is often a good idea to do the discontinuity measurements (stick length, joint and bedding dip, etc.) first. It is better to measure the depth and dip of every mechanical discontinuity so that thereafter any calculations (*RQD*, frequency etc.) can be made. If using a rock mass classification system somewhere in the project remember to gain the data to fill in the ratings. This may involve noting joint roughness, infilling etc., and also means that the rock mass classification system must be chosen before beginning logging.

The process of making the discontinuity measurements gives another chance to look at the rocks and consider the descriptions to be used. Geologically this is quite easy, but engineering geologically rather difficult for the observer seeks to identify geological features of significance to that particular engineering work. While it is quite easy to become absorbed in the engineering geological and rock mechanical side of logging, it must not be forgotten that a proper description of the geology must also be made. This is necessary for correlation between boreholes and with surface data. In writing the descriptions, the word order should be consistent, following a recognised system such as that offered in BS5930 (1999). Any departures from that system should also be consistent and the reason for them explained in the report.

Features in the cores resulting from drilling, have to be identified. Thus, ends of core runs tend to be more broken while core springs may produce scratch marks simulating bedding. These should be noted on the record. Because the core spring is situated above the cutting face of the drilling bit extraction of the core may leave a core stub at the bottom of the borehole. If this is recovered in the next run then the core recovery for that run could exceed 100%. If a stub is left at the bottom of the hole and the next core run completely fills the core barrel then cores may be crushed.

Logging the strength of the rock cores by some form of simple test may be necessary. This may be done using such devices as the Point Load Tester, but these break the core and may thus reduce the amount of core available for accurate laboratory testing. The author does not recommend this test. The ISRM 'Suggested Methods' (Brown 1981) includes a method of using the Schmidt hammer on cores in the laboratory, but the author prefers cross diameter acoustic logging or the use of the Equotip (a kind of mini-Schmidt hammer).

If samples are extracted then something should be put in their place (ideally a block of wood cut to length and labelled) and noted. Core boxes should have a logging record pinned to the inside of the lid on which the logger notes who and when the cores were logged and also records any samples taken.

In the initial stages of logging, errors of the driller in placing the cores in the box may be encountered. It is useful to be familiar with top and bottom criteria to identify cores that may have been put in upside down. Before logging rock it is useful to wash the cores and look at them wet, for wet cores show more than dry ones. In soils a little drying often helps reveal delicate sedimentary structures such as lamination. Cores may have to be broken to determine the rock type. If this is done note the locations of breaks made; an indelible felt pen is useful for this. If possible, at the end of the logging of a number of boreholes, all logs should be examined again to check consistency of description.

Discontinuity Orientation

The dip of discontinuities may readily be measured on cores from a vertical borehole, however, the direction of dip can only be measured with special, and usually expensive, equipment. If the direction of dip of the dominant integral discontinuity (bedding or foliation) is known, it is possible to measure the direction of dip of other discontinuities relative to it.

Daily Logging Rate

A good core logger, giving a full engineering geological description for a complex project, can usually manage to log about 30 m of core a day before mental indigestion sets in. It is important to be comfortable to do the work; logging in the mud in a mosquito-infested swamp usually produces poor logs. It is much better to do the work in a well-lit, air-conditioned hut, sitting in a comfortable chair with a pot of coffee at hand.

3.3.3
A Theoretical Example

Figure 3.15 is a drawing of three core boxes to illustrate some of the points made above. The Equotip rebound values have to be converted to unconfined compressive strength with the help of Fig. 3.7. The drawing Fig. 3.15 is two dimensional and limited in what it shows by the constraints of the graphics programme used for its construction. Details are as follows:

1. Three core boxes are illustrated containing cores obtained by rotary core drilling from 15.00 to 38.06 m depth. The upper two boxes contain 160 mm diameter core, the lowest 120 mm diameter core. The driller has marked the beginning and end of each core run, which are measured from the drill rods, and has also estimated the depth

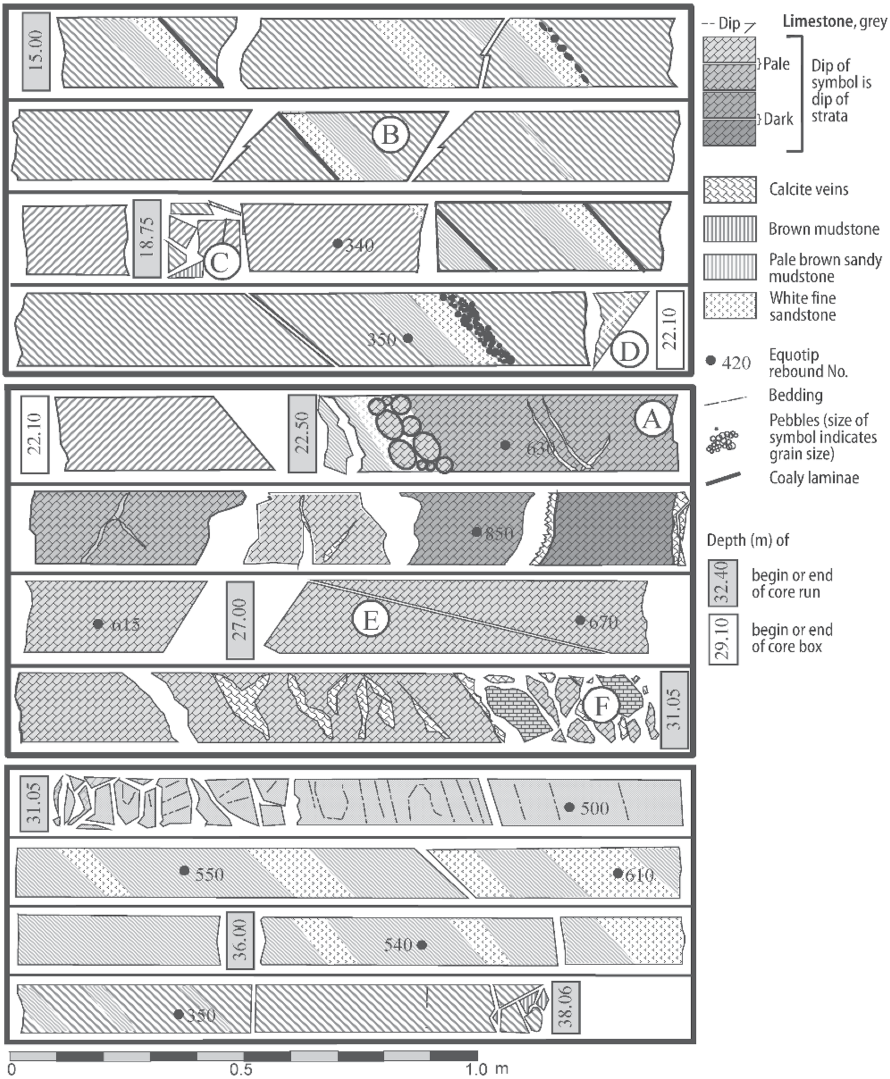


Fig. 3.15. A theoretical example of a series of core boxes

of the end or beginning of the core in each core box. If there has been any core loss then these latter depths may be inaccurate.

2. The cores have been placed in the core boxes to more-or-less fill them without breaking the cores more than is necessary. However, there will be some 'fitting' breaks and one of the first tasks in logging is to identify them. One such break is to be found at point 'A'; there are others. They would not be counted as natural discontinuities for *RQD* measurements etc.

3. The cores in the first box show a 'fining upwards' succession of rocks (conglomerate ⇒ sandstone ⇒ sandy mudstone ⇒ mudstone ⇒ coal). The stick at 'B' shows this succession inverted and has thus been placed upside down in the box.
4. The rocks at 'C', just below the 18.75 m core run marker are rather broken and may represent an over-drilled core stub, left in the hole at the end of the previous run and disturbed at the beginning of the next run.
5. The fragment of rock at 'D' would not appear to be in the correct place and may come from the bottom of the run at 22.50 m.
6. The joint at 'E' may be open. If, when its two surfaces are placed in contact, the thickness of the core is less than the diameter drilled then the joint is open and the aperture can be calculated.
7. At 'F', the core is very broken and at this point, the driller decided to reduce core diameter, presumably as a result of encountering drilling difficulties. Such difficulties would be recorded in the daily drilling log. The implication is thus that the breaks in the core above and below 31.05 m are a consequence of natural phenomena and not the drilling itself. The same remark could apply to the broken core just above 38.06 m but here it might also be the core cracking in the vicinity of the core spring on breaking off the core to pull it to surface.

Describing Lithologies

There are clear differences between the strata above about 22.70 m, from 22.70 to about 31.00 m and below this last depth. In the case of the strata above 22.70 m there would appear to be cyclic deposition in fining up sequences. There are various ways of describing the sequence. If the project concerns putting foundations on the rock mass it would probably be sufficient to record the number of cycles of deposition and to indicate their nature. If the rocks are likely to be exposed in a slope or tunnel wall it would be prudent to emphasise the presence of the coaly laminae which could squeeze or which could form planes on which sliding might take place. Alternatively, a more factual approach could be taken and the rocks closely described layer by layer with measurements of depth and thickness.

At about 22.75 m a conglomerate is found composed of limestone pebbles and may well mark an unconformity. A check on the relative ages of the limestone and overlying cyclic deposits would be of value for the unconformity surface could be that of a very irregularly eroded landscape. The limestones show fractures, perhaps solution opened, and healed by calcite veining. The end of the core shows solution features, there is significant core loss, and the driller may have recorded encountering cavities as the hole was drilled.

The limestones are not all the same. They are generally pale grey but two pieces, perhaps once joined by a calcite vein, are dark grey and show different bedding dips. This could be a consequence of faulting. Below about 30.90 m the mudstones and sandstones are again encountered although lacking indications of cyclic deposition. However, while their bedding below about 32 m is regular and thin between 31 and 32 m it becomes variable with indications of folding. This can be syn-sedimentary, but is probably faulting.

Structure and Strength

Measurements of bedding dip would be taken as frequently as the dip varied. The relative orientation of joints could also be measured. Strength may be assessed by the Equotip rebound number or some other method but indicated in the log by the terms used in the scale of strength for rocks, e.g. moderately strong. The description of the cores shown in Fig. 3.15 is given as in Chap. 7.

3.4 Further Reading

Eddleston M, Walthall S, Cripps JC, Culshaw MG (1995) Engineering geology of construction. Geological Society of London (Engineering Geology Special Publication 10)
Geological Society of America (1957–1978) Engineering geology case histories 1–11. Geological Society of America
Harris C (ed) (1996) Engineering geology of the Channel tunnel. Thomas Telford, London