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Chapter 18

Rock behaviour

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Rocks are naturally occurring, polycrystalline materials that play a wide range of roles in civil engineering. Their behaviour can be understood in terms of their intrinsic textures (grains, cement, voids and discontinuities) and through the application of standardised testing methods. Rocks and soils display a wide range of rheological properties, even under near-surface conditions. The void space in rocks is generally filled with water, at some pore pressure. For many engineering applications rocks may be considered as poroelastic, with a proportion of the applied load being supported by the pore pressure. Thus, their deformation is related to the effective stress (= total stress – pore pressure) as is generally found for soils. Rocks have a wide range of stiffness, strength and permeability, all of which will determine their suitability for different engineering applications. Most importantly, rocks are heterogeneous, displaying various forms of layering or grain fabrics, and are almost ubiquitously fractured. Thus the rock mass properties vary over a range of scales (heterogeneity) and orientations (anisotropy).

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18.1 Rocks

Rocks are naturally occurring, polycrystalline materials that play a wide range of roles in civil engineering, ranging from their support for structures, to their use as construction materials.

All rocks can be considered as being made up of the following components (**Figure 18.1**):

> ***Grains*** – discrete elements of either individual crystals or aggregates of crystals, usually having a distinct composition and shape;
> ***Cement*** – usually crystalline material that binds the grains together;
> ***Voids*** – spaces between grains, usually in the form of interconnected pores, often filled with water.
> ***Discontinuities*** – macroscopic surfaces that separate the rock mass into blocks or layers. They may take the form of fractures (or joints), movement surfaces (faults), bedding planes or other surfaces. The discontinuities usually have markedly different physical properties from the rest of the rock material.

The composition and arrangement of these components gives rise to a variety of different rock types (that geologists describe with a bewildering range of names), which have a wide range of engineering properties. *The nature of the grains, cement, voids and discontinuities is important in evaluating the behaviour of rocks and rock masses.* Some understanding of the nature of rocks is necessary in order to discuss their physical behaviour.

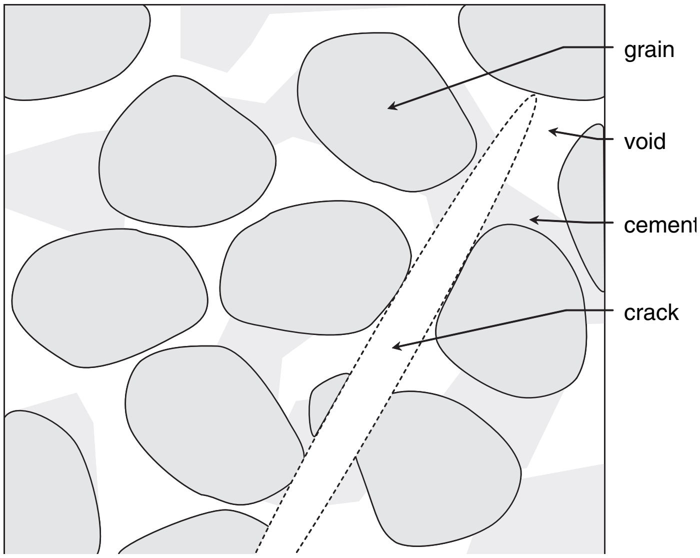

Figure 18.1 Diagrammatic representation of the components that make up a typical rock

18.2 Classification of rocks

Geologists recognise three broad categories of rocks, based on their mode of formation:

> ***Sedimentary rocks*** – that form at or near the surface of the Earth from grains, usually eroded from previously formed

rock. The grains are then transported across the surface and deposited to form new ***sediments***. The main agents for these processes are water (in the form of rivers and seas), ice and wind. We can regard soils as a form of sediment.

Igneous rocks – that form by crystallisation of grains from molten rock either deep within the Earth (magma) or at the Earth's surface (lava).

Metamorphic rocks – that form deep within the Earth due to solid-state recrystallisation of other rock types.

Igneous and metamorphic rocks share many characteristics and consist of an interlocking aggregate of mineral grains, usually resulting in strong and stiff materials. The boundaries between grains are usually very narrow, with no cement, and it is the interlocking nature of these boundaries and the lattice-scale forces between grains that impart their strength. Void space is small, with the porosity (ratio of void space to total volume) generally being $< 1\%$.

Sedimentary rocks on the other hand are generally formed as loose aggregates of grains, initially with a large amount of void space (porosities of 30% or greater). In the Earth, this porosity is usually saturated with water (and occasionally other fluids). The water may contain dissolved materials that can precipitate in the pore space to form cement.

At an initial stage, when porosities are high, the sediments resemble soils in their mechanical behaviour (see Chapter 17 *Strength and deformation behaviour of soils*). As the sediments are buried they experience elevated pressures and temperatures, leading to consolidation (usually by expulsion of water from the void space) and cementation. These processes essentially convert the 'soil' to rock. Near-surface chemical processes, generally termed diagenesis by geologists, accelerate the conversion of sediment to rock.

There is no clear boundary between rocks and soils, and many of the methods used to describe their behaviour are based on the same principles of continuum and granular mechanics. Both materials can exhibit considerable heterogeneity, often an important factor in assessing their engineering behaviour, but some important differences exist between the two.

Soils are essentially particulate (or granular) materials, with particle sizes that are usually many orders of magnitude less than the length scales of the imposed engineering loads. Thus they are generally treated as continuum materials, their microscale granularity being approximated by macroscopic parameters. On the other hand, the block sizes of rocks are often of a similar length scale to the applied loads and the discrete nature of rock masses is usually of greater importance in rocks than for soils (e.g. Hudson and Harrison, 1997).

Generally, both rocks and soils contain water, the void space often being saturated. Thus in both materials the effects of fluid pressure and the principles of effective stress are of great importance. However, the fundamental behaviour of soil changes dramatically with water content, changing from solid → plastic → liquid at the plastic and liquid limits, respectively. Rock properties are also affected by water, but not to anything like the same extent; for example, the strengths of dry and saturated rocks rarely differ by more than a factor of two.

18.3 Rock composition

An important distinction between the crystalline (igneous and metamorphic) and sedimentary rocks is the composition of the grains. The bulk composition of the Earth's crust is dominated by the elements silicon (Si) and oxygen (O) that readily combine to form the silica ion (SiO_4^{4-}). The next most abundant elements are the metals aluminium (Al), iron (Fe), magnesium (Mg), calcium (Ca), potassium (K) and sodium (Na), which form positively charged ions and combine with the SiO_4^{4-} to form a group of minerals known as rock-forming silicates. These include the common minerals quartz and feldspar, together with a complex array of other silicates (pyroxenes, amphiboles, micas, etc.). Most igneous and metamorphic rocks consist of a small number, typically three or four, of these rock-forming silicates, the composition of which depends on the chemistry of the patent magma or rock, and the pressure–temperature conditions under which the rock formed. The details of these minerals need not concern us here; what is important is that they generally have moderate to high strengths, contributing to the strength of igneous and metamorphic rocks.

Many of these rock-forming silicates weather on exposure to the Earth's surface, and are converted to various types of clay minerals and other salts; the latter often dissolve in water. An exception is the mineral quartz, which is highly resistant to weathering. Weathering leads to a reduction in strength of igneous and metamorphic rocks, and is facilitated by fracturing of the rock mass. Thus a key issue in the behaviour of such rocks is the rock mass quality.

Weathering leads to the development of sediment – hence, we would expect this to be composed mainly of quartz and clay minerals. Sediment is also generated by the precipitation of dissolved salts, either through evaporation (e.g. rock salt) or more commonly by biochemical action, mainly involving fixing of dissolved salts in the shells of invertebrate organisms. The nature of the sediment determines the type of sedimentary rock that is subsequently produced:

Quartz sand	→	sandstone
Clay minerals	→	claystone (also called mudstone or shale)
Calcite shells	→	limestone
Dissolved salts	→	rock salt (and other 'evaporates')
Organic matter	→	coal, oil, etc.

18.4 Porosity, saturation and unit weight

From the previous description of rock in terms of grains, cement and voids, we can consider the grains and cement together as a solid phase (**Figure 18.2**), with the voids being either liquid (pore water) and/or gas (air).

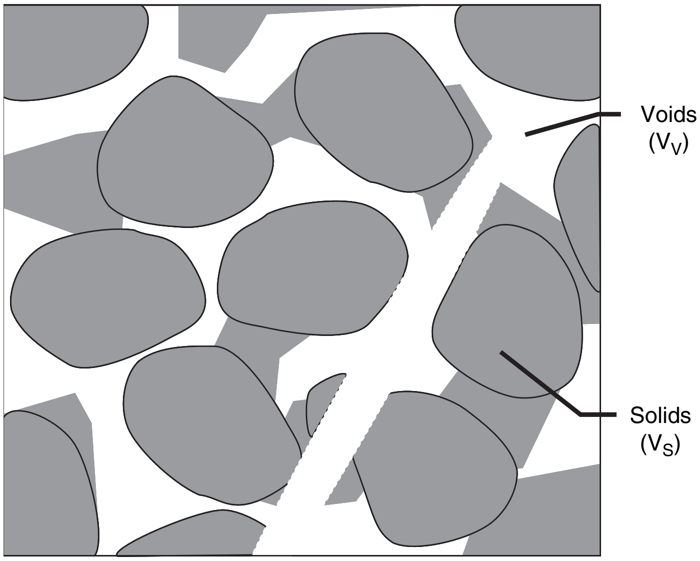

Figure 18.2 Rock components (as Figure 18.1) classified into solids (V_S) and voids (V_V)

The volume of the rock (V_R) is made up of solids (V_S) and voids (V_V), where $V_R = V_S + V_V$. The relative proportions of these define two parameters:

$$\textit{Porosity}\ (n) = V_V / V_R \text{ or } V_V / (V_S+V_V) \qquad (18.1a)$$

$$\textit{Voids ratio}\ (e) = V_V / V_S \qquad (18.1b)$$

where $e = n / (1 + n)$ and $n = e / (1 + e)$. (18.1c)

Since the volume of voids can be filled with water and/or air: $V_V = V_W + V_A$, which we can express in terms of the **saturation** S, where:

$$S = V_W / V_V. \qquad (18.2)$$

Porosity, voids ratio and saturation are usually expressed as a fraction (0 – 1), but can be expressed as a percentage.

Knowing the densities (ρ) of the solid, water and air phases, and using g = 9.81, the weights of the phases may be calculated, where $W = \rho\, g$.

For example, ***unit weight*** $(\gamma) = (W_S + W_W + W_A) / (V_S + V_W + V_A)$ (18.3a)

$$\textit{unit dry weight}\ (\gamma_d) = W_S / (V_S + V_W + V_A). \qquad (18.3b)$$

These parameters are discussed more fully in Chapter 17 *Strength and deformation behaviour of soils*, but can provide a useful basis for characterising the multi-component nature of rocks and, hence, assessing rock behaviour. For example the uniaxial compressive strength (UCS) is related empirically to the porosity for different rock types, which provides a useful guide to rock strength in the absence of tests.

18.5 Stresses and loads

Stress exists throughout the Earth's crust and may be changed by surface and subsurface construction. There are four universal components responsible for this stress:

(1) The weight of the overlying column of rock, which is a relatively simple function of depth and rock density (unit weight), and is usually referred to as **overburden stress** (σ_V).

(2) **Fluid (or pore) pressure (P)**, which arises from the fact that most rocks are basically two-phase materials consisting of mineral grains (which may or may not be cemented together) and fluid-filled pores and cracks.

(3) **Thermal stresses** arising from heating or cooling of rock, which tends to cause rocks to expand or contract.

(4) **Externally applied loads** that may be imposed by geological processes (tectonics, topography, etc.) or by construction.

These four components interact in different ways, but their combined effect is to act on or load materials to induce strain (change in shape and/or volume). In civil engineering we are mainly concerned with how the ground responds to externally applied loads, but we should not overlook the possible effects of other sources of stress.

Most rocks contain pores and cracks that are generally saturated by water. The externally applied loads create both a stress in the framework of grains and cement, and a pressure in the fluid, which we refer to as fluid pressure or pore presure. Terzaghi (1943) suggested that the applied loads may be supported by both the stress in the solids and the fluid pressure in the pores. At the grain boundaries, these two tractions oppose one another and create an **effective stress**, such that:

$$\text{Effective stress} = \text{Total stress} - \text{Pore pressure} \qquad (18.4a)$$

$$\sigma' = \sigma - P_f. \qquad (18.4b)$$

In a granular material, it is this effective stress that promotes deformation, with the relationship between effective stress and strain being determined by the rheology and properties of the material. The effective-stress principle is applied almost universally in soil mechanics and widely in rock mechanics. We will discuss this further in the section on poroelasticity (section 18.8).

18.6 Rock rheology

There are three basic responses of materials to applied stress that are easily recognised from plots of stress against strain or strain rate (**Figure 18.3**):

Elasticity – where the strain is linearly proportional to the stress (**Figure 18.3(a)**). This typifies the behaviour of solid materials and the ratio of stress to strain is referred to as the stiffness (Young's modulus, rigidity, etc.). In the ideal case, the deformation is completely recovered on removal of the stress and the rock exhibits no significant change in structure. Many crystalline rocks approximate this behaviour and are fairly rigid, i.e. they exhibit a high stiffness, with Young's modulus being measured in GPa. This is one reason for their widespread use as construction materials.

Viscosity – is where the material flows and is the basic characteristic of liquids. In rock mechanics we often perform ***creep*** experiments where a sample is allowed to deform under a constant applied stress and the strain plotted against time, the slope being the strain rate (**Figure 18.3(c)**). If the strain rate is proportional to stress, the material exhibits linear or Newtonian viscosity, where the viscosity is the ratio of stress to strain rate. Some rocks exhibit Newtonian viscosity, but more generally the behaviour is more complex (nonlinear viscosity).

Yield – occurs when a material behaves elastically at low stress, but is ductile (i.e. flows) at higher stresses. The stress at which this transition takes place is the yield stress (**Figure 18.3(b)**). This behaviour is typical of plastic materials.

Rocks, in common with most other materials, exhibit all of these basic rheologies, the stiffness, viscosity and yield stress often being complex (nonlinear) and dependent on the temperature, confining pressure and strain-rate.

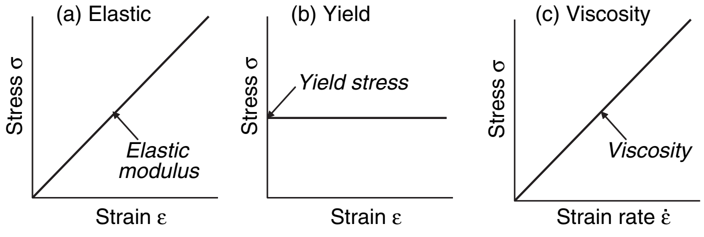

Figure 18.3 Idealised rheological behaviour showing (a) linear elasticity, (b) plastic yield and (c) Newtonian viscosity

It is useful, at least from a conceptual basis, to model rocks by combining the three basic rheological elements in various ways. For example, the stress–strain curves for rock (**Figure 18.4(a)**) at room temperature are elastic, whereas high confining pressures show a close correspondence to an elastic-plastic rheology. Confining pressure effectively inhibits failure and the rock exhibits a reasonably clearly defined yield stress at ~500 MPa (**Figure 18.4 (a)**). Yield stress and the stiffness decrease with increasing temperature (**Figure 18.4(b)**), as does the viscosity (not shown in **Figure 18.4**).

The rheology of a material is described by a ***constitutive law*** – an equation relating the deformation (strain) induced in the rock to the applied stress (or *vice versa*). We have encountered simple examples of these in the previous section. However, to define a constitutive law more rigorously we need to consider the relationships between different components of stress and strain. In general, the deformation can be described by a second-order strain tensor, ε_{ij}, which is related to the displacements (u):

$$\varepsilon_{ij} = \tfrac{1}{2}\left(\frac{\delta u_i}{\delta x_j} + \frac{\delta u_j}{\delta x_i}\right). \qquad (18.5)$$

Three commonly encountered deformation types (unconfined compression, simple shear and volumetric strain) are illustrated in **Figure 18.5**.

18.7 Elasticity and rock stiffness

For elastic behaviour, the relationship between stress and strain is linear and, for small deformations, can be described by a series of stiffness (or elastic) constants, defined by the ratio of different stress-to-strain components. The following elastic constants are widely used and relate to the three common deformation types (**Figure 18.5**) as follows:

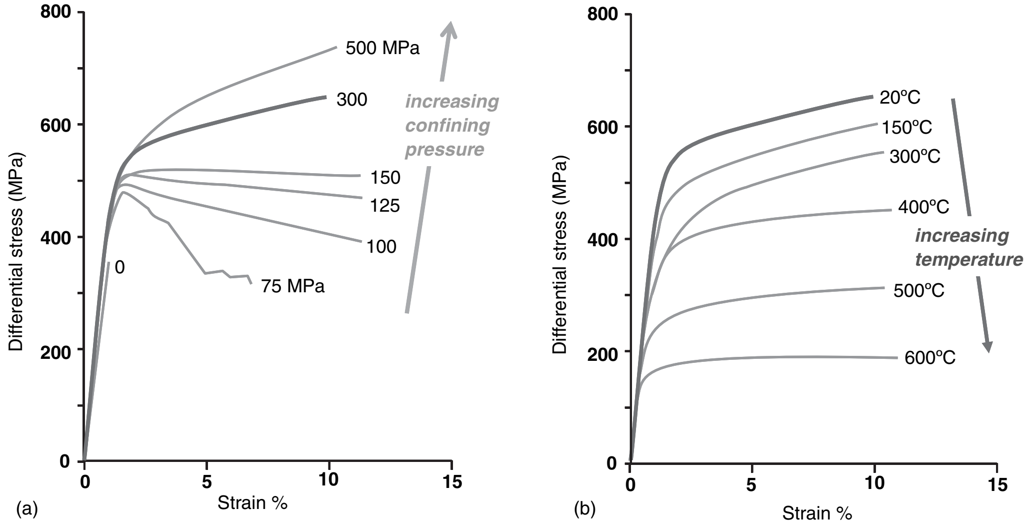

Figure 18.4 Stress–strain curves from triaxial tests on Solenhofen Limestone (after Heard, 1960). All tests at strain rate of 2×10^{-4} s^{-1}; (a) changing confining pressure at constant temperature of 25°C; (b) changing temperature at 300 MPa confining pressure

Young's modulus (***E***) where $E = \sigma_{33} / \varepsilon_{33}$. This is the stiffness in unconfined (uniaxial) compression (**Figure 18.5(a)**), where σ_{33} is the only non-zero stress although there will be lateral strains in the other principal directions (i.e. $\varepsilon_{11} = \varepsilon_{22} \neq 0$).
Poisson's ratio (ν) where $\nu = - \varepsilon_{33} / \varepsilon_{11}$, is the ratio of lateral to axial strains in uniaxial compression.
Shear modulus (***G***) where $G = ½ (\sigma_{31} / \varepsilon_{31})$; it is easily determined from simple shear experiments (**Figure 18.5(b)**).
Bulk modulus (***K***) where $K = \sigma_{00} / \varepsilon_{00}$ and σ_{00} is the mean stress or uniform confining pressure (**Figure 18.5(c)**). The **compressibility** (c) is simply $1/K$.

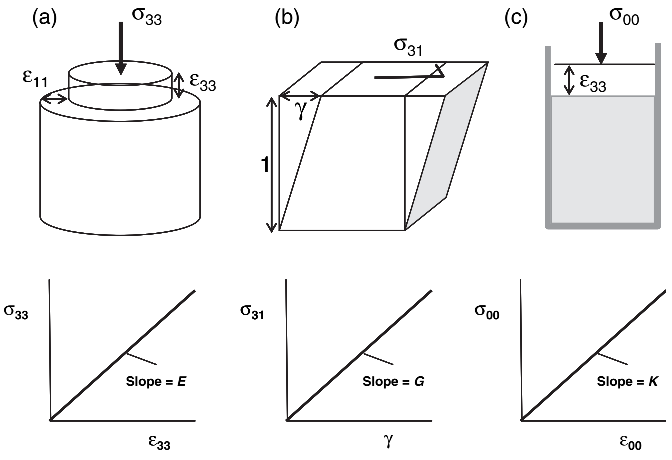

Figure 18.5 Three types of deformation involving different components of stress and strain and leading to different descriptions of rock stiffness

More generally the **constitutive equation** for linear elasticity is given by the equation:

$$\sigma_{ij} = \lambda\, \delta_{ij}\, \varepsilon_{00} + 2G\, \varepsilon_{ij} \tag{18.6}$$

where δ_{ij} is the Kronecker delta, which has a value 1 if i = j and 0 if i ≠ j. The term λ is a fifth elastic constant know as **Lame's constant** and, together with G, provides a more concise mathematical formulation of the constitutive equation in the theory of elasticity. The other, more practical stiffness constants can be related to λ and G, as follows

$$K = \lambda + 2\, G / 3 \tag{18.7a}$$

$$E = (3\lambda + 2G) / (\lambda/G + 1) \tag{18.7b}$$

$$\nu = \lambda / [2(\lambda + G)]. \tag{18.7c}$$

Any two elastic constants can be used to define the material and, hence, all other constants (**Table 18.1**). The range of elastic properties in natural materials is very large, especially if we include water, as illustrated in **Table 18.2**.

The discussion of elasticity presented above assumes that rocks are homogeneous and isotropic. In reality a rock mass is inhomogeneous (properties vary with location) and anisotropic (properties vary with direction). A major contribution to anisotropy comes from the grain fabric due to depositional processes (commonly parallel to bedding) and ductile deformation. Fractures also contribute significantly to anisotropy, both on a macroscopic scale and due to microcracks (e.g. Goodman, 1989; Hudson and Harrison, 1997). This elastic anisotropy is

	Young's modulus E	Poisson's ratio ν	Bulk modulus K	Rigidity (Shear modulus) G	Lame constant λ
E, ν			$\frac{E}{3(1-2\nu)}$	$\frac{E}{2(1+\nu)}$	$\frac{E\nu}{(1+\nu)(1-2\nu)}$
E, G		$\frac{E}{2G}-1$	$\frac{EG}{3(3G-E)}$		$G\frac{E-2G}{3G-E}$
E, K		$\frac{3K-E}{6K}$		$\frac{3KE}{9K-E}$	$\frac{3K(3K-E)}{9K-E}$
ν, G	$2G(1+\nu)$		$\frac{2G(1+\nu)}{3(1-2\nu)}$		$\frac{2G\nu}{(1-2\nu)}$
ν, K	$3K(1-2\nu)$			$\frac{3K(1-2\nu)}{2(1+\nu)}$	$\frac{3K\nu}{(1-\nu)}$
ν, λ	$\frac{\lambda(1+\nu)(1-2\nu)}{\nu}$		$\frac{\lambda(1+\nu)}{3\nu}$	$\frac{\lambda(1-2\nu)}{2\nu}$	
K, G	$\frac{9KG}{(3K+G)}$	$\frac{3K-2G}{2(3K+G)}$			$K-\frac{2G}{3}$
λ, K	$9K\frac{K-\lambda}{3K-\lambda}$	$\frac{\lambda}{3K-\lambda}$		$3\frac{K-\lambda}{2}$	
λ, G	$G\frac{3\lambda+2G}{\lambda+G}$	$\frac{\lambda}{2(\lambda+G)}$	$\lambda+\frac{2G}{3}$		

Table 18.1 Conversion of elastic constants

	Crystalline rock	Soil	Water
Youngs's modulus (MPa)	4×10^4	$10^1 - 10^2$	0
Bulk modulus (MPa)	2×10^4	10^{-1}	2.2×10^3
Rigidity (MPa)	2 × 104	$10^1 - 10^2$	0
Yield stress (MPa)	$\sim 2 \times 10^2$	$10^{-3} - 10^{-1}$	0
Poisson's ratio	0.1 – 0.25	0.2 – 0.45	0.5
Viscosity (Pas)	$\sim 10^{19}$	$\sim 10^4$	10^{-4}

Table 18.2 Simplified material properties of rock, soil and water (liquid)

manifest in directional variation in seismic wave velocities, but discussion of this is beyond the scope of this study. We will return to anisotropy of rock mass strength in a later section.

18.8 Poroelasticity

Poroelasticity is an attempt to describe the deformation of a solid material with connected voids (pores) that are saturated in fluid, and is a useful description for many soils or rocks (e.g. Biot, 1941; Wang, 2000).

Consider a rock under an isotropic confining pressure σ_C, with pores that are subject to a fluid pressure P. The volumetric strain e will be affected by both pressures σ_C and P. We can conceptualise the material as comprising a solid 'framework' and a series of pores, shown schematically in **Figure 18.6(a)**. Based on the law of superposition, this stress system can be considered as the addition of (a) a confining stress (σ_C -P) acting on the outer boundary, with no pore pressure (**Figure 18.6(b)**), and (b) the pore pressure P acting on all boundaries (**Figure 18.6(c)**).

The total volumetric strain e_A is simply:

$$e_A = e_B + e_C \tag{18.8}$$

but:

$$e_B = 1/K\,[\sigma_C - P] \text{ and } e_C = 1/K_G\,[P] \tag{18.9}$$

where K and K_G are the bulk modulii of the rock and mineral grains, respectively.

Combining equations (18.8) and (18.9) and rearranging gives:

$$e_A = 1/K\,[\sigma_C - \alpha\,\Delta P]. \tag{18.10}$$

Thus, deformation is produced by an effective stress:

$$\sigma' = K\,e_A = \sigma_C - \alpha\,P \tag{18.11}$$

where $\alpha = (1 - K/K_G)$ is a dimensionless constant, usually referred to as the Biot constant. Note that equation (18.11) is identical to (18.4b), and hence Terzaghi's effective stress principle, for the case $\alpha = 1$.

For unconsolidated materials, where $K << K_G$, the Biot constant is $\alpha \approx 1$, as was originally proposed for soils (Terzaghi, 1943). For unfractured, crystalline rock the grain framework may be stiffened by cement and/or interlocking grains, thus $K \rightarrow K_G$ and $\alpha \rightarrow 0$. Partially cemented sediments and fractured rock would be expected to have $0 < \alpha < 1$. For example, the bulk modulus for quartz is ~38 GPa, but that of sandstones is around 5–10 GPa; therefore we would expect $\alpha = 0.75$–0.9.

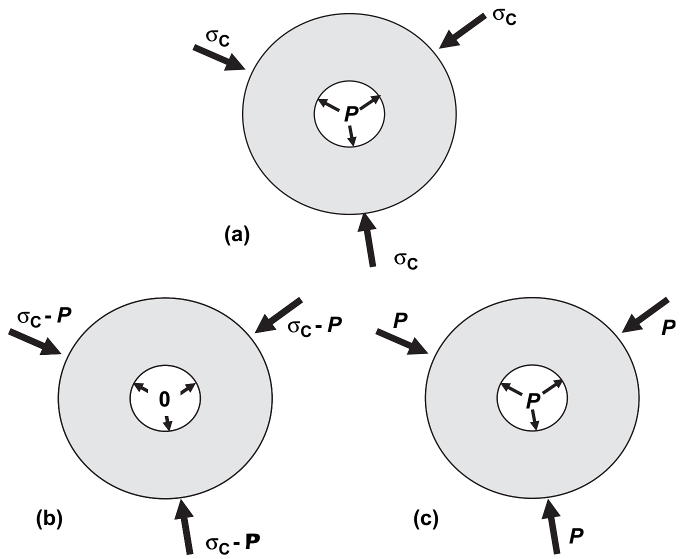

Figure 18.6 Decomposition of confining stress (σ_C) and pore pressure (P) – for explanation see text

18.9 Failure and rock strength

In simple terms, we can recognise two broad types of failure in rocks:

(a) Brittle failure is where the rock undergoes some sort of fracture, usually accompanied by a volume increase (as cracks develop) and preceded by an essentially elastic behaviour of the rock mass.

(b) Ductile failure is where the rock undergoes some form of plastic yielding, usually accompanied by a volume decrease (collapse of pore structure) and preceded by some form of elastic compression or compaction.

Traditionally rock mechanics has focused on the brittle type of failure and soil mechanics on ductile yielding, but in reality both types of failure are seen in both rocks and soils.

The strength of a rock can be considered as the stress at which some sort of failure occurs. In rocks, strength is a complex concept and will depend not only on the properties of the rock components (grains, cement, voids and discontinuities) and their interactions, but on the type of failure and the conditions under which it occurs. Thus any specification of rock strength must include a careful description of the test conditions, or have been obtained under standard (i.e. pre-specified) conditions.

A simple view of rock failure is represented in **Figure 18.7**, which incorporates three widely recognised types of failure criteria.

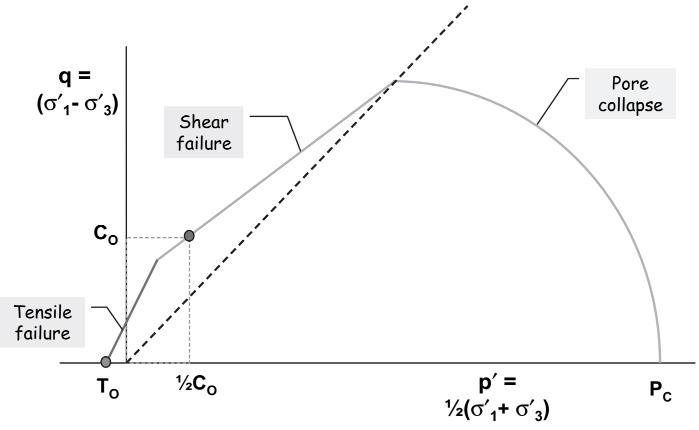

Figure 18.7 Plot of differential stress (q) against mean effective stress (p′) showing the three main types of failure mechanism T_O – tensile strength, C_O – uniaxial compressional stress (UCS), P_C – preconsolidation pressure

Tensile failure occurs when one of the principal effective stresses is negative (tensile) and cracks form normal to the minimum principal stress (σ_3), the magnitude of which is the tensile strength (T_O) i.e.

$$T_O = -\sigma_3. \tag{18.12}$$

The concept is simple, but direct tensile testing is difficult to achieve and interpret, and most tensile strengths are measured indirectly, e.g. from a Brazilian test. For most rocks tensile strengths are low, generally 0–30 MPa.

Following the work of Griffith (1921), we now appreciate that materials fail in tension due to the growth of micro-cracks. For a uniaxial remote tensile stress (σ_r), failure will occur when the stress concentration at the crack tip attains a critical value (K_C – the critical stress intensity factor or fracture toughness).

$$K = Y\,\sigma_r\,(\pi a)_{\frac{1}{2}} \tag{18.13}$$

where a is the half-length of the crack and Y is a factor (usually ~1) that depends on the geometry of the crack and sample.

Combining equations (18.12) and (18.13) and using $Y = 1$ gives:

$$T_O \approx K_C\,(\pi a)_{-\frac{1}{2}}. \tag{18.14}$$

Thus the tensile strength is a function of fracture toughness (a material property) and crack length (a textural property). The micro-cracks and other defects in an unfractured rock generally approximate to the size of the grains or pores (10^{-3} to 10^{-4} m) and K_C is in the range 0.3–3 MPa m^{½}. Thus, a tensile strength of 5–170 MPa would be expected for most rocks, as is observed.

Shear failure occurs on planes oblique to the principal stresses that, hence, experience a shear stress. A simple and widely used criterion was developed by Coulomb, which states that the shear stress (τ) must exceed a linear function of the normal stress (σ_n), such that:

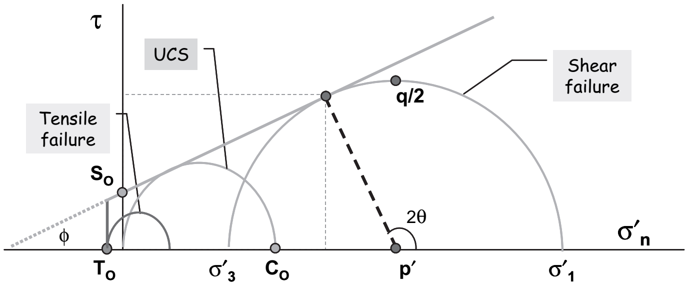

Figure 18.8 Mohr diagram (plot of shear stress, τ, against normal effective stress, σ'_n) showing conditions for tensile and shear failure T_O – tensile strength, C_O – uniaxial compressional stress (UCS), S_O – cohesion, φ – angle of friction, p' mean effective stress, $q/2$ – maximum shear stress = ½ (differential stress)

$$\tau \geq S_O + \mu\,\sigma_n \tag{18.15}$$

where S_O is the cohesion – the shear stress required for failure in the absence of a normal stress, and μ is the coefficient of internal friction. This behaviour is analogous to frictional sliding (see section 18.11).

Coulomb failure is represented by a straight line on a plot of shear stress against normal stress – the Mohr diagram (**Figure 18.8**). Failure will occur on planes where 2θ is normal to the failure envelope, i.e. $2\theta = 90 + \varphi$, where φ is the angle of friction and $\mu = \tan(\varphi)$. For most rocks φ = 20º–50º, hence θ = 55º–70º, with shear fractures forming at 20º–35º to the maximum compressive stress (σ_1) – a commonly observed orientation in tests and nature.

When triaxial test data are plotted on either the Mohr diagram or on a plot of σ_1 against σ_3 they tend to show a nonlinear relationship. Hoek (1968) originally proposed an empirical failure criterion for rocks, which was modified by Hoek and Brown (1980) to:

$$\sigma_1 = \sigma_3 + [\mathrm{m}\,C_O\,\sigma_1 + \mathrm{s}\,C_O^2]^{½} \tag{18.16}$$

where C_O is the uniaxial compressive strength, and the parameters m and s are chosen to best fit the data. For more discussion of this criterion see Chapter 49 *Sampling and laboratory testing*.

Tensile and shear failure both involve the development of cracks and are generally accompanied by small volume increases (dilatency). At high confining pressures, many porous rocks show a form of plastic yielding associated with localised **pore collapse** and compaction. This behaviour is typical of soils (Chapter 17 *Strength and deformation behaviour of soils*). This is shown schematically in **Figure 18.7** by the addition of an ‘end cap’ which meets the p′ axis at a pressure equivalent to the pre-consolidation pressure in soils. Stress paths, such as that produced by hydrostatic compression (q = 0), that lead to failure by pore collapse have a lower ratio of q/p than that for shear failure. Such deformation is best analysed using the techniques of critical state soil mechanics (see Chapter 17 *Strength and deformation behaviour of soils*).

18.10 Strength testing

The field and laboratory testing of rock samples is described in detail elsewhere (Chapters 47 *Field geotechnical testing* and 49 *Sampling and laboratory testing*). Tests are usually performed on rock cylinders of standard shape and size, and some are briefly introduced below.

- **Uniaxial (unconfined) compressive test –** A simple and widely used test in which a cylinder or cube of rock is compressed between platens with no confining stress. The stress producing failure is the unconfined compressive strength (UCS).
- **Brazilian test** – A cylinder of rock is loaded between two platens transverse to its axis. This test is used to determine the tensile strength (T).
- **'Triaxial' test** – Axial compression (σ_1) of a cylinder under a radial confining stress ($\sigma_2 = \sigma_3$) – geometry similar to UCS. This test is usually run at several confining stresses and the failure envelope constructed, usually by plotting on a Mohr diagram (as in **Figure 18.8**).
- **Hydrostatic compression test** – This is usually carried out in a triaxial rig and involves increasing the confining pressure, in the absence of an axial load, until there is a volumetric collapse of the pore space. This is similar to the pre-consolidation pressure in soil mechanics and is used to define the plastic yield surface (**Figure 18.7**).
- **Shear tests** – The direct shear test involves a simple shear loading of a rectangular prismatic sample in a shear box. It is widely used to measure the shear strength (S_S) of soils, but is not suitable for anything but the weakest of rocks. A **torsional ring-shear test** has been developed that involves the twisting of a hollow disk between rigid end-disks to impart a shear on the specimen, which can be applied to a wider range of rock strengths.
- **Point load test** – This is a widely used test involving transverse loading of a cylinder of rock between two conical 'points' of standard shape (60° conical angle and tip radius of 5 mm). It can be performed with portable apparatus and applied to borehole core and irregular rock samples.

Other, field-based methods of measuring rock strength have been developed, which usually rely on an empirical calibration of some strength measure (usually UCS) to some physical response measured in the apparatus. A good example of this approach is the widely used **Schmidt hammer**, which measures the rebound of a spring-loaded rod propelled against a rock surface to estimate UCS. A **scratch test** determines the normal and tangential forces required to attain a constant depth of scratch on a rock surface, which can be related empirically to UCS.

A scheme for characterising rock strength based on simple field classification is outlined in **Table 18.3**. Essentially this is a subjective scheme based on the response of the rock to a series of simple physical tests using one's hand or a hammer. The scheme conforms broadly with the Working Party Report (Geological Society of London 1977) and found in Clayton *et al*. (1995) and Waltham (2009).

In terms of using these tests to describe rock behaviour it is important to remember that they are usually carried out on small pieces of the rock mass, typically at the cm-scale. Such tests may be useful in the characterisation of pieces of aggregate and building stone, but do not directly characterise the resultant structures (concrete, wall or foundation). Nor do the tests relate directly to the behaviour of the rock mass as a whole in groundworks, slopes, tunnels, etc. The following considerations are important in utilising laboratory estimates of rock strength.

- **Sampling –** Is the small specimen used in a test representative? All rocks are heterogeneous, usually at a wide range of scales – does the sampling capture this heterogeneity? This is particularly important in evaluating many sedimentary rock units that are made up of layers of different rock types and textures, with samples of a suitable size often being much easier to obtain from the thicker and stronger layers. These effects may be less of a problem in igneous and metamorphic rocks, although the fabrics in the latter produce anisotropy, which requires careful treatment (see section 18.16).

Description	UCS (MPa)	φ	Cohesion (MPa)	Field test	Rock type(s)
Very strong rock	300	50	20	repeated hammering to break	most igneous rocks
Strong rock	100	45	9	breaks with hammer	greywacke, quartzite, gneiss
Average rock	30	40	3.3	dented by hammer	sandstone, limestone
Moderately weak rock	10	35	1.35	cannot be broken by hand	shale, claystone
Weak rock	3	32	0.46	crumbles under hammer blows	soft chalk
Very stiff soil / Very weak rock	1	30	0.17	easily broken by hand	sand
Stiff soil	0.3	28	0.054	indented by finger nail	marl
Firm soil	0.1	25	0.020	moulded by fingers	clay
Soft soil	0.03	22	0.007	easily moulded by fingers	clay

Table 18.3 Field estimation of rock strength (UCS, internal friction angle for intact rock (φ) and cohesion)

- ***In situ* conditions** are difficult to replicate in a laboratory test for a number of reasons. Changes in boundary stress, and fluid pressure and saturation occur during the extraction, transport and storage of samples that can lead to permanent physical and chemical damage, which cannot be reversed under test conditions. This is common in soils and poorly consolidated rocks, but is less important in most rocks.
- Virtually all routine tests are on intact rock and *do not measure the contribution of the rock fractures.*

18.11 Behaviour of discontinuities

Discontinuities are macroscopic surfaces that separate the rock mass into blocks or layers. They include fractures (or joints), movement surfaces (faults) and bedding planes or other surfaces. The discontinuities usually have markedly different physical properties from the rest of the rock matrix and may fail in tension or shear.

Experiments of sliding on planar surfaces cut in rock show a simple linear relationship between the shear stress (τ) and normal effective stress (σ_n') at low normal stresses (**Figure 18.9**), such that:

$$\tau = \mu_S \, \sigma_n' \tag{18.17}$$

where μ_S is the coefficient of sliding friction. Byerlee (1978) proposed an average of $\mu_S = 0.85$, but at higher normal effective stresses ($\sigma_n > 200$ MPa), he found:

$$\tau \approx 100 + 0.65\, \sigma_n' \text{ (MPa).} \tag{18.18}$$

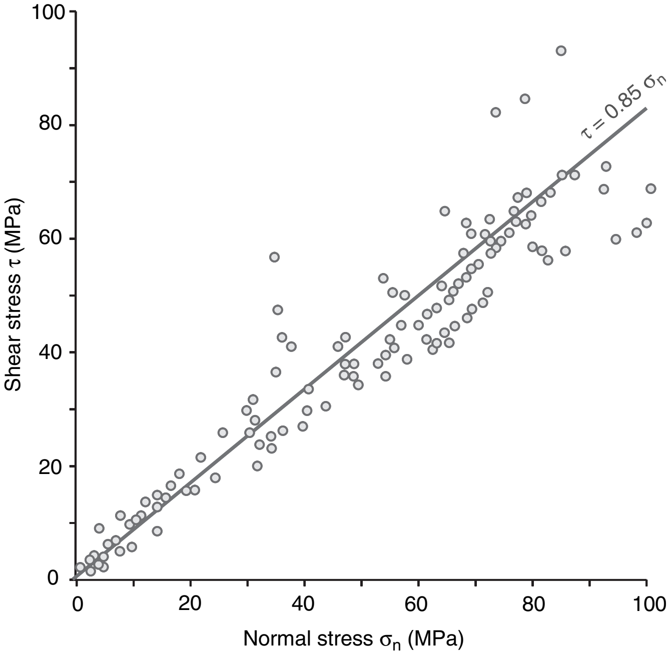

Figure 18.9 Data obtained by Byerlee (1978) for frictional sliding of planar rock surfaces at low normal stress

These results appear to be largely independent of rock type (at least for well consolidated and crystalline rocks). Based on Byerlee's results, it is widely assumed that cohesionless friction characterises many fractured rocks, with coefficients of sliding friction generally being between 0.65 and 0.85 – this has become known as 'Byerlee's Law'. The presence of weak material (e.g. clay fault gouge) in the fault plane can considerably reduce the coefficient of sliding friction. Water in the fractures mainly supports a pore pressure that controls the effective stress.

Since both shear and normal stress depend on the orientation of the surface in relation to the principal stress axes, failure by frictional sliding on rock fractures will be strongly controlled by the orientation of the fractures. If, however, sufficient variation in fracture orientation exists in a rock mass, then shear failure on optimally oriented fractures may be the dominant failure mechanism (e.g. Zhang and Sanderson, 2001).

18.12 Permeability

Rocks consist of solid phases (grains and cement) separated by voids. Flow of fluid between connected voids creates permeability in the porous medium. Flow is possible only through the connected void fraction, but for many rocks this is most of the void space. An exception is a rock such as pumice, which is a volcanic rock with isolated, unconnected gas bubbles; these make the rock light enough to float in water, but the rock is virtually impermeable – hence does not get 'waterlogged' and sink.

For most porous rocks, the flow is laminar (Reynolds number R_e<<1000) and the flow rate is linearly proportional to the pressure gradient. This was originally demonstrated by Henry Darcy in 1856, who showed that the flux (Q – m^3s^{-1}) of water in a pipe full of sand and subjected to a constant pressure drop (ΔP) or head (h) was proportional to its cross-sectional area (A) and inversely proportional to its length (L). Thus:

$$Q = -K_H \, A \, \Delta P / L. \tag{18.19}$$

The negative sign indicates that flow is from high to low pressure and the constant (K_H) is known as the hydraulic conductivity. K_H has SI units of ms^{-1} and depends on the nature of both the rock and the fluid.

$$K_H = k \, \rho \, g \, / \, \mu \tag{18.20}$$

where ρ and μ are the density and viscosity of the fluid, g is the gravitational acceleration and k the intrinsic permeability of the material. The intrinsic permeability has units m^2 with rocks typically having values in the range 10^{-18} to 10^{-10} m^2. The Darcy ($\approx 10^{-12}$ m^2) is often a more convenient unit as many rocks have permeabilities in the Darcy to milliDarcy range.

In civil engineering we are normally concerned only with groundwater, whose properties change relatively little. For water, at room temperature, $\rho g/\mu \approx 10^7$ in SI units; hence, $K_H \approx 10^7 k$. Again because of the low values K_H is often measured in m/day or m/year. Another common practice in civil engineering is to refer to K_H as a ‘coefficient of permeability’ (or even just ‘permeability’), a situation that can lead to confusion, and one that is both unnecessary and unhelpful.

A porous medium can be modelled as a system of pores, with porosity (n), connected by much finer tubes or throats of radius (r). Models can be used to relate the pore structure to the intrinsic permeability. For example, a simple model of a bundle of cylindrical tubes gives:

$$k \approx r^2 n / 8. \qquad (18.21)$$

Thus for $r = 10^{-5}$ m and $n = 0.1$ this yields an intrinsic permeability of ~1 Darcy, equivalent to a hydraulic conductivity of a little less than 1 m/day.

The size and distribution of rock components (grains, voids, cement and fractures) have a major influence on the hydraulic conductivity of rocks.

18.13 Fracture-controlled permeability

Many rocks have very low porosities, such as most igneous and metamorphic rocks, and many cemented limestones. Others have very small grain size (and hence pore-throat radius), such as claystones, and this results in very low intrinsic permeability. Small specimens when tested in the laboratory confirm this. On the other hand when tested in the field, many of these rocks have permeability in the milliDarcy to Darcy range that can be attributed to flow in fractures. Thus it is the rock mass, rather than grain-scale, properties that control flow.

Fracture flow can be modelled by laminar flow between two parallel plates (**Figure 18.10(a)**) where:

$$Q = -(Wh^3/12) \cdot (\rho g/\mu)\, \Delta P/L. \qquad (18.22)$$

This is the well-known ‘cubic flow law’. Since the cross-sectional area $A = Wh$, it follows that the intrinsic permeability of the fracture is $k = h^2/12$; this would be the permeability of a layer assigned to model the fracture.

For a set of fractures oriented parallel to the pressure gradient (**Figure 18.10(b)**), with aperture (h) and density (d = number per unit length), then from equation (18.22), it is clear that the intrinsic permeability of the rock mass as a whole is proportional to the fracture density (d) and the cube of the fracture aperture (h) and is

$$k = d\, h^3 / 12 \qquad (18.23)$$

since d has SI units m^{-1}, it follows that k has units m^2.

Fractures are an important component of the permeability of many rocks. A single fracture of aperture $h = 100$ microns (10^{-4} m) in a cubic metre, of rock (i.e. $d = 1\ m^{-1}$) would provide a permeability of ~10^{-13} m^2 (or 100 milliDarcys), from equation (18.23). This is equivalent to the permeability of many sandstones used as oil or water reservoirs and would represent a hydraulic conductivity $K_H \approx 1$ m/day.

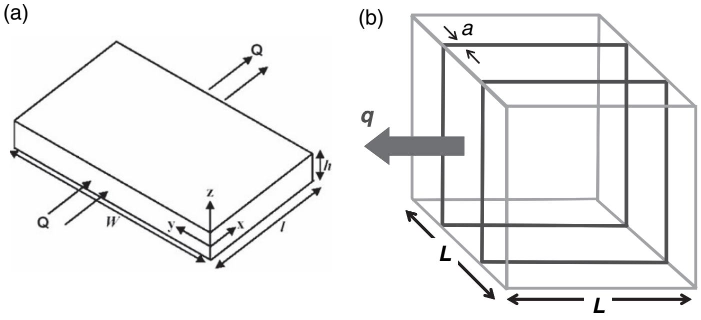

Figure 18.10 Flow of fluid through set of parallel fractures

18.14 Rock mass characterisation

Rock mass classification schemes have been around for over a century. They have generally been developed for assessment of behaviour in different engineering environments (tunneling, slope stability, etc.). Early schemes were simple and qualitative, e.g. Terzaghi (1946); more recent schemes use standardised measures and an algorithmic approach that can be implemented using spreadsheets.

Terzaghi’s descriptions (simplified from his original paper) are:

- *Intact* rock contains no discontinuities and, hence, breaks across the rock matrix, either across grains or along grain boundaries.
- *Stratified* rock consists of layers separated by boundaries that may represent discontinuities across which the rock may have less resistance to separation and/or shear.
- *Moderately jointed* rock contains a network of joints that generally separate the rock into blocks, which interact along a large proportion of their surface and may be intimately interlocked.
- *Blocky and seamy* rock consists of grains and other rock fragments that are imperfectly interlocked and interact through small areas of contact.
- *Crushed* rock consists of small fragments and grains with no re-cementation that generally are surrounded by voids and interact at point contacts. The voids may be saturated with fluid.
- *Squeezing* rock flows into large voids (tunnel and other excavations) without perceptible volume increase. Such rocks typically have microscopic particles of mica or clay minerals with a low swelling capacity.
- *Swelling* rock flows into large voids on account of expansion. Such rocks contain clay minerals such as montmorillonite, with a high swelling capacity.

This scheme is entirely qualitative, but has the advantage of focusing attention on the main characteristic of the mass behaviour. It is rarely used in modern engineering design.

Rock quality designation (RQD) is a single parameter used widely to describe the degree of rock fracturing in borehole core and was developed by Deere *et al.* (1967). RQD is simply the percentage of intact core pieces longer than 0.1 m (4 inches) in the total length of core. The core should be at least

50 mm or 2 inches in diameter. RQD is dependent on the spacing and direction of fractures relative to the borehole axis, and should exclude any fractures induced by drilling or handling of the core.

RQD clearly depends on spacing of fractures – the inverse of the number of fractures per unit length (λ). For a negative exponential distribution of fracture, i.e. one obtained from random placement of fractures, Priest and Hudson (1976) showed that:

$$\mathrm{RQD} = 100\, e^{-\lambda t}\, (1 + \lambda t) \qquad (18.22)$$

where t is the threshold length used to determine RQD. Using the conventional value $t = 0.1$ m, RQD varies between 5% and 95% in a range of 3 to 50 fractures/m and thus is insensitive to variations outside this range.

Rock mass rating (RMR) was designed for the estimation of rock strength (Bieniawski, 1973, 1989) and it uses six quantitative measures to classify a rock mass:

1. Uniaxial compressive strength of rock material
2. Rock quality designation (RQD)
3. Spacing of discontinuities
4. Condition of discontinuities
5. Groundwater conditions
6. Orientation of discontinuities.

Values of each of these measures are used to define ‘ratings’. For measures 1–5 in the above list these ratings are numbers usually between 0 and 15–30, which sum to a total between 0 (good rock) and 100 (poor rock). The final factor (6) is expressed as a negative rating reflecting the favourable (low –ve value) or unfavourable (high –ve value) of the orientation of fractures to the structure (tunnel wall, slope, etc.). The magnitude of the –ve value also varies with the type of excavation, being greater for slopes than tunnels.

Factors 2 and 3 above are closely related, and together account for 40% of the RMR, indicating that the ‘degree of fracturing’ is a key factor in this rock mass classification. RMR also incorporates the condition of the fractures (alteration and water content as well as the strength of the rock matrix.

The classification system is generally applied to different regions within the rock mass, usually based on the distribution of different rock types and major discontinuities.

18.15 Rock tunnelling quality index, *Q*

This was developed by the Norwegian Geotechnical Institute for underground excavations (Barton *et al*., 1974) and is based on six quantitative measures:

1. RQD (as defined above)
2. Jn – the number of joint sets
3. Jr – the joint roughness number
4. Ja – the joint alteration number
5. Jw – the joint water reduction factor
6. SRF – a stress reduction factor.

These parameters are used to define three ratios:

1. RQD/Jn represents the structure of the rock mass. As RQD is assessed relative to a 10 cm length of intact core, the ratio is crudely related to the size of fracture blocks.
2. Jr/Ja is a ratio that accounts for the roughness and frictional characteristics of the fracture walls and/or filling materials.
3. Jw/SRF is a ratio that consists of two stress parameters and attempts to describe the *in situ* loading conditions within the rock mass. Jw is a measure of water pressure, which reduces the effective normal stress, hence the shear strength of fractures. SRF is a measure of the total stress or load applied to the rock mass, but also incorporates effects such as loosening due to excavation and squeezing loads in plastic materials.

For a simple introduction to the Q system see Waltham (2009, pp. 86–87).

Both the RMR and Q schemes use geological and engineering parameters to provide a quantitative assessment of rock mass quality. Many of the parameters used are similar, but the calculations and weightings differ. Both schemes consider the degree of fracturing and the conditions of the fractures (groundwater, roughness, alteration, etc.). A significant difference is that RMR uses compressive strength directly, whereas Q only considers this in relation to *in situ* stress (through the SRF parameter). The RMR also incorporates the orientation of the fractures relative to the structure. Some estimate of orientation can be incorporated into Q using guidelines presented by Barton *et al*. (1974).

Both the RMR and Q schemes are best implemented using standardised procedures, which can be easily and consistently executed using spreadsheets, and details of the precise implementation of the methods are not given here.

18.16 Anisotropy

As well as the presence of discontinuities, there are often marked differences in rock properties with orientation within the rock mass. The major causes of such anisotropy are related to depositional layering in sedimentary rocks and flow fabrics in metamorphic and some igneous rocks; generally the latter have the greatest homogeneity. The anisotropy of rocks can be of three fundamentally different types:

1. Intrinsic anisotropy at the grain-scale, caused primarily by the preferred orientation of mineral grains. The mineral lattice properties may be strongly isotropic, hence alignment produces a macroscopic anisotropy in the rock sample. Alternatively the mineral grains may have a shape fabric and, hence, even for isotropic minerals, the alignment of

grain boundaries produces anisotropy. Preferred orientation of micro-cracks can produce similar anisotropy.

2. Many rock masses consist of layers of different rock materials and these produce a composite material, whose properties are not simply the average of the two (or more) materials involved. An example of this sort would be thin clay layers in sandstones, where the lower stiffness, strength and permeability of the clay may dominate the physical behaviour of the material. In such cases there is usually a strong directional dependence on the material properties.
3. The presence of fractures (see section 18.11).

Recognition of one or other of these types of anisotropy necessitates careful design of procedures to characterise the material – sampling different layers, testing large representative samples, testing in different orientations, etc. Conversely, one must be very careful in applying test results to construction in such anisotropic and/or composite materials.

For example, anisotropy is a major consideration in many claystones (shales). Stiffness is generally greater for compression normal to layering. Strength (e.g. UCS) is generally greater for compression normal to layering, but for shear failure in fissile shales the strength may be less when loaded in directions oblique to layering.

18.17 References

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.