

## Geological Materials

### 2.1 Important Characteristics of Geological Materials

Engineers work with large volumes of soil and rock which will contain variable amounts of fluid in their pores and fractures. It is helpful to distinguish the material from which these volumes are made from the mass which they form. Sediments are made from particles, big and small, and “rocks” are made from rock!

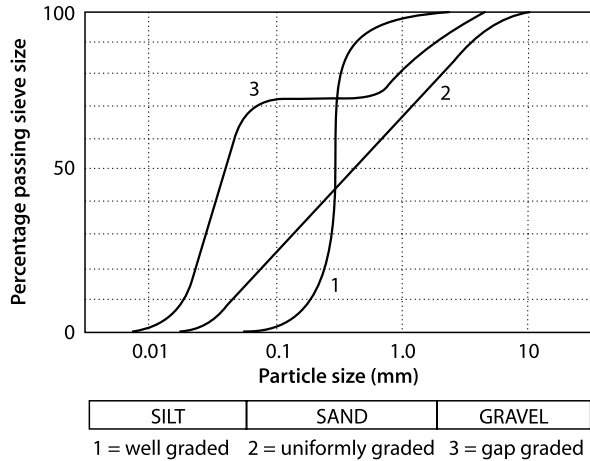
#### 2.1.1 Sediments

The coarsest sediments are those produced by landsliding and glaciation which may transport fragments of rock so large that an examination at close range may fail to recognise that they have been displaced. More commonly, rock fragments found below eroding cliffs may be many tonnes in weight. Such very large fragments may be further eroded during river transportation to gravel and boulder size. These fragments are recognisably rock but, as they disintegrate to yet smaller and sand-size grains, the grains tend to be largely of single minerals. The type of mineral of which they are composed will depend upon the source rock and the degree of abrasion suffered during transportation. Thus the most common sand-forming mineral is quartz, but in limestone areas the grains may be predominantly calcareous. If there are local sources of less erosion-resistant minerals, such as mica, these may be found mixed with more resistant minerals transported from distant sources. Grains of all sizes will be, to some degree, rounded by abrasion during transportation and the degree of roundness achieved is of geotechnical significance, for angular grains tend to interlock and give greater shear strength than more rounded grains. The distribution of grain sizes in sediment may vary (Fig. 2.1).

Uniformly graded sediments comprise more or less equally distributed representatives of many grain sizes. Well graded sediments are mostly of one grain size while gap graded sediments lack a range of grain sizes. Well graded sediments tend to have greater porosity and thus greater permeability than uniformly graded sediments because there are fewer finer particles to fill pore spaces between larger particles. At silt size, the particles and the pore spaces between them are very small so that permeability is very low and movement of water slow. For this reason, attempts to compact water saturated silt may result in raised pore water pressures and the subsequent liquefaction of the deposit.

All of the sediments so far described are granular and the grains do not adhere to each other. The yet finer grained sediments, clays, are formed of particles less than

Fig. 2.1. Types of grading in granular sediments



0.002 mm in diameter and often much smaller, and are commonly very small plates of clay minerals bonded together by electro-chemical forces. This bonding gives the clay cohesion allowing the material to be moulded. Clay minerals result mainly from the weathering of other rock forming minerals. Thus kaolinite results from the weathering of feldspars in granitic rocks. Clay micas are a common constituent of clays and range, with increasing degradation, from the primary mineral muscovite to the secondary minerals sericite and, at the end of the series, illite. Muscovite has a low content of combined and adsorbed water but this, together with interstitial water, increases to the end of the series. The montmorillonite group of clay minerals, now known by the earlier term ‘*smeectites*’, are renowned for their capacity to expand on taking in water and give rise to extreme swelling. Members of the chlorite group of clay minerals also have a capacity for swelling.

Organic deposits of plant remains may be laid down in swampy environments to form highly compressible peat deposits. Plant remains may also be mixed intimately with clays to form organic clays and have a significant influence on the behaviour of these materials.

2.1.2  
Intact Rock Materials

‘Intact’ rock is commonly taken to mean a piece of rock about the size of a laboratory test specimen (usually a cylinder of core no larger than about 100 mm long and 50 mm diameter) without obvious cracks or breaks. Most rocks are formed of mineral grains or other rock fragments bonded together in some way. The amount of pore space present, the size of the pores and the nature and quantity of the cement has a major effect on the mechanical properties of the intact rock material. In general terms, the greater the porosity, the weaker the rock and, of course, the weaker and less abundant the cementing mineral, the weaker the rock also.

No rock can be stronger than the minerals of which it is composed and the nature of the rock forming minerals has a dominating influence on the behaviour of some

rocks. Thus rocks formed from soluble minerals such as calcite and gypsum, pose problems from past solution and future solubility. Evaporites are effectively monomineralic rocks whose formation is associated with the evaporation of mineral charged water. They include such minerals as halite (rock salt), gypsum and anhydrite and potash salts (such as a carnallite and polyhalite). *Evaporites*, particularly rock salt, will flow under pressure and may be found in salt domes pushed, and still moving from their original position, upward into overlying strata. The susceptibility of these rocks to creep gives problems in mines and tunnels.

### 2.1.3

#### Fluids and Gasses

The main fluids of importance in engineering geology are water and oil. It is almost impossible to over-emphasise the importance of water in determining the engineering behaviour of geological materials and masses. Water is almost incompressible and when present in the pore spaces of a material, if only in small amounts, will modify the behaviour of that material under stress. The behaviour of clays, in particular, is very much dependent on moisture content. Water is seldom pure and contains dissolved minerals, such as sulphates, which may react with engineering materials. Salinity and acidity limits its use in such processes as the manufacture of concrete. On freezing, water expands and ground heave and ice wedging are important mechanisms causing ground disruption and slope instability.

Oil as a fluid in the ground is not often of direct importance in civil engineering unless there are projects proposed at great depth in which oil may be inadvertently encountered. However, it is a common cause of contamination in industrial sites and becomes a component of sediments that has to be considered when such sites are re-developed.

Many different gasses can occur underground particularly in volcanic areas but, apart from air, in the pores of ground above the water table the most important commonly encountered gas is methane. Methane is explosive and is associated with underground works in carbonaceous rocks, but may be also be derived from peaty organic matter contained within unconsolidated clays and silts. It is non-toxic but may be asphyxiating by reducing the oxygen content in the air. The author has had an unfortunate experience with hydrogen sulphide (the rotten egg gas) which is highly toxic and comes from the decay of organic materials and the action of acid waters on pyrites. It was released during a water well pumping test in the Middle East; at first the smell warned of its presence but it then seemed to disperse. It was however the author's sense of smell that was deadened and who, with continued inhalation, turned slightly blue and was carried off unconscious.

## 2.2

### Description of Materials

The identification and description of soil and rock in samples or in the mass is commonly one of the tasks with which the young geotechnologist is entrusted in his or her first employment. It thus may sometimes be thought to be something of little importance, to which scant attention may be given by more experienced personnel. This

is not so. In most projects the design of the project is based upon an evaluation of ground conditions which is established from the description of samples and natural or excavated outcrops, and thence the testing of chosen materials. Most testing is undertaken on materials which represent the various bodies identified by visual inspection. Thus correct identification is of vital importance and is the firm basis on which all geotechnical models are founded. Many significant engineering mishaps have originated in poor description, and the inaccurate description of ground conditions is a common cause for contract claims.

Identification and description are a basis for classification. Features or properties of the material are identified or measured and then described using standard phraseology. Features may be colour, mineralogy, structure, texture, presence of weathering etc.; properties may be strength, deformability, permeability etc. Classification using these features and properties assigns a name to the material, grades properties and so forth. The aim of this procedure is to give a means of reliable communication between geotechnologists so that no engineering errors occur as a consequence of misunderstanding.

The name to the soil is given by its grain size. Many grain size classifications exist, each with minor variations reflecting problems in the particular country of their origin. Terminology is not uniform throughout the world. A simple grain size classification for soils, following BS EN ISO 14688-1, is given in Table 2.1. A full description may include such physical properties as relative density (for sands) or strength (for clays) and a description of geological structure. Other features such as colour may be added to help distinguish one stratum of material from another. A description of sand might thus be:

*“Dense thinly bedded grey fine SAND,”* while the description of clay might be: *“Stiff thinly laminated brown CLAY”*

**Table 2.1.** Grain size classification for engineering soils

Soil name	Qualifier	Grain size (mm)
Boulders		> 200
Cobbles		63 – 200
Gravel	Coarse	20 – 63
	Medium	6.3 – 20
	Fine	2 – 6.3
Sand	Coarse	0.63 – 2
	Medium	0.2 – 0.63
	Fine	0.063 – 0.02
Silt	Coarse	0.02 – 0.063
	Medium	0.006 – 0.02
	Fine	0.002 – 0.006
Clay		< 0.002

Many soils are mixtures of two or more grain sizes to give descriptions such as ‘silty SAND’, ‘sandy CLAY’ and ‘silty sandy GRAVEL’. Detailed guidance as to the percentage of subordinate grain sizes required to give a qualified remark (such as ‘slightly sandy’, ‘sandy’ or ‘very sandy’) is given in BS EN ISO 14688-1. It is important to include accurate assessments of the proportions of subordinate grain sizes in a description because their presence may significantly modify the behaviour of the main body of material.

Strength is one of the most important parameters for engineering purposes and so scales of strength have been devised. One such scale is given in Table 2.2. It should be stated whether the rock is fresh (un-weathered) and thus at its greatest strength or has been weakened by weathering to, say, a decomposed condition. A typical description for a fragment of rock material might be “*Decomposed thinly bedded red coarse micaceous SANDSTONE, weak*”.

Geotechnology deals mostly with the mechanics of earth materials and masses, and the reader may wonder why it is necessary to give a geological name to a rock material if its mechanical properties are known. Rock names are based on mineralogy, texture, grain size, genesis, geographical location and other parameters which may have no direct bearing on mechanical properties. This is in contrast to soil names which, since they depend on grain size, tell something about their properties such as porosity and permeability. However, rocks must be named *because of the association of the rock type with other phenomena which may be of significance in engineering*.

Table 2.2. Strength scale for clays and rocks

Strength category		Description	Field definition
Guide to shear strength of clays (kPa)	20	Very soft	Extrudes between fingers when squeezed
	40	Soft	Very easily moulded with fingers
	75	Firm	Moderate finger pressure required to mould
	150	Stiff	Moulded only by strong finger pressure
	300	Very stiff	Can be indented by thumbnail
		Hard	Can be scratched by thumbnail
Guide to unconfined compressive strength of rocks (MPa)	1.0	Extremely weak	Crumbles in hand
	5	Very weak	Thin slabs break easily under hand pressure
	25	Weak	Thin slabs break under heavy hand pressure
	50	Medium strong	Lumps or core broken by light hammer blows
	100	Strong	Lumps or core broken by heavy hammer blows
	250	Very strong	Lumps only chip with heavy hammer blows. Dull ringing sound
		Extremely strong	Rocks ring on hammer blows. Sparks fly

For example, *Limestone* if associated with underground caverns, would suggest the possible presence of solution cavities, high permeability, and water resources. *Quartzite* is associated with high strength and abrasiveness. Basalt, if *extrusive* as lava flows, is associated with scoriaceous and rubble deposits between flows, very irregular thickness, interlayers of weathered material, buried soil horizons and poorly defined jointing, except in the centre of thick flows. If *intrusive* as thin dykes or cylindrical plugs, jointing may be well defined. Shale and schist have very closely spaced bedding or foliation discontinuities, giving many opportunities for failure and anisotropic properties.

Exact identification is not the task of the engineer but, because the engineer on site may be the first person to see new excavations, he or she should be able to identify approximately any rock exposed, for this identification might lead to the early recognition of a problem. This identification need not be as accurate as a geologist should give, but should be not too far from the truth. Figure 2.2 gives an 'Aid to the identification of rocks for engineering purposes' which may be used by engineers. Engineering geologists are, however, required to give the correct name to a rock for this might lead to associations with more complex problems than those mentioned.

The distinction between igneous, sedimentary and metamorphic rocks is often best made by a simple visual examination of the outcrop.

### 2.3

#### Properties and their Measurement

The following factors will determine the properties of a dry material:

- the types of minerals to be found in the grains and the physical properties of those minerals,
- the size and shape of the grains,
- the density of the packing of the grains,
- the nature and strength of the bonding between the grains,
- the distribution of these grains within the material, whether random, layered, orientated and so forth.

These factors are clearly of importance and there have been attempts to correlate basic characteristics with material properties. As an illustration of one of the early works that of Koerner (1970) on sands may be cited. Following undertaking many triaxial tests he concluded that:

$$\varphi d = \varphi f + \varphi \delta \quad (2.1)$$

where  $\varphi d$  is the friction angle measured in drained triaxial shear,  $\varphi f$  is the frictional component and  $\varphi \delta$  is a dilational component. A value of  $\varphi f$  was proposed based on the mineral grain properties of the saturated sands tested such that:

$$\varphi f = 36^\circ + \Delta\varphi_1 + \Delta\varphi_2 + \Delta\varphi_3 + \Delta\varphi_4 + \Delta\varphi_5 \quad (2.2)$$

where  $\Delta\varphi_1$  = a particle shape correction;  $\Delta\varphi_2$  = a particle size correction;  $\Delta\varphi_3$  = a correction for grading;  $\Delta\varphi_4$  = a correction for relative density, and  $\Delta\varphi_5$  = a correction for

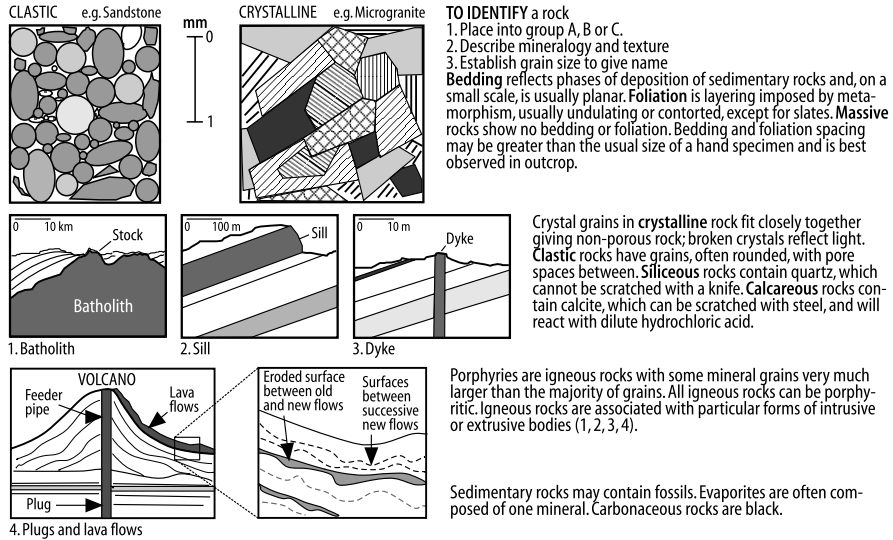
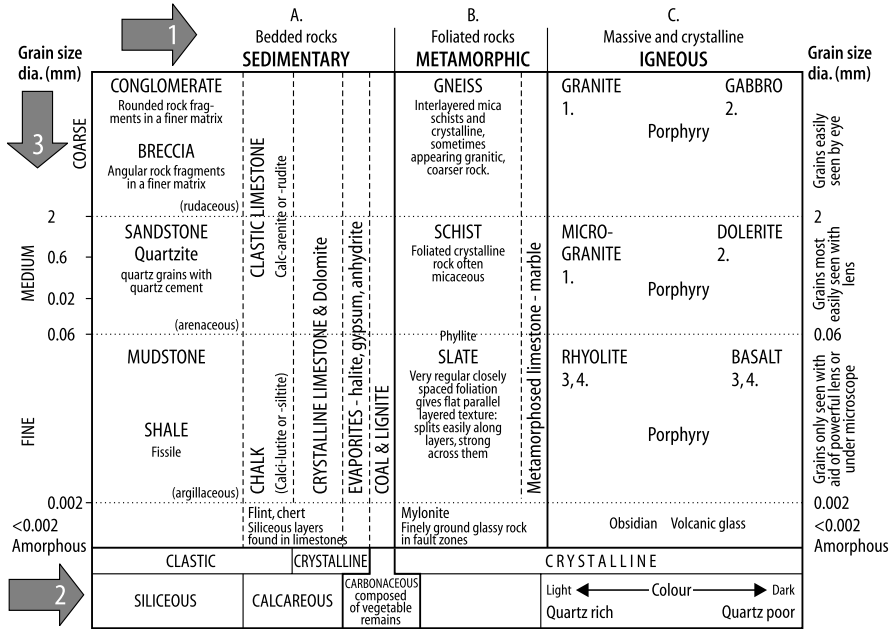


Fig. 2.2. Aid to the identification of rocks for engineering purposes (based on a similar table in BS5930 (1999) and proposals by the author to International Standards Organisation)

mineralogy. Most of the five factors considered to determine material properties are incorporated in Koerner's formula.

Presently, material testing is undertaken in the laboratory or in the field and the parameters measured are generally related to the following basic properties:

- density
- porosity and permeability ‡
- strength ‡
- deformability ‡
- abrasivity ‡
- environmental reactivity

The last property relates to the stability and durability of materials when they are taken out of their 'natural' environment and placed in another. Thus, rock taken from a quarry and used as concrete aggregate may, when placed in the new environment of concrete, chemically react with the cement and change its mineralogical character.

Most geological materials are anisotropic as a result of the way in which they were formed or deposited. Thus most sediments are bedded, metamorphic rocks may have lineations or foliations and igneous rocks may be banded, so that the properties of the materials vary with the internal structure and texture of the material. Results of tests on these materials will vary depending on the direction of testing. In some cases the internal anisotropy may be so slight as to be insignificant and for all practical purposes the material may be considered to be homogeneous and isotropic; much of the background theory of soil mechanics and rock mechanics is based on the assumption that the materials dealt with are isotropic and homogeneous. Anisotropy affects almost every property of a material but those properties most affected are marked '‡' in the list above. In certain clearly anisotropic materials, such as schists, slates and shales, and layered sediments, variations in material properties due to anisotropy may be of vital importance in certain projects.

### 2.3.1

#### Types of Test

There are very many different tests which are used to measure the properties listed above. The properties which are measured may be divided into three groups. These are:

- *Material properties* – which are obtained by such observations or laboratory testing as give a scientifically valid result whatever appropriate test procedure is applied. The instrumentation used to measure these properties will influence the accuracy of the result. Examples of such properties might be mineral content, chemical content and density.
- *Test properties* – which are measured by observing the reaction of the test specimen to a given test procedure. The property measured is recognised as a property in materials science but the result obtained depends very much on test procedure. Examples of such properties are permeability, porosity, strength, modulus of deformation, cohesion and so forth.
- *Empirical properties* – which are the result of performing the test in a standard way using particular apparatus on a particular style of test specimen. These tests are of-



ten associated with parameters which are not recognised in physical science but are useful with regard to a particular engineering process. Examples might be such 'properties' as ASTM Toughness, the Slake Durability Index, the Los Angeles abrasion value, and the Polished Stone Value.

Many tests are difficult and expensive to undertake and in the beginning of an investigation it is sometimes sufficient to have an approximate idea of the value of certain properties. The results of some easily and cheaply executed tests may have a relationship to the results of more complex and expensive tests. If this relationship is known then the cheaper test may be undertaken to obtain an approximate idea of the value of an otherwise more expensively obtained property. Such tests are called *index* tests.

### 2.3.2

#### Limitations of Testing

The reaction of a ground mass to an engineering process depends partly on the engineering behaviour of the materials of which the ground mass is composed. Material behaviour is determined by testing in which the intent is to subject samples of materials, in the condition that they are found underground, to the changes in conditions which will arise as a consequence of engineering construction. This intent is seldom realised firstly because it is almost impossible to extract a sample from the ground in a truly undisturbed condition and secondly because, for practical reasons, testing must be undertaken in a much shorter time than that in which the changes consequent to engineering construction take place. Clearly the condition of the sample tested is also of significance to the accuracy of the test result, which would be influenced by any flaws developed as a result of sample extraction and preparation. In particular, differences in moisture content between the material *in situ* and as a laboratory sample have major influence on test results. As well as these factors the way in which the tests are performed is of importance.

### 2.3.3

#### Size and Shape of Sample

In the simple uniaxial compressive strength test, the test specimen is usually a solid cylinder. The distribution of stress within the specimen is such that failure under uniform load equal to that exerted by the test machine is not expected for samples whose length to diameter ratio is less than 2:1. In soil mechanics this ratio is generally accepted and also mostly in rock mechanics. However, in rock mechanics there is no generally accepted standard for testing, although some recommendations (such as the "*Suggested Methods*" of the International Society for Rock Mechanics, Brown 1981) are widely adopted.

Special shapes of sample have been devised to give more reliable results for certain tests. The '*dog bone*' sample in Fig. 2.5 is intended to give tensile and compressive strength values free of the complications caused by stress distribution in the sample, but the cost of preparation of such shapes is usually prohibitive.

It is obvious also that the size of the sample must be considerably greater than the grain size of the material. Thus, for example, a 50 mm diameter, 100 mm long speci-

men of basalt would be acceptable for a strength test, but this size of sample would be inappropriate for a coarse conglomerate in which the size of an individual 'grain' might be a significant proportion of the size of the specimen. Test specimens are mostly specimens without fractures and are thence described as specimens of intact rock.

#### 2.3.4

##### Rate of Loading, Testing Machine and Platens

The rate of application of load on the test specimen has a significant effect on the value of failure strength measured (Price and Knill 1966). Almost all rates of loading applied in the laboratory are far higher than the rates likely to be applied in engineering construction, so that to some extent, all values are false. Usually, the rate which is adopted is the slowest which can be used in the circumstances. Sometimes testing standards or recommendations suggest rates to be used.

Testing machines may be regarded as either "*soft*", i.e. "*flexible*" or "*stiff*". The test frames of the former deforming significantly under load so that, at the first crack in the test specimen the release of strain energy from the machine causes explosive failure of the specimen. Stiff machines were built so massively that there was little deformation of the frame. Some differences in strength results have been observed between the different machine types, but they are not very significant unless the tests are high load tests in rock mechanics or, for the particular project, post failure behaviour must be examined. Nowadays the 'stiffness' is created by servo-controlled systems that allow the deformation of the sample to control the rate at which it is loaded, so avoiding explosive failures.

In rock mechanics, platen conditions are important. Results may be affected by differences in metal or other material used for these plates. A more basic problem arises from the constraints on sample diametral expansion imposed by friction developing between platen and sample when their stiffness is not the same. This has been partially solved by the design of platens incorporating devices to allow the sample to dilate radially but such devices are too complex for standard use.

#### 2.3.5

##### Standards

Results of tests on materials will vary depending upon the test procedure applied and this variation may be of engineering significance. To be assured that results of tests from different laboratories are comparable tests are undertaken following established *standards* or *norms* set up by national or international bodies. Examples of such organisations are the British Standards Institution (BSI), the American Society of Testing and Materials (ASTM), the International Standards Organisation (ISO), the International Society of Rock Mechanics (ISRM), the Deutsche Industrie-Norm (DIN) and the Nederlands Normalisatie Instituut (NNI). Not all tests are standardised and standards for particular tests vary, usually but slightly, from country to country. All routine tests should be performed to a standard and that standard recorded in reports and scientific papers.

## 2.4 Density and Unit Weight

Most rocks and all soils can be regarded as containing voids between grains. These voids may be filled with water, air or both. Thus, three conditions are possible – dry, partially saturated and saturated. The in situ or ‘bulk’ density of the material is determined by the relative proportions of mineral particles, water, and air in a given volume of material, and the specific gravity of the mineral particles.

Density ( $\rho$ ) is the amount of mass in a given volume and the units are  $\text{Mg m}^{-3}$ . Unit weight ( $\gamma$ ) is the weight of a unit volume, and the units are  $\text{N m}^{-3}$ . Usually, because the Newton is a small force, unit weight is expressed in  $\text{kN m}^{-3}$ . Formulae appropriate to the calculation of various parameters associated with density and unit weight are given in Table 2.3.

**Table 2.3.** Formulae in common use for parameters related to density and unit weight

Density ( $\rho$ )	$\rho = \frac{\text{Mass (M)}}{\text{Volume (V)}}$	(2.1)
Unit weight ( $\gamma$ )	$\gamma = \frac{\text{Weight (W)}}{\text{Volume (V)}}$	(2.2)
Particle specific gravity ( $G$ )	$G = \frac{W_s}{V_s \gamma_w}$	(2.3)
Void ratio ( $e$ )	$e = \frac{\text{Volume of voids}}{\text{Volume of solids}} = \frac{V_v}{V_s}$	(2.4)
Moisture content ( $m$ )	$m = \frac{\text{Weight of water}}{\text{Weight of solids}} \cdot 100\%$	(2.5)
Note: Because of the way in which moisture content is defined, it is possible to have materials with $m > 100\%$ .		
Degree of saturation ( $s$ )	$s = \frac{\text{Volume of water}}{\text{Volume of voids}} = \frac{V_w}{V_v} \cdot 100\%$	(2.6)
Bulk unit weight ( $\gamma_b$ )	$\gamma_b = \frac{\text{Total weight}}{\text{Total volume}} = \frac{W_s + W_w}{V_s + V_v}$	(2.7)
Saturated unit weight ( $\gamma_{\text{sat}}$ )	$\gamma_{\text{sat}} = \frac{\text{Saturated weight}}{\text{Total volume}}$	(2.8)
Dry unit weight ( $\gamma_d$ )	$\gamma_d = \frac{\text{Dry weight}}{\text{Total volume}}$	(2.9)

$V_s$  = volume of solids;  $V_v$  = volume of voids;  $V_w$  = volume of water;  $W_s$  = weight of solids;  $W_w$  = weight of water;  $\gamma_w$  = unit weight of water.

Most granular soils have a wide range of unit weight within which the soil may occur in situ. Figure 2.3 shows a 'theoretical' soil composed of perfectly spherical uniform diameter grains at minimum bulk density with, for this soil, the largest possible spaces between the particles, so that the void ratio is at maximum while the bulk density is minimal. If the soil is compacted to maximum bulk density by vibration and/or weight of material above, the particles will close together, giving minimum size pore spaces and void ratio. The in situ density lies usually somewhere between maximum and minimum; the relative density expresses where the in situ density lies between minimum and maximum values for a particular soil and is a measure of the state of compaction which the soil has reached.

Relative Density (*R.D.*) may be therefore expressed in terms of void ratio (*e*) or unit weight ( $\gamma$ ):

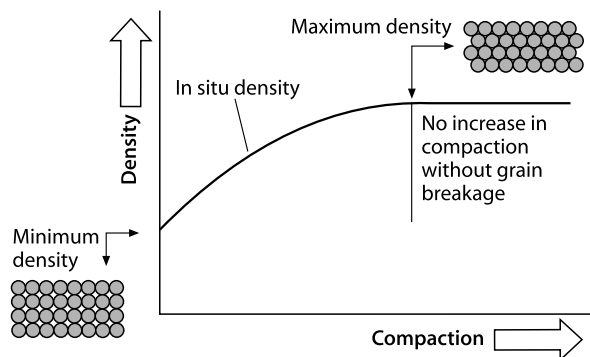
$$R.D. = \frac{e_{\max} - e}{e_{\max} - e_{\min}} = \frac{\gamma_{\max}}{\gamma_{\text{in situ}}} \left[ \frac{\gamma_{\text{in situ}} - \gamma_{\min}}{\gamma_{\max} - \gamma_{\min}} \right] 100\% \quad (2.3)$$

where, for a particular sample,  $\gamma_{\min}$  = minimum possible unit weight,  $\gamma_{\max}$  = maximum possible unit weight and  $\gamma_{\text{in situ}}$  = in situ unit weight.

Relative density assumes that the maximum density is achieved without alteration to the size and shape of the grains, i.e. the grains are not fractured and broken into smaller pieces by compaction and that the grains are solid, i.e. they do not contain pores as do the grains of pumice and chalk. The degree of compaction achieved by a natural deposit may be classified by relation to the relative density. Terminology is given in Table 2.4.

Since relative density suggests the degree of compaction a granular soil has achieved relative to the maximum it might achieve it indicates the settlement that can take place under extra load, such as that of a foundation. In Fig. 2.3 there are, in two dimensions, four point contacts between the grains at minimum density and six contacts at maximum density, indicating an increase in shear resistance of granular materials at higher degrees of compaction (Table 2.4). Because relative density relates to both settlement

Fig. 2.3. In situ density related to maximum and minimum densities



**Table 2.4.** Terminology for relative density and approximate relationships to other properties of common granular soils such as sand, silt and gravel

Relative density (%)	Dry unit weight ( $\text{kN m}^{-3}$ )	Descriptive term	$\phi$ (°)	SPT 'N' value
0 – 15	< 14	Very loose	< 28	0 – 4
15 – 35	14 – 16	Loose	28 – 30	4 – 10
35 – 65		Medium dense	30 – 36	10 – 30
65 – 85	16 – 17.5	Dense	36 – 41	30 – 50
85 – 100		Very dense	> 41	> 50

on and strength of foundation materials it is of importance in foundation design. Generally, relative density is measured using the Standard Penetration Test (SPT) in boreholes and the SPT- $N'$ -value is much used in foundation design. In trial pits it may be assessed approximately by the resistance of the soil to excavation. It is also an important parameter in the assessment of the potential for liquefaction of sands during earthquakes.

## 2.5 Porosity and Permeability

Porosity is a measure of the voids in sediment and rock, and the masses they form. Permeability is a measure of the extent to which these voids are connected.

### 2.5.1 Porosity

Porosity is a measure of the volume of voids in a material or mass. In materials porosity depends upon the space between grains; in masses it would also include any space provided by open fissures and joints.

$$\text{Porosity}(n) = \frac{\text{Volume of voids}}{\text{Total volume}} \quad (2.4)$$

Intergranular porosity is often determined in the laboratory by comparison between dry and saturated weights of the sample. To become fully saturated, voids must be in contact, and both interconnections and voids must be large enough to allow the flow of water under reasonable pressures. One or both of these conditions is often not satisfied, particularly for fine grained materials, and the results of such tests are properly described as apparent porosity. Note that the execution of the porosity test may damage the sample. There are some porous rocks, such as vesicular basalts, that are porous but whose porosity cannot be measured by conventional means because the voids are not in contact i.e. the material is impermeable.

### 2.5.2

#### Permeability

Permeability is a general term which describes the ability of a porous medium to allow the flow of fluid through it. For flow to take place through a saturated material there must be a difference in total head. In engineering geology the usual concern is with the flow of water through a granular soil or rock mass. The head of water  $h$  is the height of the water level above the point at which the head is measured. Thus, the pressure of water at that point  $p = h\gamma_w$ , where  $\gamma_w$  = unit weight of water.

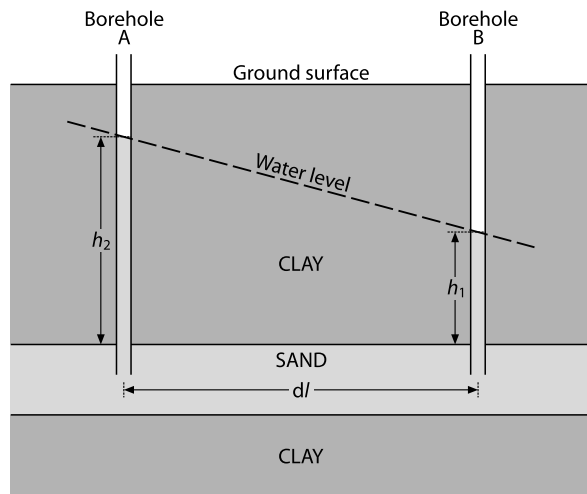
In 1856, Darcy conducted laboratory experiments to analyse the flow of water through sand, and showed that, if  $Q$  is the quantity of water passing through a sand tube of cross-sectional area  $A$  in time  $t$ , then the specific discharge  $q$  of water flow is given by:

$$q = \frac{Q}{A} \quad (2.5)$$

In this experiment the flow was intergranular and non-turbulent; these are conditions most likely to be encountered in engineering geology except where flow is fast as in gravel, open rough fractures in rock or through some clays where the clay minerals may interfere with free flow.

Darcy also showed that the specific discharge was directly proportional to the hydraulic gradient ( $i$ ), which is the loss of total head per unit length of flow (Fig. 2.4). Thus  $q \propto i$  and  $q = Ki$ , where the constant of proportionality  $K$  (the *coefficient of permeability*) is dependent upon both the properties of the porous medium and the fluid. Since the fluid is water,  $K$  is also called the *hydraulic conductivity*. The properties of the fluid which effect  $K$  are fluid density and viscosity, which change with temperature.  $K$  is therefore temperature dependent. Thus

**Fig. 2.4.** The hydraulic gradient between A and B is  $(h_2 - h_1) / dl = dh / dl = i$



$$Q = AKi$$

(2.6)

and

$$v = \frac{Q}{A} = Ki$$

(2.7)

where  $v$  = a theoretical velocity of the flow across the area  $A$ . The coefficient  $K$  has units of velocity.

In soil mechanics and rock mechanics the usual unit for  $K$  is  $\text{m s}^{-1}$ , but in other applications common units are  $\text{mm s}^{-1}$ ,  $\text{cm s}^{-1}$ ,  $\text{m year}^{-1}$ ,  $\text{ft day}^{-1}$ . The constant of proportionality,  $K$  is determined by the properties of both fluid and medium. Guide values of hydraulic conductivity for rocks and soils are given in Table 2.5. For the medium, the properties which determine the rate of flow are mean grain diameter ( $d$ ), distri-

**Table 2.5.** Guideline values for hydraulic conductivity (permeability) of rock and soil

	Type	Hydraulic conductivity $k$ ( $\text{m s}^{-1}$ )
Rock materials	Granite	$10^{-13} - 10^{-12}$
	Slate	$10^{-13} - 10^{-12}$
	Limestone	$10^{-12} - 10^{-9}$
	Dolomite	$10^{-11} - 10^{-10}$
	Mudstone	$10^{-9} - 10^{-10}$
	Schists	$10^{-6}$
	Sandstone	$10^{-5} - 10^{-9}$
Rock masses	Gneiss	$10^{-5}$
	Granite	$10^{-5}$
	Lignite	$10^{-4}$
	Sandstone	$10^{-4}$
	Mudstone	$10^{-6}$
	Limestone	$10^{-4} - 10^{-6}$
Soil	Gravel	$1 - 10^{-3}$
	Sand	$10^{-2} - 10^{-5}$
	Silty sand	$10^{-5} - 10^{-7}$
	Silt	$10^{-7} - 10^{-9}$
	Clay	$< 10^{-9}$

Note: These values are taken from the literature. They serve as a guide to relative permeabilities.

bution of grain sizes, nature of grain packing and degree of interconnection of the pore spaces. For a particular medium, these properties will not change and therefore will be the same regardless of which fluid is flowing. This is a property which is exclusively that of the medium, and is termed the *specific* or *intrinsic permeability*. The constant of proportionality  $K$  can now be re-defined, thus:

$$K = \frac{Cd^2 \rho g}{\mu} \quad (2.8)$$

where density  $\rho$  and viscosity  $\mu$  are properties of the fluid, and  $Cd^2$  is a function of the medium alone.  $C$  is another constant of proportionality and is a function of the medium properties other than mean grain diameter which determines  $d^2$ . Consequently, the intrinsic permeability  $= Cd^2$ . Confusion arises, because the designated symbol for intrinsic permeability is  $k$ , which is the symbol used in soil and rock mechanics for coefficient of permeability, which more properly should be called hydraulic conductivity.

Intrinsic permeability is used mostly in the petroleum industry, where the fluid under consideration may be petroleum, water or even methane or a mixture of all. The units of intrinsic permeability are  $\text{m}^2$ , but since values in this unit are very small, petroleum engineers have adopted the darcy as the unit ( $1 \text{ darcy} = 0.987 \mu\text{m}^2$  and  $1 \mu\text{m}^2 = 10^{-12} \text{ m}^2$ ).

## 2.6 Strength

All geological materials have some ability to resist failure under the action of stresses; this is their strength. Most values quoted as the 'strength of' a certain material are the stresses at failure, the ultimate failure strength. Usually, testing is done on small samples in the laboratory. Strengths measured are:

- uniaxial (or unconfined) compressive strength, which is the stress at failure of a sample under compression;
- uniaxial (or unconfined) tensile strength which is the stress at failure of a sample under tension;
- triaxial strength, which is the stress at failure of a sample that is confined. This is usually accomplished by placing the sample under compression while it is restrained laterally by a minor horizontal stress.

The units of strength are force/area, for example  $\text{kgf cm}^{-2}$ ,  $\text{N mm}^{-2}$ ,  $\text{kN m}^{-2}$ ,  $\text{MN m}^{-2}$ . Recently it has become fashionable to use pascals, particularly for compressive strength ( $1 \text{ pascal (Pa)} = 1 \text{ N m}^{-2}$ ).

### 2.6.1 Water Content and Drainage

The effects of water content and drainage conditions on test results are best understood in relation to intergranular stresses (effective stresses). At a point at depth  $d$  below the surface which is overlain by saturated material of unit weight  $\gamma_{\text{sat}}$  the total stress



at  $d = d\gamma_{\text{sat}}$ . The pressure of water in the pores between grains of material is  $d\gamma_{\text{water}}$ . Total pressures are reduced by water pressures to give intergranular pressures, so that intergranular pressure =  $d\gamma_{\text{sat}} - d\gamma_{\text{water}}$ .

When a saturated sample is loaded in a test so that no water can drain from the sample (an undrained test), the test load is taken partly by the intergranular contacts and partly by the pore water. The pore water is virtually incompressible; increasing strain on and shortening of the sample cannot now produce reduction of pore size and greater compaction of the sample, but can only cause pore water to push between grains, lower intergranular friction and cause failure at lower stress levels than would otherwise be the case.

All undrained tests on fully saturated soils whose particles are not bound together, give values of friction equal to zero. The fact that some undrained triaxial tests on these materials give values of  $\phi$  above zero is because they are not tested in a truly fully saturated condition and some water movement into unsaturated pore spaces is possible. For the same reasons, saturated rock gives lower strength values than dry rock.

### 2.6.2

#### Strength Tests

Test arrangements and formulae to calculate strength are given in Fig. 2.5.

#### *Compressive Strength*

This is the stress (load/area) at which a sample of material (cylinders in the case of soil or rock, cubes in the case of concrete) fails under compressive stress.

#### *Tensile Strength*

This is the stress at which a sample of material fails under tensile stress. Tensile strength may be measured directly, but in practical terms it is difficult to do. In rocks it may be measured indirectly by the *Brazilian test* or the pierced disc test. A further variation of these tests is the *Point Load test*, which is now used as a field method for the determination of rock strength.

The Brazilian test attempts to measure tensile strength by developing tension across the diameter of a rock disc which is being subjected to compression through a vertical load. Because the tensile strength of rock is between one quarter and one tenth of the compressive strength, the tensile stresses being developed horizontally as a result of vertical compressive stress should be enough to cause tensile failure before failure in compression can occur. In fact, intense contact compressive stresses between sample and platen may cause local fracturing which then proceeds from the circumference towards the tensioned centre. The values of tensile strength measured are thus incorrect. A modified Brazilian test using curved platens to overcome stress concentrations has been tried (Mellor and Hawkes 1971).

#### *Shear Strength*

Material loaded under both major and minor stresses fails in shear. The shear strength of a material under shear load depends upon confining stress and is not a unique value.

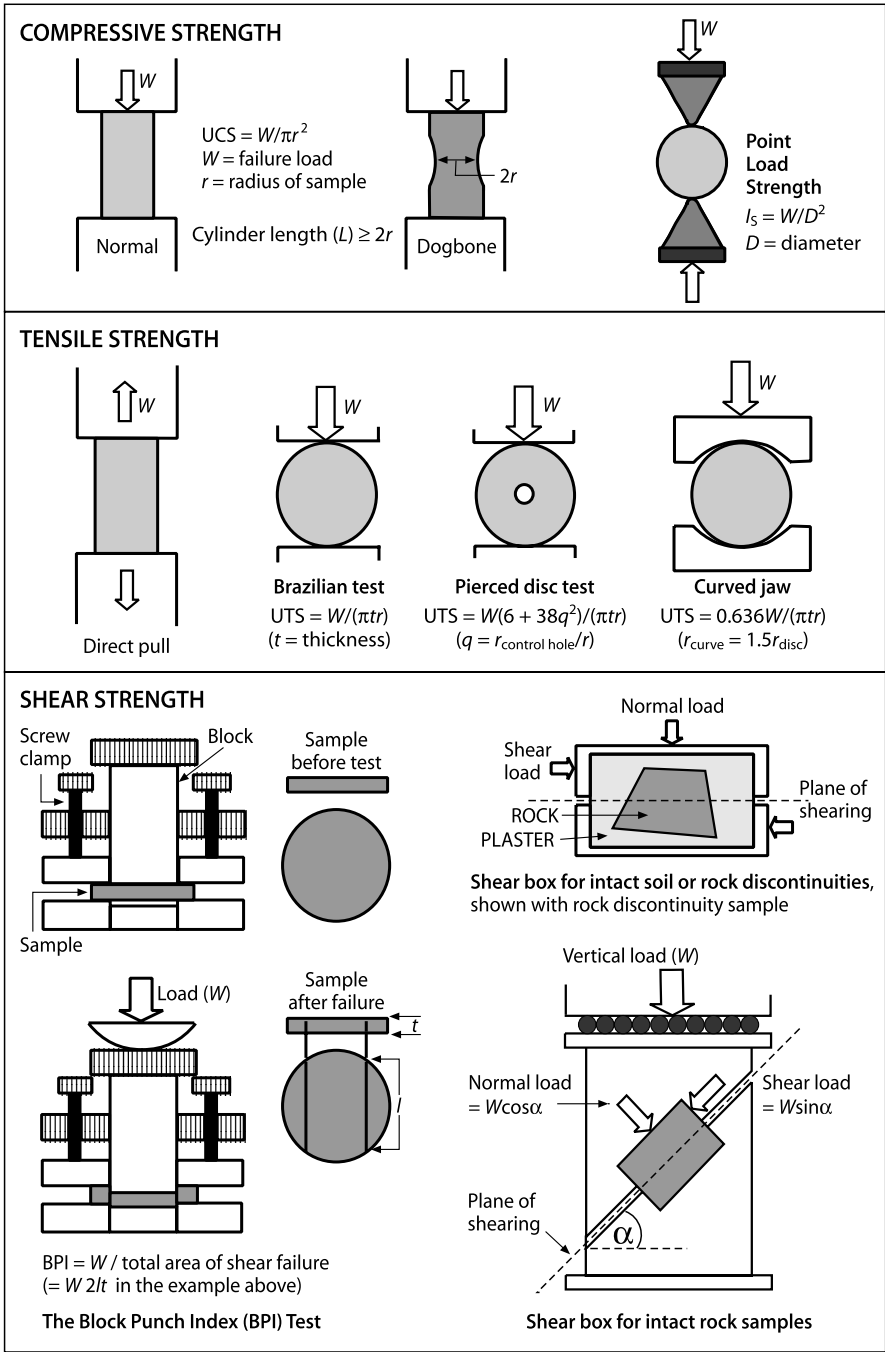


Fig. 2.5. Simple laboratory tests on rock samples. UCS and UTS = unconfined compressive strength and unconfined tensile strength

Unconfined, or uniaxial, strengths may be considered as shear strengths under no confining stress, a situation which rarely occurs in nature. Shear strength is described in terms of parameters  $c$  (cohesion) and  $\phi$  (angle of shearing resistance).

Consider a block of weight  $W$  resting on a horizontal plane (Fig. 2.6). The weight  $W$  produces an equal and opposite reaction  $R$ , without any tendency for the block to move. If a horizontal force  $S$  is applied and increased until the block is just about to slide, the reaction, or resultant force  $R$ , will be inclined at an angle  $\alpha$  to the vertical, so the horizontal component of  $R$  is  $S = R \sin \alpha$  and the vertical component of  $R$  is  $W = R \cos \alpha$ . The angle  $\alpha$  will have increased to a limiting value  $\phi$ . At this point the horizontal force resisting sliding,  $S = W \tan \phi$ . The parameter  $\tan \phi$  is known as the coefficient of friction.

Shear strength can be seen to be related to frictional resistance within a material. However, materials whose grains are bonded together may have additional strength produced by the adhesion or cementing between grains. Coulomb's law states that:

$$\sigma_\tau = c + \sigma_n (\tan \phi) \quad (2.9)$$

where:  $\sigma_\tau$  = shear stress at failure (shear strength) =  $S / A$ ;  $c$  = cohesion (adhesion or cementing);  $\sigma_n$  = normal stress on sliding plane =  $W / A$ , and  $\phi$  = angle of frictional resistance.

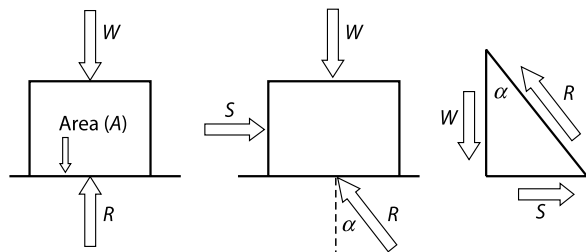
According to these properties, three types of geological material may exist. These are:

- $c = 0$  (materials exhibiting no cohesion, such as dry sand)
- $c$  and  $\phi$  materials (soils and rocks with both cohesion and internal friction)
- $\phi = 0$  (materials exhibiting no internal friction)

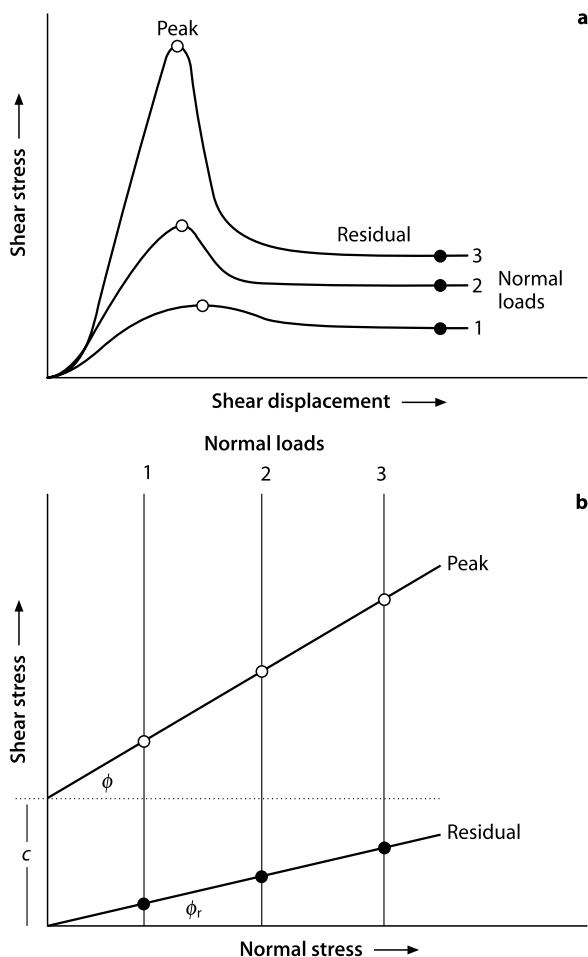
In the *shear box* test (Fig. 2.5), shear force and normal force are applied directly, and it is only necessary to plot graphs of shear stress against normal stress, and shear strength (Fig. 2.7) against shear displacement to find  $c$  and  $\phi$  and then peak and residual shear strengths. Peak strength may be defined as the maximum resistance of the sample to shear force while the residual strength is the resistance of the failed shear surface after considerable movement.

Shear boxes may be used to determine  $c$  and  $\phi$  on intact material samples (Fig. 2.5), or along existing discontinuities which must be aligned along the plane of shearing in the box.

Fig. 2.6. The concept of shear strength



**Fig. 2.7. a** Stress/displacement graphs; **b** Graphical calculation of peak and residual strength parameters



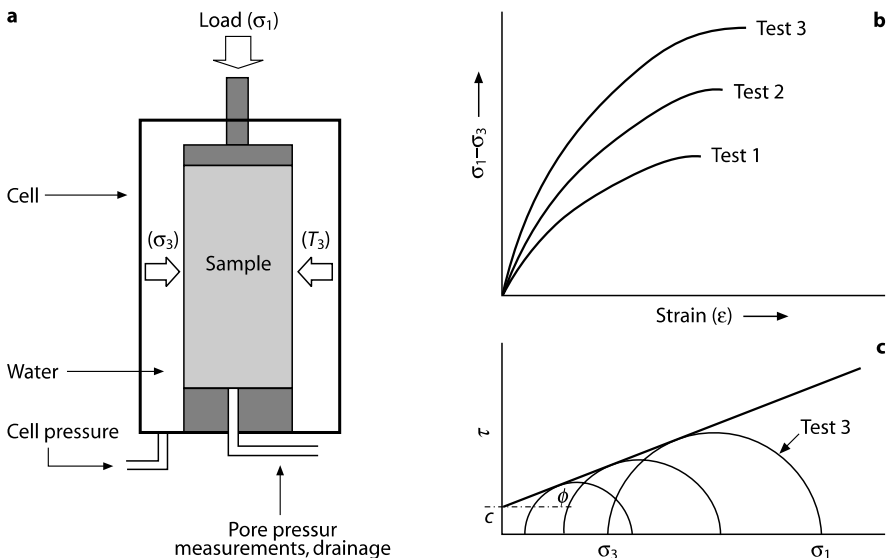
Shear strength parameters of intact rock are difficult to measure, because loads and strengths are generally high, at least compared to soils, but it can be done (Fig. 2.5). Most shear testing on rock is done on discontinuities such as joints, which requires the sample to be cast into a form which fits the box. To determine the shear strength at failure, shear displacement must be plotted against shear stress. A peak failure load is normally clearly evident (Fig. 2.7a). After failure shear stress can fall to an approximately constant value. This residual strength value may be plotted on the shear strength/normal stress graph to find residual values of  $c$  and  $\phi_r$ , which are designated to  $c_r$  and  $\phi_r$ , and may be determined for intact material or for existing discontinuities (Fig. 2.7b). The ring shear apparatus may be used for determination of residual values for soils. The shear displacement is circular and limited only by the length of time over which the test is conducted.

### Triaxial Strength

To try to determine the effect that stresses caused by surface construction would have on underlying soils or rocks it would seem sensible to take samples from various depths below the construction, subject them to the appropriate confining overburden pressure and thence impose the additional vertical construction stresses appropriate to the sample depths. The *triaxial cell* has been designed to do this.

In the *triaxial test* (Fig. 2.8) the cylindrical sample is placed in a test cell which is constructed so as to allow the application of an all round hydraulic pressure on the sample and also the application of a load which will bring the sample to failure. The test sample is surrounded by an impervious membrane to prevent contact with the cell fluid. The hydraulic pressure is kept constant for each test and the deviator stress ( $\sigma_1 - \sigma_3$ ) increased until failure occurs. The Mohr circle diagram is used to plot the shear and normal stresses.

There are very many possible variations of the triaxial test. A truly accurate test would impose the construction load at the rate with which construction takes place and allow for drainage of the sample. Such testing is expensive, so a variety of tests are possible approaching reality with an increasing degree of complexity and expense. In the simplest, the *quick undrained test*, samples are tested without drainage, as this gives a measure of their strength when weakest, and without the measurement of their pore water pressures (which takes time). The triaxial test is not confined to soils and increasing strength of the apparatus allows stronger materials to be tested (Hoek and Franklin 1968).



**Fig. 2.8.** **a** the triaxial test apparatus; **b** plots of deviator stress vs. strain for three cell pressures; and **c** Mohr circle plots to calculate  $c$  and  $\phi$

## 2.7 Deformation

If a sample of material is placed under load, it deforms; that is, it changes in shape and perhaps also in volume. If, when the load is removed, the deformation instantly and completely disappears, the material may be said to be elastic. In this case the relationship between stress and strain is often linear. If a cylinder of length  $l$  is loaded by a stress  $\sigma$  applied to the top of the cylinder, the length may be shortened by an amount  $dl$ . The strain  $\varepsilon$  is expressed by:

$$\varepsilon = \frac{dl}{l} = \frac{\text{Length decrease}}{\text{Original length}} \quad (2.10)$$

For elastic materials, strain is related to stress by Hooke's law, so that:

$$E = \frac{\sigma}{\varepsilon} \quad (2.11)$$

$E$  is commonly referred to as Young's Modulus.

If an elastic cube is deformed by shear stress, the cube changes its shape to become a rhomb. The small angle  $d\theta$  which lies between the sides of the cube and that of the rhomb is a measure of the shear strain. The stress/strain relationship is given by the Modulus of Rigidity ( $G$ ).

$$G = \frac{\text{Change in shear stress}}{\text{Change in shear strain}} = \frac{\tau}{d\theta} \quad (2.12)$$

If the elastic solid is changed in volume due to a change in hydrostatic pressure, then the stress/strain relationship is given by the Bulk Modulus ( $K$ ).

$$K = \frac{\text{Change in hydrostatic pressure}}{\text{Change in volumetric strain}} = \frac{d\sigma}{d\varepsilon_{\text{vol}}} \quad (2.13)$$

If a cylinder of Young's Modulus  $E$  is shortened by axial load, it will bulge outwards. If the shortening of length is  $dl$ , and the increase in diameter is  $dd$ , then the ratio of these strains is given by Poisson's ratio  $\nu$ :

$$\frac{dd}{dl} = \nu \quad (2.14)$$

### 2.7.1

#### Types of Rock Deformation

Few, if any, materials behave in a truly elastic manner, and various styles of deformation may be observed from the stress/strain curve plotted during a load test. If the plot is more or less linear, then the material may be described as elastic. If not, the material may be described as 'plastic' or some mixture of elastic-plastic behaviour (Fig. 2.9). It is then prudent to speak of the *Modulus of Deformation* rather than a Young's modulus. The moduli chosen for use in deformation calculations depend mostly on the levels of stress expected in the engineering situation (Fig. 2.9). See also Table 2.6.

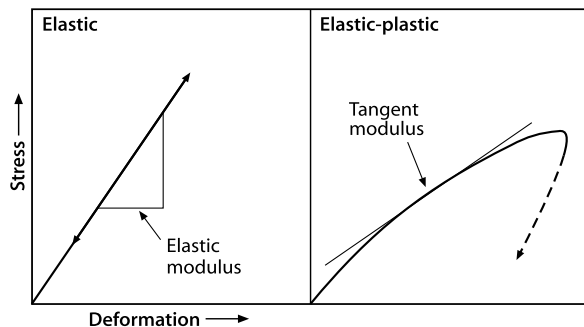
**Creep.** If a specimen is loaded to failure in a testing machine, the test is often completed in a very short time, and the strength value measured may be described as the *instantaneous* or short-term strength. If the same specimen were instead to be subjected to a lower stress for a long period of time, it may be found that the rock fails by a process known as *creep*, at a stress sometimes very much lower than that which gave the instantaneous strength. Values of *long-term strength* may be as low as 25% of instantaneous strength. In general terms the greater the initial stress the shorter the time to creep failure. Recognition of the effects of creep on rock strength is important in the design of mine pillars, particularly if the magnitude of constant overburden load stress is a high proportion of the instantaneous strength. In civil engineering, the loads applied to rock materials are usually but a small fraction of their instantaneous failure strength so unless the rock is very weak and/or the loads are unusually high, there is little need to be concerned about the effects of creep.

### 2.7.2

#### Consolidation of Soils

Application of load to soils produces 'consolidation'. In the case of clayey soils, the load is carried initially by both the solid particles of the soil and the water within the pore spaces. In time, with continual application of the load, the water will slowly be driven out of the material. This is a slow process because of the low hydraulic conductivity (permeability) of the clay, but as the water is driven out, there will be an increase in

Fig. 2.9. Types of rock deformation under stress



**Table 2.6.** Illustrative values of elastic moduli for some fresh rocks

Rock type	Unconfined compressive strength (dry) (MPa)	Elastic (tangent at 50% UCS value) modulus (dry) (GPa)
Augite olivine basalt <sup>a</sup>	398	75.6
Dolerite <sup>a</sup>	261	52.7
Dunite <sup>a</sup>	133	97.6
Granite <sup>a</sup>	201	66.3
Quartzite <sup>a</sup>	229	52.1
Granite biotite gneiss <sup>a</sup>	308	71.7
Mica gneiss <sup>a</sup>	154	41.3
Crystalline limestone <sup>b</sup>	190	70
Calcarene <sup>b</sup>	3.5	1.2
Chalk <sup>b</sup>	22	11

<sup>a</sup> From Verhoef et al. 1984; <sup>b</sup> from Swart 1987.

effective stress as the load is carried more and more by the soil particles. The result will be a decrease in volume of the soil i.e. consolidation, the magnitude of which is related to the load. Engineers are concerned to know by how much a clay will consolidate, and in what time, so that the amount of settlement of a structure supported by the soil can be determined. The amount of consolidation may be expressed by the coefficient of volume change,  $m_v$ :

$$m_v = \frac{\text{Volumetric change in vertical direction}}{\text{Unit of pressure increase in vertical direction}} \quad (2.15)$$

The higher the value of  $m_v$  the more the soil will consolidate (Table 2.7).

Consolidation is also a natural process. In an uninterrupted sequence of deposition any soil will consolidate under the load of a younger bed laid down on top of it. Such soils are said to be *normally consolidated*. These are mostly younger soils. Beds of older clays may, after their deposition, have been subject to erosion so that some of the upper beds have been removed. The beds remaining then display consolidation characteristics that are not appropriate to the overburden that presently covers them. They are then said to be *over-consolidated*. Over-consolidation can also result from the weight of the ice sheets that covered glacial clays during the Pleistocene and melted at the end of this period. The differences in consolidation characteristics depend upon the geological history of a deposit. Thus settlement of structures may be expected to be much greater on younger normally consolidated soils than on the older over- or geologically consolidated soils. It should be noted that consolidation, as described in soil mechanics, is but the early stage of diagenesis.

To predict the effect that settlement will have on an engineering structure it is also necessary to know how quickly this will take place so that the structure may be de-



**Table 2.7.** Typical values of  $m_v$  (from Smith 1968)

Soil	$m_v$ ( $\text{m}^2 \text{MN}^{-1}$ )
Peat	10 – 2
Normally consolidated clay	2 – 0.25
Stiff clay	0.25 – 0.125
Hard clay (boulder clay)	0.125 – 0.0625

signed to accommodate these movements. Tests are thus conducted to determine both the coefficient of volume change,  $m_v$ , to deduce the magnitude of settlement, and the coefficient of consolidation,  $C_v$  (expressed in units of area/time) to deduce the rates of settlement.

## 2.8 Abrasiveness

Quartz is a common rock forming mineral and, because of its resistance to weathering, one of the commonest components in soils. It is also one of the hardest minerals, being number 7 on Moh's scale of mineral hardness, implying that it will scratch most other common minerals. It will also scratch steel. There are a number of engineering processes which involve cutting rock using hard picks, teeth, bits etc. The economy of the operation may depend upon the amount of wear on the cutting tools, which have to be replaced when they become blunt. This wear will depend upon the quantity, grain size and shape of the quartz in the rock or soil, how firmly the quartz is cemented into the matrix (expressed by the rock or soil strength), the properties of the cutting device and the mechanics of cutting.

A number of tests have been developed to assess abrasiveness of a rock. Most involve pressing a steel pin against the rock with uniform force, scratching the rock for a specified distance and observing the loss of steel from the pin. The Cerchar test involves moving a specially shaped pin a short distance over a prepared rock sample and thence measuring weight loss. In the pin-on-disc test a specially prepared pin is pressed against a rotating disc of specimen rock for a given number of revolutions. In both of these empirical tests the quality of the steel making up the pin is specified. Relative abrasiveness may be assessed from knowledge of rock mineral content and strength. Schimazek and Knatz (1970), using a pin-on-disc test, developed the ' $F$ ' value (a wear factor), using the formula:

$$F = \frac{Qtz.Eq. \times d \times BTS}{100} \text{ (N mm}^{-1}\text{)} \quad (2.16)$$

where  $Qtz.Eq.$  = quartz equivalent or content (% by volume),  $d$  = mean diameter of quartz grains (mm), and  $BTS$  = Brazilian tensile strength (MPa).

The higher the value of  $F$  the greater the weight loss of the pin and, it may be presumed, the greater the wear on cutting tools. The reader is warned that the subject of

abrasion is one of great complexity relating to the science of tribology, which deals with friction, wear and lubrication and is thus clearly of importance in mechanical engineering. Verhoef (1997) has reviewed the subject of wear on cutting tools and the form of site investigations necessary to gather appropriate data to attack these problems while Deketh (1995) has reviewed abrasion tests.

## 2.9

### Environmental Reactivity

Rocks and soils underground exist within their environment, which may be defined in terms of stress, temperature, moisture content etc. When ground materials are exposed the environment will change and the ground materials may themselves change in response. Stress release may induce rebound and cracking but also the minerals forming the materials may change. Some rocks, for example shales and mudstones, may on exposure rapidly weather and decay and begin to revert to their original muds.

Slaking tests, in which small samples of rock are subjected to repeated wetting and drying, may demonstrate susceptibility to weathering if the sample breaks up into smaller pieces after a number of wet/dry cycles. The slake durability test (Franklin and Chandra 1972) subjects samples of rock to both wetting, drying and some degree of abrasion.

Many minerals contained in materials become unsuitable if subjected to changes in their environment as the consequence of engineering. Thus, for example, pyrite and marcasite will suffer expansion as the result of oxidation on exposure to air, moisture and perhaps bacterial action and higher temperatures. If these minerals are present in rock (and they are quite common in some Carboniferous shales and mudstones) they will expand when exposed. This could result in upward heave of foundations built on these rocks, changes in the stability of tunnel walls etc. Other minerals such as chalcedony and chert will react to the cement in concrete if they are present in rocks used for concrete aggregate.

## 2.10

### Index Tests

Certain tests require little in the way of sample preparation and are so simple to conduct that the characteristics of soils and rocks, regardless of how complex these are, have been linked to them. Thus a simple test can indicate a range of material characters and the behaviour to expect from them, such as strength, permeability, and likely change in the presence of water. These tests are called *Index tests*.

### 2.10.1

#### Soils

##### *Grading*

Granular materials are described with reference to their particle size and grading curves. Grading curves are obtained by passing the material through a series of sieves down to fine sand size. Silts and clays are graded by observing the velocity of fall of

the grains through water. The result of particle size analysis is a grading curve (Fig. 2.1). Reference may be made to the effective size ( $D_{10}$ ) of a material. This is the largest diameter in the lower 10% of the material. The uniformity coefficient is  $D_{60}/D_{10}$ . The value of the grading curve as an index is that correctly naming the soil leads to associations with soil behaviour which have been learnt by general experience.

### ***Atterberg Limits (also called Consistency Limits)***

An important test for clayey soils is the Atterberg limit or consistency test (BS1377). This examines the effect of changes of moisture content on the plasticity of clay soils. In the worst condition, clay could be just a suspension of clay particles in water, which would then act as a rather dense fluid. If the water is allowed to dry out the point will be reached where the material is a very soft solid and exhibits a small shear strength. This limiting value of moisture content is the *liquid limit* (*LL*). As further moisture dries out from the material it becomes stronger. Above a certain moisture content the soil will behave plastically, that is it will deform under stress but not regain its shape when the stress is removed. If the moisture content is less than this *plastic limit* (*PL*) the soil, when loaded, may break with a brittle fracture. The *plasticity index* (*PI*) is the range of moisture content within which the soil acts as a plastic material. Thus:

$$PI = LL - PL \quad (2.17)$$

## **2.10.2 Rocks**

### ***Point Load Test***

Engineering in rock is usually concerned with deformation of rock mass rather than strength of rock material, and material strength tests serve only as indicators of quality. The Point Load test (Broch and Franklin 1972) has been used since 1972 and has become the most popular of the simpler techniques. It is used in laboratory and field for logging mechanical strength of rock specimens obtained from boreholes sunk by rotary core drilling. A core is loaded between two 'points' which are steel cones subtending an angle of  $60^\circ$  and terminating in a hemisphere of 5 mm radius. The core is usually loaded across a diameter. The test specimen should be at least 1.5 times as long as the diameter. The strength value obtained from the Point Load test varies according to core diameter. In the standard test, results are converted to a standard 50 mm diameter core by the use of a conversion chart.

Correlations between Point Load strength ( $I_s$ ) and unconfined compressive strength have been derived from experimental evidence, but unfortunately the multiplying factor appears to vary from one rock type to another. Bowden et al. (1998) record authors giving correlation factors ( $UCS/I_s$ ) ranging from 4 to 50, with the majority falling in the range 10 to 30. For stronger rocks correlation factors of about 24 seem reasonable. For weaker rocks, into which the points may sink before the rock breaks, about half this value may be appropriate. The Point Load test is useful but it has a number of problems (Brook 1985; Price et al. 1978). It may also be noted that, in sedimentary rocks,

much Point Load testing takes place along the bedding, the direction of greatest weakness and thus good correlation with the unconfined compressive strength, measured at right angles to the bedding, seems unlikely.

Enthusiastic and unrestrained Point Load testing on cores may destroy core which could have been used for more complex testing requiring the 2:1 height/diameter ratio. The combination of these problems suggests that other rock strength index tests are to be preferred and the use of the Point Load test on core be discontinued.

### ***Block Punch Test***

The Block Punch test is a test, developed for use as an index test, in which a thin plate or disc of rock is sheared by a punch device (Fig. 2.5). The Block Punch Index (*BPI*) is given by:

$$BPI = \frac{W}{t} \times (L_1 + L_2) \quad (2.18)$$

where  $W$  = failure load;  $t$  = disc or plate thickness and  $L_1$  and  $L_2$  are the lengths of the shear planes.

Extensive testing (van der Schrier 1988) has given correlations with unconfined compressive strength ( $UCS = \pm 6 \times BPI$ ). The accuracy of the test seems to be as good as the Point Load strength test as an index for unconfined compressive strength and it has the advantage that thin plates or discs, about 10 mm thick, can be cut by diamond saw from small pieces or cores of rock. Thus cores too fragmented to fulfil the Point Load test specimen dimension criteria can be tested while longer cores can be tested without destroying those big enough for more complex testing.

### ***Schmidt Hammer Test***

The Schmidt hammer is a device designed to estimate the strength of concrete (but which can be used on rock) by measuring the rebound of a spring-impelled hammer striking on the surface of the mass. Several varieties of the hammer differ in the energy imparted by the spring – the L-type is generally the most satisfactory for rocks. The rebound number is converted to unconfined compressive strength by means of graphs, which are based on laboratory testing evidence. Correlation between rebound number and  $UCS$  is not absolute, and rather like the use of Point Load Strength index, it might be better used for comparison between rocks if the link to  $UCS$  were avoided. The value of rebound is affected by the attitude of the hammer, whether vertically pointing down or up, horizontal or inclined.

### ***Velocity Tests***

The velocity of waves through a material can give some indication of other properties of the material, such as strength, density and elastic moduli. Measurements may be made on intact specimens in the laboratory, or on rock masses in the field. A comparison of laboratory and field velocities may give an indication of rock mass properties.

The velocities of waves through a cylindrical sample of material can be measured accurately. The pulse generated at one end of the specimen is picked up at the other end, and recorded. For a known density the elastic constants can be calculated:

$$E = 2\rho V_s^2(1 + \nu) \quad (2.19)$$

$$\nu = \frac{0.5(V_p/V_s)^2 - 1}{(V_p/V_s)^2 - 1} \quad (2.20)$$

$$K = \rho \left( V_p^2 - \frac{4}{3} V_s^2 \right) \quad (2.21a)$$

$$G = \rho V_s^2 \quad (2.21b)$$

where  $V_p$  and  $V_s$  are compressional and shear wave velocities,  $\rho$  is density,  $E$  is Young's modulus,  $\nu$  is Poisson's ratio,  $G$  is shear modulus and  $K$  is bulk modulus.

### ***Slaking and Slake Durability***

Slaking tests consist of cyclically saturating and drying a fragment of rock and observing its loss of weight after a number of cycles. The slake durability test (Franklin and Chandra 1972) includes an element of erosion for rock fragments are placed in a rotating drum. The test is suitable only for weak or weathered rocks there being generally too little effect on strong fresh rock.

### ***Combined Parameter Indices***

An index may be established using more than one property. An important index is the ductility number (sometimes referred to as the brittleness index) which is the ratio  $UCS/UTS$  (Fig. 2.5). Values of ductility number of more than 15 indicate a rock which will shatter when impacted by cutter tools. Values below about 9 indicate that the rock will crush under tool impact. The brittle/ductile behaviour boundary is of great importance in rock cutting.

## **2.11**

### **Range of Values for Material Properties**

Although the mechanical values for a rock or a soil can vary considerably, they do exhibit a characteristic range of values and this range becomes the sort of value that "would be expected".

2.11.1  
Soils

Some typical values for the properties of granular soils are given in Tables 2.8 and 2.9. Ranges of shear strengths for clays are given in Table 2.2.

The properties of clay soils are very dependent upon their moisture content and they can be much more complex, mineralogically, than more granular soils. Consequently the range of values of properties is much greater (Table 2.10). There are some special ‘problem’ soils encountered in the world, which give particular engineering difficulties. In many cases the properties are due to the mode of origin and/or subsequent geological history, in which environment often plays a leading role. Some examples are given below.

*Loess*

Loess is a yellow, calcareous, porous, wind blown silt, characteristic of Pleistocene sequences in several large areas on every continent. Generally more than 80% of the loess particles consist of quartz, 10 to 20% of feldspar and up to 20% of carbonate. The percentage of clay size particles may range up to 15%. The structure of the material is dominated by the fine quartz particles, between twenty and fifty micrometres in length. The open structure of the quartz particles is often strengthened by clay minerals, and cemented by calcium carbonate. The whole fabric is essentially delicate, which makes the structure liable to sudden collapse. Loess soils cause great difficulties for civil engineers. They tend to collapse on saturation under load, or at least when the moisture content is changed they consolidate very rapidly – 95% of settlement can occur in ten minutes. The effect is local, which can lead to very large differential settlements of structures founded on them. Similar loss of strength may occur after disturbance by other means, for example by large loads or earthquakes.

*Sensitive Clays*

The shear strength of an undisturbed clay is generally greater than the shear strength of the same remoulded clay at the same moisture content. There is a loss of strength on remoulding, and the ratio of the undisturbed to remoulded strength is called the

**Table 2.8.** Some properties of granular soils

Property	Gravel	Sand
Specific gravity	2.5 – 2.8	2.6 – 2.7
Bulk unit weight ( $\text{kN m}^{-3}$ )	14.1 – 22.6	13.7 – 21.6
Dry unit weight ( $\text{kN m}^{-3}$ )	13.7 – 20.6	13.0 – 18.6
Angle of friction ( $^{\circ}$ )	33 – 45	27 – 46
Porosity (%)	25 – 40	25 – 50
Shear strength (kPa)	180 – 550	100 – 400
Permeability ( $\text{m s}^{-1}$ )	$10^{-1}$ – $10^{-4}$	$10^{-3}$ – $10^{-6}$

**Table 2.9.** Some properties of Dutch sands

Unit weight (kN m <sup>-3</sup> )	Relative density (%)	Natural moisture content (%)	$\phi$ (°)	Location
14.8	21	4	30	Eems
15.5	53	8	33	Eems
16.3	67	8	35	Eems
16.0	70	4	40	Eems
17.1	95	8	41	Eems
16.9	91	4	42	Eems
17.1	95	12	43	Eems
18.7	90	12	44	Echteld
20.0	100	8	46	Berg en Terblijt

**Table 2.10.** Some properties of silt and clay soils

Property	Silt	Clay
Specific gravity	2.63 – 2.67	2.55 – 2.76
Bulk unit weight (kN m <sup>-3</sup> )	17.65 – 21.2	14.5 – 21.2
Dry unit weight (kN m <sup>-3</sup> )	14.2 – 19.2	11.6 – 21.2
Void ratio	0.34 – 0.82	0.42 – 0.95
Liquid limit (%)	24 – 36	> 25
Plastic limit (%)	14 – 25	> 20
Permeability (m s <sup>-1</sup> )	10 <sup>-6</sup> – 10 <sup>-9</sup>	10 <sup>-9</sup> – 10 <sup>-12</sup>
Cohesion (kN m <sup>-2</sup> )	< 70	15 – 200
Angle of friction (°)	25 – 35	4 – 17
Coefficient of consolidation (m <sup>2</sup> yr <sup>-1</sup> )	12	5 – 20

sensitivity ( $S_t$ ) of the clay. The measure of sensitivity proposed by Terzaghi is the ratio of the peak undisturbed strength ( $C$ ) to the remoulded strength ( $C_r$ ). Thus:

$$\text{Sensitivity } (S_t) = \frac{C}{C_r} \quad (2.22)$$

All clays show some degree of sensitivity; the term ‘quick’ is applied to clays which have a sensitivity of over 16 (Skempton and Northey 1952). The more sensitive the clay,

the more difficult the engineering problems posed. The term 'quick clay' is sometimes used to denote clays of such sensitivity that they behave as viscous liquids when in a remoulded state. Small earthquakes may trigger of a mass movement which may be responsible for great loss of life and damage to property.

### ***Sabkha***

The term 'sabkha' is used in Arabic speaking countries to mean 'salt encrusted flat surface', of which there are many in coastal and inland areas of Arabia, particularly along the southern shore of the Persian Gulf. In engineering geology and civil engineering, sabkha is now used as a description of geological material originating in particular environments of deposition, and consisting of sandy or silty sediment which is at least partly cemented by salts, usually calcium carbonate and calcium sulphate, but often with a variety of other salts present also. Most geologists apply the term only to the coastal environment. At times of rain, flash flood or extra high tidal level this crust may soften very rapidly.

The geotechnical difficulties may be summarised as follows:

- very variable bearing capacity, vertically;
- very variable deformability (compressibility);
- rapid change due to growth or change, for example hydration, of new or existing minerals;
- effect of salt on concrete and steel.

### ***Duricrust***

If, in a semi-arid climate, the ground-water table lies not far below surface, water drawn to the surface will evaporate and deposit any minerals held in solution within the near surface materials, resulting in their cementation. If the material is already rock then pore spaces become filled and make the rock stronger. The minerals in solution may be leached from materials far below surface so duricrust layers may be underlain by leached material, weaker than it was in its original condition.

The cementing material is most commonly calcite so that form of duricrust is termed *calcrete*. However, a silicious cement gives *silcrete*, a ferruginous cement *ferrocrete* and so forth, while there are many local names for these materials. They are widespread in many parts of the world and much attention has been drawn to their occurrence in the Middle East (Fookes 1978). Thicknesses of duricrust may range from a few centimetres to several metres with considerable changes in thickness in but a short horizontal distance.

Duricrusts are formed near surface and commonly near the sea but there have been considerable variations in climate and sea level in Pleistocene and Holocene times. Because of this duricrusts may now be found underwater (Dennis 1978) or above sea level along shore lines. It is thus important, in a site investigation, to establish the recent geological history of the area under investigation to determine whether or not duricrust is likely to occur.



### Glacial Soils

There is a very large range of glacial soils whose nature and properties vary according to the material available, the environment of transport, i.e. on top of, within, or near the base of the glacier, and the environment of deposition. Thus, tills are soils deposited directly by the ice and fluvio-glacial soils are deposited by meltwater either close to the ice front, i.e. *periglacial*, or at greater distance, i.e. *proglacial*.

The term *till* is used in preference to the more commonly used term ‘boulder clay’ to which it is synonymous. Boulder clay is a generic term rather than a lithological description, but it is generally misunderstood by engineers who may complain with justification that the material described as boulder clay on the map has neither boulders nor clay as constituents. The character of till is determined by a number of factors:

- the position of transport – on or in the glacier
- the nature of the bedrock over which the glacier is passing
- the nature of the rock through which contributory glaciers have passed
- the mode of deposition of the material

Some tills originate from the ‘plastering’ of sediment transported at or very near the base of the glacier and dropped on to the bedrock. Such material would be overridden by the ice and if clayey would be over-consolidated as a result. The material is likely to consist of finely divided ‘rock flour’ produced by the grinding action of the ice. If the bedrock was shale or mudstone, the proportion of clay mineral in the till will be high, and the till will be plastic. If the bedrock is arenaceous or igneous, the clay mineral fraction may be very small, but the primary mineral particles are likely to be of clay size. However, plasticity and moisture content are very different in the two materials, but in both cases there is likely to be a scarcity of boulders. A till originating in this way is described as lodgement till.

Material which is carried along on the surface or within the glacier will be dropped as the ice melts, a process which is gradual. Accumulations of such material are known as ablation tills, and are likely to have a much higher proportion of boulders than lodgement tills. The boulders and gravel sized fractions are angular, and it is common to find clasts of rock which must have been transported long distances. All tills are variable in their nature and consequently in their properties. In general they consist of an assortment of rock debris ranging in size from rock flour to boulders, but all sizes are not usually present. Gap grading is usual.

Glacial debris may be transported by meltwater and thereby become sorted and stratified; these are *fluvio-glacial deposits*. Many such deposits consist of gravel and sand and these frequently have low relative densities. Such pockets of low density are difficult to locate, and the problem is usually anticipated and dealt with by assuming their presence and using compaction equipment before construction begins. Coarse grained fluvio-glacial deposits are often important sources for concrete aggregate.

Of the great variety of fluvio-glacial deposits, *varved* clays are perhaps the most difficult, from a geotechnical point of view. These are deposited in areas away from

the glacier front, in quiet water conditions but where the sediment is contributed on an annual basis. Fine material reaches the depositional area in late winter and early spring, when the meltwater current is slow: in late spring and summer the faster current transports larger sized material, so that in a complete year a varve consists of clay sized particles giving way to silt sized. This variation in particle size produces variation in material properties. Thus, a range of liquid limits from 30 to 80, and plastic limits from 15 to 30, has been reported from a Canadian varved clay, which also has a sensitivity of about 4.

**Organic Soils**

Organic matter in the form of partly decomposed vegetable or animal remains, even when present in very small amounts, may significantly affect the physical and chemical properties of soil. Small amounts of colloidal organic matter in a clay, for instance, will significantly increase both liquid and plastic limit and increase compressibility. The presence of organic matter inhibits the setting of cement.

*Peat* is an accumulation of partially decomposed vegetable material. Complete decomposition is prevented if the vegetation accumulates in water-logged anaerobic conditions, where a series of complex biochemical and chemical changes may interact to produce humus and thereafter peat. There are many varieties of peat depending upon the nature of the vegetation which contributed and the details of the conditions and reactions. However, two main varieties may be distinguished for geotechnical purposes – the fibrous, mainly woody, peat, and the amorphous, granular variety. The difference may be summarised in the table of some geotechnical properties (Table 2.11).

Peats formed in early post-glacial times may occur at depth, buried beneath more recent sediments. Frequently, therefore, the geotechnical problems concern the bearing capacity and consolidation characteristics of the material. For peat, these are difficult to define. Calculations of settlement are difficult because peat does not conform to normal consolidation theory. Amorphous granular peat behaves in a similar manner to clay, but fibrous peat has rather different consolidation characteristics because of the variation in size of the fibrous material and consequently the size of the voids between the fibres.

Where possible, peat is usually removed and replaced before construction. Alternatively, piles may be used to take foundation loads below the peat. If sufficient time is available, the peat may be pre-consolidated by ‘surcharging’; i.e. by loading before

**Table 2.11.** Some geotechnical properties of peat

Property	Amorphous granular peat	Fibrous peat
Void ratio (%)	< 10	< 25
Moisture content (%)	> 500	< 3 000
Bulk unit weight (kN m <sup>-3</sup> )	< 12	5 – 10
Dry unit weight (kN m <sup>-3</sup> )	6.5 – 13	6.5 – 13



**Table 2.13.** Age, location and other characteristics of the limestones whose properties are displayed in Figs. 2.10 to 2.13

Sample No.	Age	Location	Grain size	CaCO <sub>3</sub> [MgCO <sub>3</sub> ] content (%)	Clastic (Cl) or Crystalline (Crys)
1	Magnesian Limestone (Permian)	Yorkshire (UK)	Medium	62 [38]	Cl
2	Chalk (Upper Cretaceous)	near Hull (UK)	Fine	98 [1]	Cl
3	Lower Oolitic Limestone (Jurassic)	Lincolnshire (UK)	Medium	93 [1]	Cl
4	Upper Oolitic Limestone (Jurassic)	Yorkshire (UK)	Medium	84 [1]	Cl
5	Wenlock Limestone (Silurian)	Shropshire (UK)	Coarse	98 [1]	Crys
6	Wooldale Limestone (Carboniferous)	Derbyshire (UK)	Medium	98 [1]	Cl/Crys
7	Vinalmont Limestone (Carboniferous)	Hainault (B)	Medium	99 [1]	Cl/Crys
8	Muschelkalk (Triassic)	Gelderland (NL)	Fine	82 [4]	Cl/Crys
9	'Fossil Limestone' (Carboniferous)	Dinant (B)	Medium	97 [2]	Cl/Crys
10	'Red Reef Marble' (Upper Devonian)	Neuville (B)	Fine	91 [2]	Crys
11	'Grey Marble' (Middle Devonian)	Aywaille (B)	Medium	97 [2]	Crys
12	'Marl' (calcarenite, Maastrichtian, Upper Cretaceous)	Sibbe (NL)	Medium	97 [2]	Cl

density vs. unconfined compressive strength and it can readily be seen that strength increases with density, reaching a maximum as rock density approaches that of calcite. However, if the clastic limestones are separated from the clastic/crystalline and the crystalline, the plot can be presented as two curves, A and B. The clastic rocks are largely homogeneous and fall on a smooth curve (A). However, the remainder have been visually assessed to be variously inhomogeneous, containing textural discontinuities (from, for example, recrystallised fossils) representing potential flaws which, when stressed, may bring the sample to failure. The straight line B indicates the range of strengths of the inhomogeneous limestones suggesting that the chance inclusion and location of textural flaws is the strength determining factor. On this

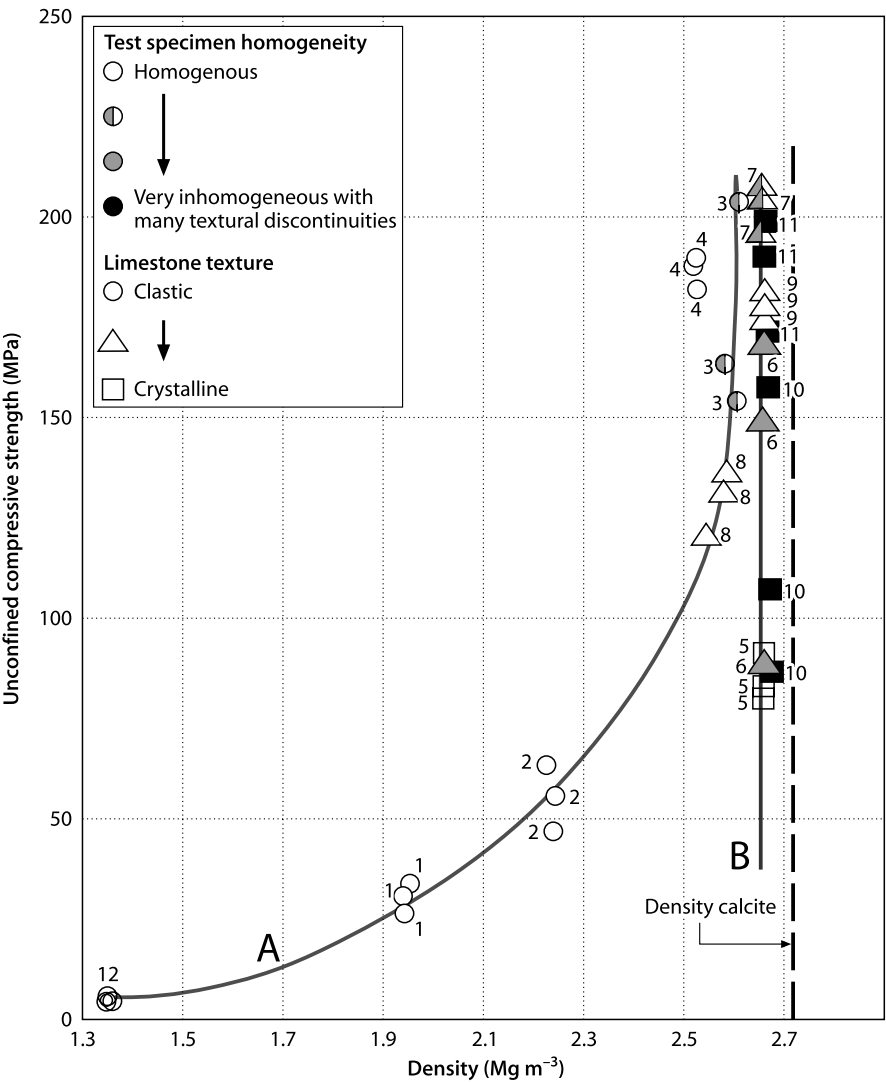


Fig. 2.10. Limestones – unconfined compressive strength vs. dry bulk density (see text)

evidence it would seem unlikely that any rock classification can be based accurately on compressive strength.

Figure 2.11 shows average tangent modulus vs. density; and here almost all the rock types now fit reasonably on a smooth curve, suggesting that for classification purposes elasticity is to be preferred to strength. Figure 2.12 depicts porosity vs. density *and* sonic velocity vs. density. The relationship between porosity and

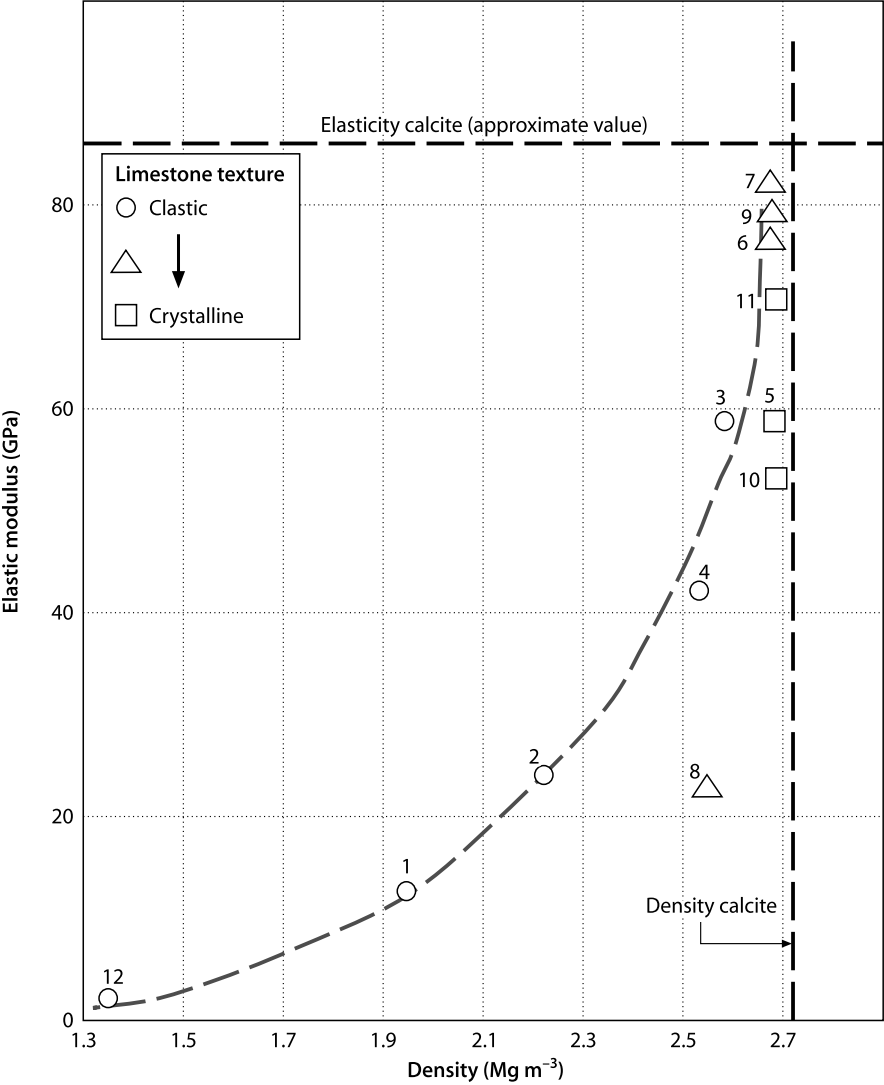


Fig. 2.11. Limestones – tangent modulus vs. dry bulk density (see text)

density is striking while it can be seen clearly that as rock density approaches that of calcite so does rock velocity approach that of calcite. Figure 2.13 combines strength, porosity and velocity plotted against density showing how, for these particular rocks, each of these properties might be used as an approximate guide to strength.

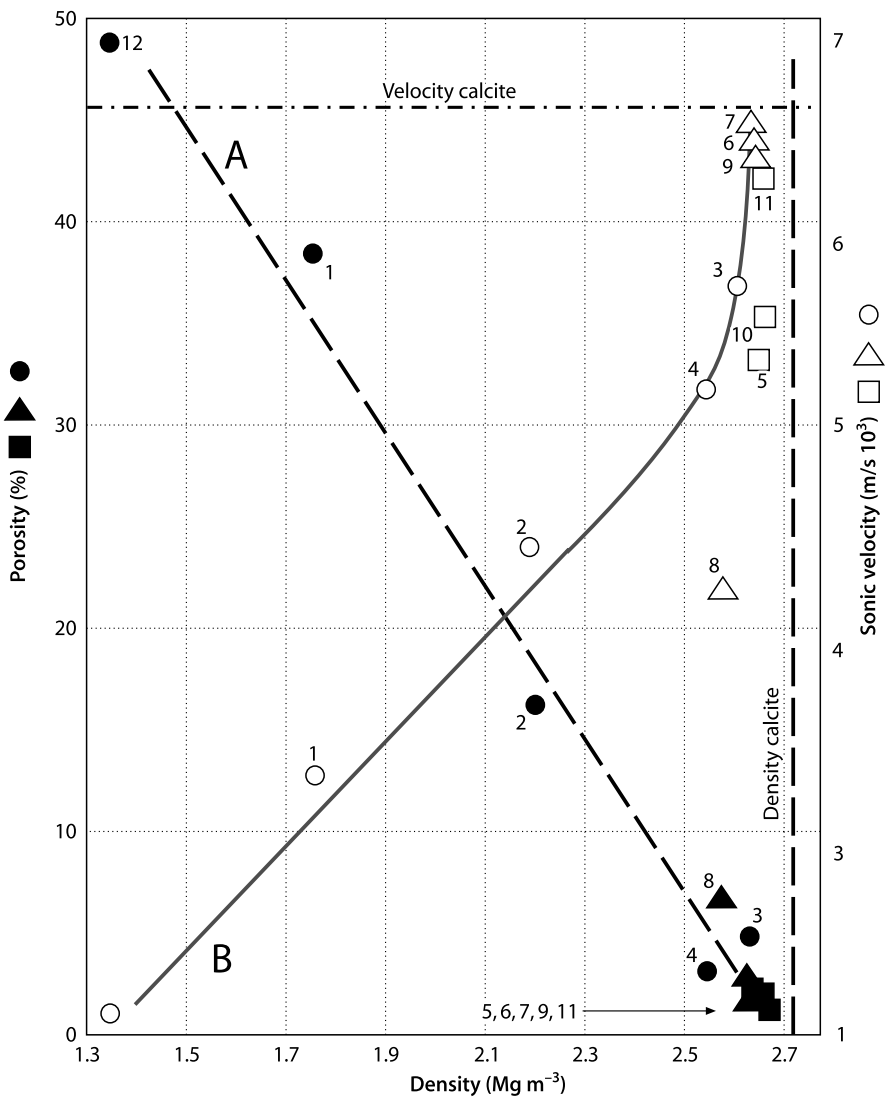


Fig. 2.12. Limestones – total porosity and velocity vs. dry bulk density (see text)

### The Choice of Test for Rocks

While the results of just about all forms of soil testing are used in some form of calculation, the results of some forms of rock testing are seldom so used. Much testing is undertaken simply to gauge the quality of rock material and the investigator must ask

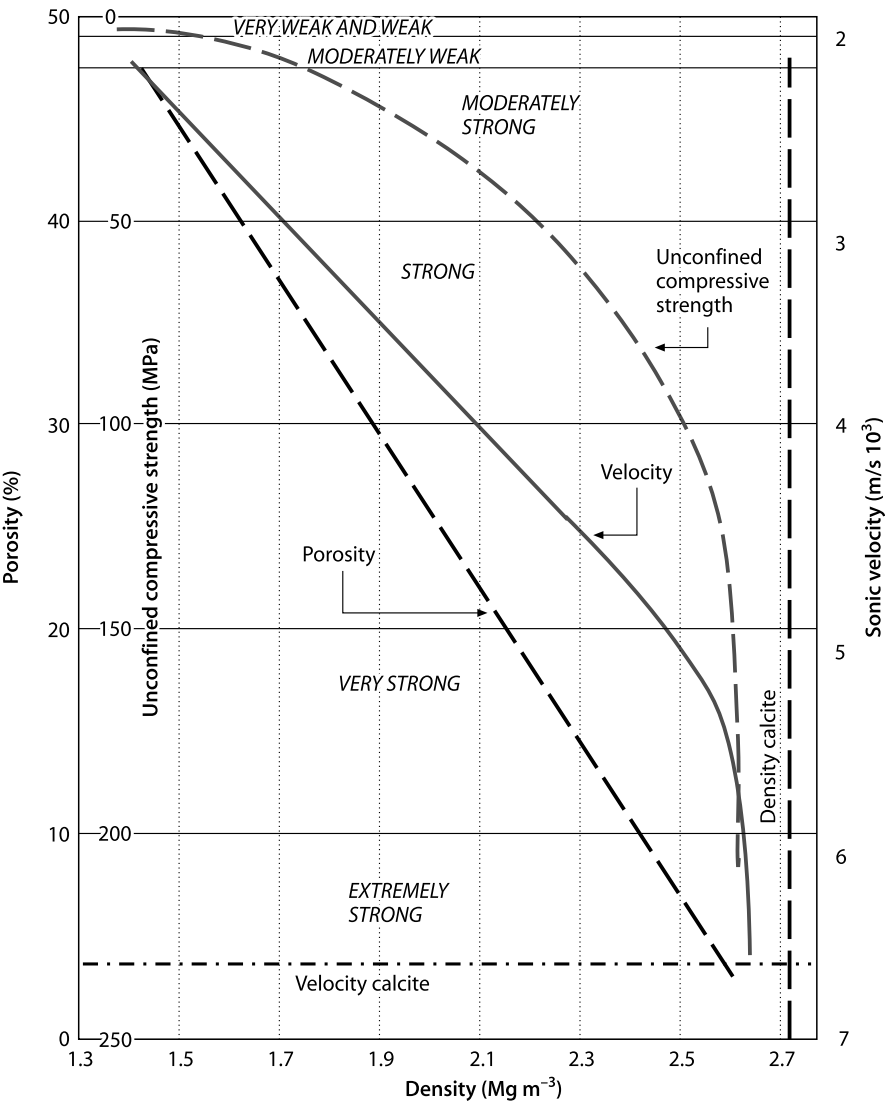


Fig. 2.13. Limestones – correlations with strength (see text)

if the task at hand requires the application of expensive testing or whether a simple quality test, such as hitting the rock with a hammer, would be sufficient to the purpose. However, if such tests are done, they must be well done, to standards. Another difficulty is the choice and availability of material for testing. In examining and testing cores the concern is to identify those weak rocks which could become overstressed by some engineering process. Unfortunately weak layers are those least likely to produce cores sufficiently intact to give test specimens of appropriate dimensions for ac-



curate testing. Accordingly it may well be wiser to place emphasis on those index tests which may be performed on small test specimens, such as the Equotip test (Chap. 3) and the Block Punch test. Such tests could be undertaken in large numbers to gain a better appreciation of rock material variability.

## 2.12

### Further Reading

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