5 Geotechnical parameters

'Putting numbers to geology'

Hoek (1999)

5.1 Physical properties of rocks and soils

For civil engineering design, it is necessary to assign physical properties to each unit of soil or rock within a ground model. These include readily measurable or estimated attributes such as unit weight, density and porosity. Other parameters that are often needed are strength, deformability and permeability. In the case of aggregates (rock used in construction for making concrete) and for armourstone, important attributes are durability and chemical stability.

5.2 Material vs. mass

Most tests and measurements are made on small-scale samples in the field or the laboratory and need to be scaled up according to theoretical or empirical rules, to include for geological variability, fabric and structure. For example, a soil mass might be made up of a mixture of strong boulders in a matrix of weak, soil-like material, and this mix has to be accounted for in assigning parameters for engineering design. Mass strength, deformability and permeability of rock masses are controlled largely by the fracture network, rather than intact rock properties; the permeability of intact rock might be 10^{-11} m/sec, which could be thousands of times lower than for the fractured rock mass.

5.3 Origins of properties

5.3.1 Fundamentals

The strength of soil and rock (geomaterials) is derived from friction between individual grains, from cohesion derived from cementation filling pore spaces and from inter-granular bonds such as those formed by pressure solution (Tada & Siever, 1989). The strength and deformability of soil is also a function of the closeness of packing of the mineral grains. Densely packed soil will be forced to dilate (open up) during shear at relatively low confining stresses as the grains override one another and deform, and the work done against dilation provides additional strength. The same principles apply to rough rock joints or fractured rock masses. Different minerals may also have fundamentally different properties – some are more chemically reactive and may form strong chemical bonds in the short term, some are readily crushed or scratched, whilst others are highly resistant to damage or chemical attack. Some, such as talc and chlorite, are decidedly slippery and if present on rock joints can result in instability.

The huge range of properties in soil and rock and how these evolve with time is illustrated by a single sample in Figure 5.1. The left-hand picture shows a graded series of sediments. The sand horizons become finer upwards, as is typical of sediments deposited from a river into a lake. At the top of the sample, there is a second sand horizon that has been deposited onto the underlying sediment. This has deformed the underlying sediments, producing a loading structure, which shows that the soil was in a very soft state at the time of formation. Contrast this with the rear of the same sample showing conchoidal fractures in what is actually extremely strong rock. The conversion from soft mud to rock has occurred over a long time but has occurred naturally and, in practical geotechnical engineering, we encounter and need to deal with the full range of materials, transitional between these end members.

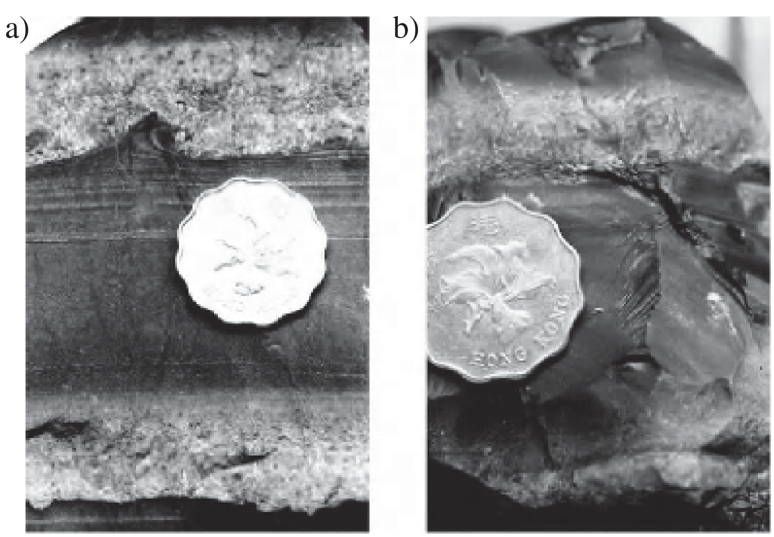


Figure 5.1
(a) Graded, probably seasonal bedding with clear evidence of soft sediment deformation. (b) Rear of the same sample with conchoidal fractures indicating the strength of this rock (probably of the order of 300 MPa).

5.3.2 Friction between minerals

Strength at actual contact points between grains of soil or rock is largely derived from electrochemical bonds over the true area of contact, which is only a very small proportion of the apparent cross-sectional area of a sample. At each contact between grains, elastic deformation, plastic flow and dissolution may take place, spreading the contact point so that the actual contact area is directly proportional to normal load. The attractive force over the true area of contact gives rise to frictional behaviour (Hardy & Hardy, 1919; Terzhagi, 1925; Bowden & Tabor, 1950, 1964). Bowden & Tabor, in particular, established that the area of asperity contact changed linearly with normal load for metals by measuring electrical resistance across the junctions. Power (1998) carried out similar tests using a graphite-based, rock-like model material (Power & Hencher, 1996).

The lower-bound friction angles for dry samples of quartz and calcite is reportedly about 6 degrees but higher when wet (Horn & Deere, 1962). The opposite behaviour was reported for mica and other sheet minerals. Perhaps linked to Horn & Deere's observations, mineral species that reportedly give higher friction values when wet are the same minerals that commonly form strong bonds during burial diagenesis through dissolution and authigenic cementation (Trurnit, 1968). It is possible that the presence of water allows asperity contacts to grow in these minerals, even in laboratory tests. Conversely mica, chlorite and clay minerals are rarely associated with pressure solution bonding and inhibit pressure solution and cementation of quartz (Heald & Larese, 1974). Some authors have questioned whether Horn & Deere's data are valid because of possible contamination and natural soil does not exhibit the same phenomena (Lambe & Whitman, 1979), but there is other evidence that basic friction of rock-forming minerals can be so low. Hencher (1976, 1977) used repeated tilt tests on steel-weighted, saw-cut samples of sandstone and slate to reduce the sliding angle from about 32 degrees to almost 12 degrees, which is approaching the low values of Horn and Deere. The reduction in strength was attributed to polishing (Figure 5.2).

5.3.3 Friction of natural soil and rock

Whilst basic friction the lower bound of minerals, originating from adhesion at asperities, might be of the order of 10 degrees or even lower, friction angles even for planar rock joints and non-dilational soil are often greater than 30 degrees yet the additional resistance (above basic) is still directly proportional to normal load. This additional frictional component varies with surface finish of planar rock joints and can be reduced by polishing (Coulson, 1971) or by reducing the angularity of sand (e.g. Santamarina & Cho, 2004). Figure 5.3 shows results from two series of direct shear tests on saw-cut and ground surfaces of granite. As

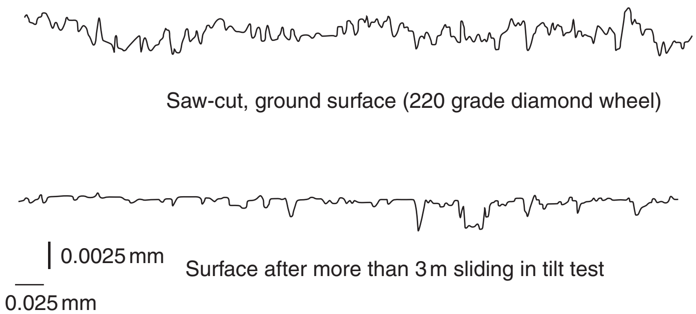


Figure 5.2 Ground and polished saw-cut surface of Delabole Slate at high magnification (top) and following repeated sliding tests (bottom).

shown in Figure 5.2, at a microscopic scale such ground and apparently flat surfaces are still rough. Each data point in Figure 5.3 is taken from a separate test with the sample reground beforehand. The upper line (inclined at 38 degrees) is the friction angle measured for moderately weathered (grade III) rock; the lower line inclined at 32.5 degrees is for slightly weathered (grade II) rock. The reason for the higher strength for the more weathered surface is because the surface finish is slightly rougher, the weathered feldspars being preferentially plucked from the surface during grinding. The key observation, however, is the precision of the frictional relationships – an increase in strength that is directly proportional to the level of normal load. Scholtz (1990) reviews the origin of rock friction and concludes that the additional strength is derived from deformation and damage to small-scale textural roughness. It is quite remarkable that this interlocking, non-dilational component still obeys Amonton's laws of friction.

The third contact phenomenon is dilation. Additional work is done against the confining normal load during shear as soil moves from a

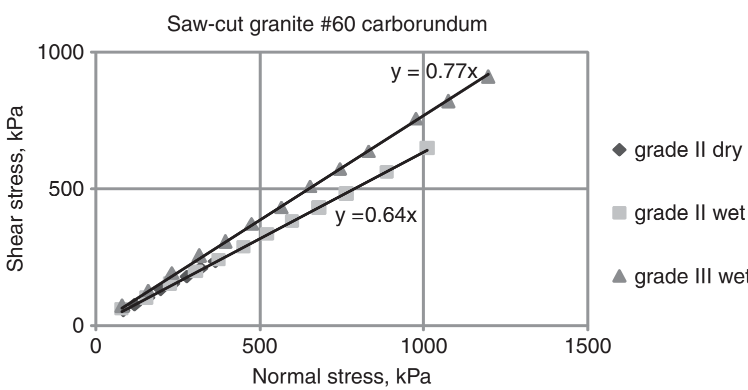


Figure 5.3 Perfect linear, frictional relationships between shear strength and normal stress for saw-cut and ground surfaces of rock. The upper line (stronger) is for moderately weathered granite, the lower for stronger, slightly decomposed rock. This paradox is explained by the fact that in the grade II rock the various mineral grains are of similar scratch resistance and therefore the surface takes a better polish during grinding than the more heterogeneous grade III rock.

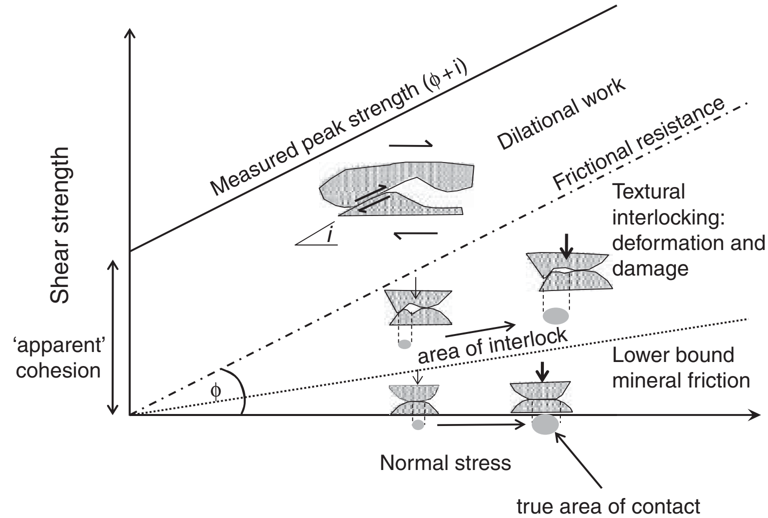


Figure 5.4
Measured strength envelope with apparent cohesion and friction, which can be corrected to a basic friction line (non-dilational).

dense to a less dense state or as a rock joint lifts over a roughness feature. If the raw strength data from a test are plotted against normal stress, then the peak strength envelope may show an intercept on the shear strength axis (apparent cohesion), albeit that the peak strength envelope may be very irregular, depending upon the variability of the samples tested. If corrections are made for the dilational work during the test, in many cases the strength envelope will be frictional: the strength envelope passes through the origin. At very high stresses, all dilation will be constrained and the soil or rock asperities will be sheared through without volume change. These concepts are illustrated schematically in Figure 5.4.

5.3.4 True cohesion

Rocks and natural soil may also exhibit true cohesion, due to cementation and chemical bonding of grains. For a rock joint, it is derived from intact rock bridges that need to be sheared through. This additional strength, evident as resistance to tension, is essentially independent of normal stress and proportional to sample size. This is discussed further below.

5.3.5 Geological factors

In Chapter 1 (Figure 1.5), the concept of a rock cycle was introduced whereby fresh rock deteriorates to soil through weathering and then sedimented soil is transformed again into rock through burial, compaction and cementation. Clearly, at each stage in this cycle the geomaterials will have distinct properties and modes of behaviour.

5.3.5.1 Weathering

In fresh igneous and metamorphic rocks, the interlocking mineral grains are linked by strong chemical bonds. As illustrated in Figure 5.5, there is almost no void space, although there may be some tiny fluid inclusions trapped within mineral grains. As weathering takes place close to the Earth's surface and fluids pass through the rock, it develops more voids as minerals decompose chemically and weathering products such as clay are washed out. The bonds between and within individual grains are weakened. Figure 5.6 illustrates how rock that starts off with a dry density of about 2.7 Mg/m^3 (typical of granite) becomes more and more porous so that by the completely

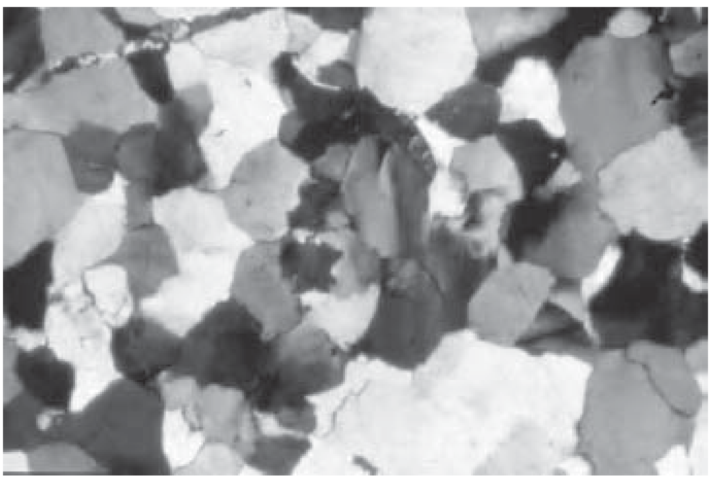


Figure 5.5 Thin section through granite, illustrating tightly interlocking fabric. Width of view approximately 20 mm.

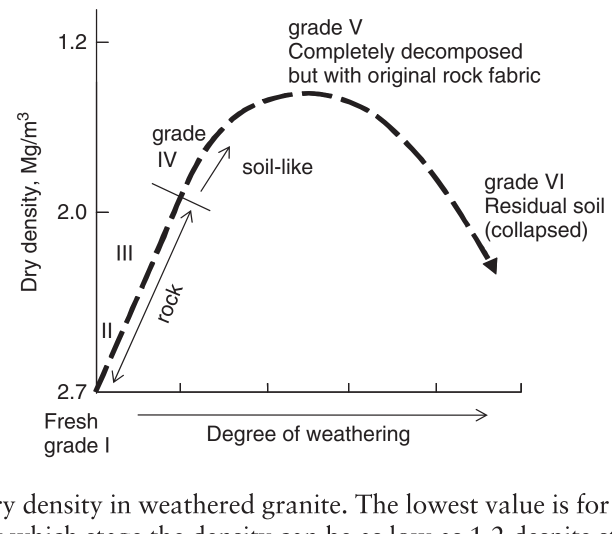


Figure 5.6 Change of dry density in weathered granite. The lowest value is for grade V, completely decomposed material, at which stage the density can be as low as 1.2 despite still having the appearance of granite (fresh state 2.7). At that stage, the material is prone to collapse to a denser, reworked, grade VI state. Based on Lumb (1962).

decomposed stage, the dry density may be reduced by more than 50% if weathering products have been washed out. The final stage is collapse to residual soil and an increase in density. Weathering is discussed in detail in Chapter 3.

Geotechnical properties at the material scale are linked quite closely to density empirically and, therefore, degree of deterioration from the rock's fresh state. Fresh granite might have a uniaxial compressive strength of perhaps 200 MPa but by the time the rock is highly decomposed the strength is reduced to 10–15 MPa and when completely decomposed perhaps 10–15 kPa. Where the rock is relatively strong, then properties and behaviour will be dominated by contained fractures; for most projects, the point at which material strength begins to dominate design decisions is where the rock can be broken by hand.

At the mass scale in weathered profiles, strength and deformation might be affected by the presence of strong corestones of less weathered rock in a weakened matrix, and the problem of characterisation is similar to that of mixed soils and rock such as boulder clay or boulder landslide colluvium, as discussed later.

Permeability in fractured rock or in weathered profiles can be extremely variable and difficult to predict, with localised channel flow providing high permeability. Elsewhere, accumulations of clay or general heterogeneity in the profile can prevent and divert water flow. The complexities of flow through weathered rock profiles and difficulties in measuring permeability are discussed in Chapters 3 and 4.

5.3.5.2 Diagenesis and lithification (formation of rock from soil)

As discussed in Chapter 3, soil is transported by water, wind or gravity from the parent rock. During the process of transportation, the sediment is sorted in size. Some soils such as glacial moraine and colluvium remain relatively unsorted. Sediments tend to be continually deposited over a very long period of time, for example, in river estuaries, and each layer of sediment overlies and buries the earlier sediment. The underlying sediment is compacted and water squeezed out. This is termed burial consolidation and is a very important process governing the strength and deformability of sediments. Grains become better packed, deformed and may form strong chemical bonds with interpenetration and sutured margins. Voids may be infilled with cement precipitated from soluble grains in the sediment (authigenic cement) or from solutions passing through the sediment pile, as illustrated in Figures 5.7 and 5.8. Many clay oozes initially have a very high percentage of voids, with the mineral grains arranged like a house of cards. With time, overburden stress and chemical changes cause the flaky minerals to align and the porosity (or void ratio) to decrease markedly, as illustrated in Figure 5.9. Burland (1990) has expressed the rate at which void ratio is reduced with burial depth as a normalised equation although there are

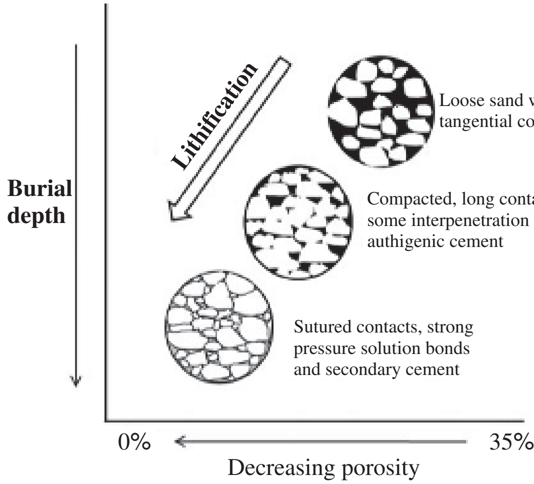


Figure 5.7 Compaction and cementation of granular soils with burial leading to increased strength, reduced deformability and lower permeability.

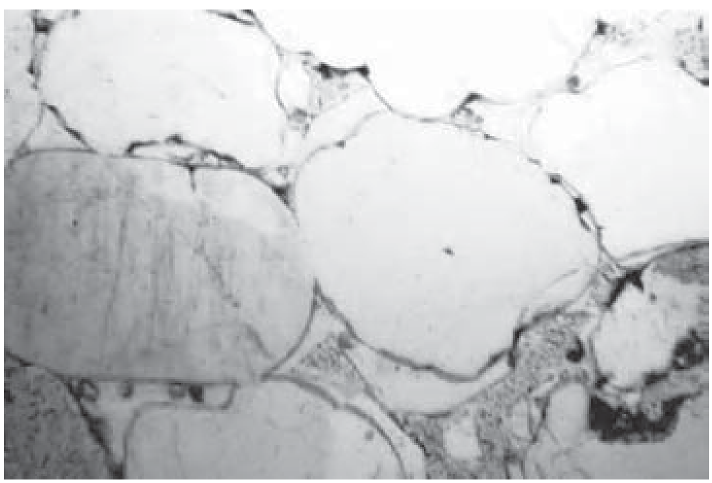


Figure 5.8 Thin section of aeolian sandstone with rounded grains of quartz, interpenetration of grains and flattened surfaces where in contact, with some pressure solution, plus authigenic cementation of grains by silica and iron oxides. As a result of these diagenetic processes, the material has been turned from loose sand into a strong rock. Triassic Sandstone, UK. Large grains about 5 mm in diameter.

often departures from this behaviour in natural sediment piles, due largely to cementation (Skempton, 1970; Hoshino, 1993). The changes in property (especially strength and deformability) that ensue from burial, compaction and consolidation are discussed in Section 5.5. At some locations, the upper part of the sediment pile is considerably stronger than might be anticipated from its shallow burial level because it has become desiccated on temporary exposure above water level. Where soils are uplifted and upper levels eroded, or otherwise loaded, and then that load removed (e.g. by the melting of a glacier), then the strength and stiffness will be relatively high and the soil is termed

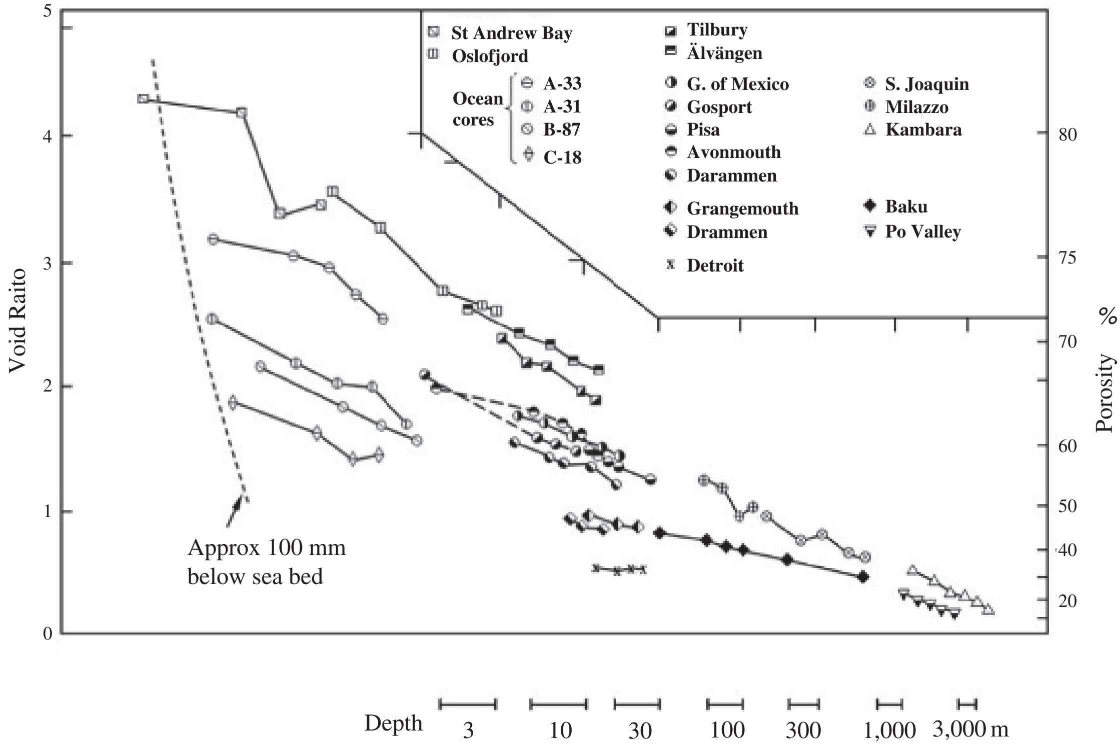


Figure 5.9 Compression curves for naturally consolidated and partially cemented clay (modified from Skempton, 1970).

overconsolidated. In the case of sand, the history of burial compaction can result in an extremely dense arrangement of the sand particles that cannot be replicated in the laboratory. Such locked sands, with grains exhibiting some interpenetration and authigenic overgrowths, not surprisingly, have high frictional resistance and dilate strongly under shear (Dusseault & Morgenstern, 1979).

5.3.5.3 Fractures

Natural fractures occur in most rocks close to the Earth's surface and in many soils once they begin to go through the processes of burial and lithification. Figure 5.10 shows a quarry face where discontinuities dominate mass geotechnical parameters such as deformability and permeability. Vertical joints in relatively young glacial till are shown in Figure 5.11. Fractures will often dominate fluid flow through the mass, as well as mass deformability and strength. They need special consideration and characterisation, as addressed in Chapters 3 and 4 and discussed later.

5.3.5.4 Soil and rock mixtures

Many soils such as glacial boulder clay and colluvium comprise a mixture of finer soil and large clasts of rock, and these need special

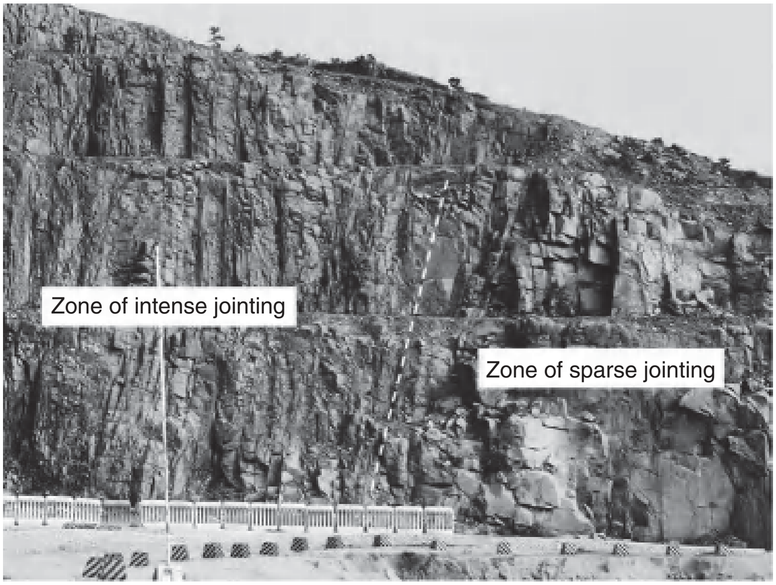
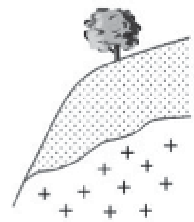
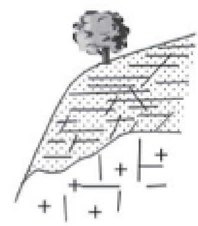
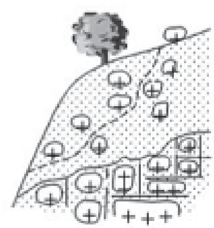
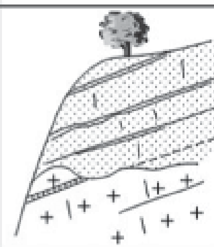


Figure 5.10 Predominantly vertical jointing (probably combined cooling and tectonic during emplacement) in granite. Mount Butler Quarry, Hong Kong.

Figure 5.11 Vertical joints developed in boulder clay. Robin Hood's Bay, North Yorkshire, UK.

Figure 5.12
Options for slope stability analysis. After Hencher & McNicholl, 1995.

Option	Schematic diagram	Approach for defining parameters and analysis
1. Treat as uniform (continuum)		• parameters from laboratory or *in situ* tests taken to be representative of zone
2. Treat as uniform but weakened by discontinuities (continuum)		• allowance made for influence (but not control) of discontinuities on mass properties (e.g. Hoek-Brown)
3. Treat as heterogeneous (continuum)		• consideration given to influence of strong inclusions with deviated failure paths
4. Treat as discontinuous due to structural control		• discontinuity controlled

consideration in terms of their properties. Weathered rocks can similarly comprise mixes of weak and hard materials but there is also the added complication of relict rock fabric and structure. The overall nature of the mass will strongly affect the options for engineering assessment, as illustrated for slopes in Figure 5.12. Geotechnical parameter determination for such mixed deposits is considered in Section 5.8.

5.4 Measurement methods

Methods of testing soil and rock are specified in standards such as BS 1377 for soil in the UK (BSI, 1990), BS 5930 for several field tests (BSI, 1999) and ASTM, more generally in the USA. The International

Society for Rock Mechanics provides guidance on many field and laboratory tests (Ulusay & Hudson, 2006). Recommendations for the same test sometimes differ, for example regarding sample dimensions and testing rate, so care has to be taken that an appropriate method is being adopted and referenced. Furthermore several different techniques or different equipment can sometimes be used ostensibly to measure the same parameters but inevitably with different results. For example, small strain dynamic tests may give very different values for soil stiffness compared with large-scale loading tests but each might be appropriate to some aspect of numerical analysis and design within a single project (Clayton, 2011). It should also be remembered that, however much they are standardised, all tests on soil and rock are experiments. There will be many variables, not least the geological nature and moisture content of the sample to be tested, so interpretation is always necessary. Further judgement is required before attempting to apply small-scale results at the larger scale (e.g. Cunha, 1990).

5.4.1 Compressive strength

Intact rock, clay and concrete are generally classified in shorthand by their unconfined (or uniaxial) compressive strength (UCS) as discussed in Chapter 4. Compressive strength is not a relevant concept for purely frictional materials such as sand, which must be confined to develop shear resistance. Indicative UCS values for various materials are presented in Table 5.1; fresh rock is often considerably stronger than the highest strength concrete. For concrete, UCS is used as a quality assurance test on construction sites.

In a UCS test the axial stress is σ_1 and the confining stresses (σ_2 and σ_3) are zero. Despite the apparent loading condition, the sample does not actually fail in compression but either in tension or in shear or in some hybrid mode. If the sample contains adverse weak fabric such as incipient joints or cleavage, then the sample will fail at lower strength than it would without the flaws. UCS is really essentially an index test used especially in rock mass classification. In practice strength can often be estimated quite adequately using index tests such as hitting with a geological hammer (see Box 5-1). UCS can also be measured using point load testing, which is quick and easy, but correlation with UCS from laboratory testing may be imprecise. The Schmidt hammer is sometimes used to estimate strength using standard impact energy to measure rebound from a rock or concrete surface. It is sensitive to surface finish and any fractures behind the impact location will cause low readings. It is also insensitive to strength over about 100 MPa. It is generally unsuitable for testing rock core – its main use in engineering geology is as an index test to help differentiate between different degrees of weathering as discussed in Chapter 4.

Table 5.1 Indicative unconfined compressive strengths for some rock, soil and concrete.

Material	Uniaxial Compressive Strength, UCS MPa	
Natural rock and soil		
Fine-grained, fresh igneous rock such as dolerite, basalt or welded tuff, crystalline limestone	>300	Rings when hit with geological hammer
Grade I to II, fresh to slightly weathered granite	100–200	Difficult to break with hammer
Cemented sandstone (such as Millstone Grit)	40–70	Broken with hammer
Grade III, moderately weathered granite	20–40	Broken with hammer
Chalk Grade IV highly weathered granite	5–30	Readily broken with geological hammer Weaker material broken by hand
Overconsolidated clay	0.6–1.0	Difficult to excavate with hand pick
Very stiff clay-rich soil	0.3–0.6	Indented with finger nail
Concrete		
High-strength concrete (e.g. Channel Tunnel liner)	50–100	
Typical structural concrete	30–50	
Shotcrete in tunnel	20–40	

Box 5-1 To test or not to test?

Many ground investigations are wasteful in that they do not target or identify critical geological features, and laboratory tests are commissioned without real consideration of whether or not they will be useful.

Example 1

Figure B5-1.1 shows the formation level (foundations) for the Queen's Valley Dam, Jersey, which was completed in 1991. The dam was to be an earth dam, which exerts relatively low stresses on its foundations, compared to a concrete dam such as an arch or gravity dam. With a maximum height of 24 m and an assumed unit weight of 20 kN/m^3, the bearing pressure might be of the order of 500 kPa. The author, who was mapping the foundations, was asked to select samples of core to be sent to the laboratory for uniaxial compressive strength testing.

Rock over much of the foundation was rhyolite that was extremely difficult to break by geological hammer and had an estimated compressive strength of more than 300 MPa. The rhyolite, however, contained numerous incipient fractures (Figure B5-1.2), which would mean that the mass strength was somewhat lower and, more significantly, would cause samples to fail prematurely in the laboratory. The author argued that if the samples were sent to the laboratory, the reported result would simply be scattered with a range from 0 to 300 MPa and what would that tell us that we didn't already know? The allowable bearing pressure for rock of this quality (Chapters 6) would be at least five times the bearing pressure exerted by the dam. In the event, the samples were still sent off to the laboratory for testing (because they had already been scheduled by the design engineers) and the money was duly wasted.

Figure B5-1.1 View of left abutment of Queen's Valley Dam, Jersey, UK, under construction.

Figure B5-1.2 Extremely strong rhyolite. Hammer and clipboard for scale.

Example 2

The Simsima Limestone is the main founding stratum in Doha, Qatar, and is found extensively across the Middle East. It is a highly heterogeneous stratum including calcarenite, dolomite and breccia. The rock is often vuggy and re-cemented with calcite. RQD can be very high, with sticks of core a metre or more in length without a fracture; elsewhere the RQD is zero. An example is shown in Figure B5-1.3.

The properties of the stratum are clearly important for design of foundations and for other projects such as dredging, as discussed in Chapter 3. UCS test data tend to be very scattered, in part because the integral flaws in many samples lead to early failure. If a strongly indurated sample with few flaws is tested,

Figure B5-1.3 Example of core through Simsima Limestone (courtesy of Karim Khalaf, Fugro, Middle East).

then it can give UCS strength of 60 or 70 MPa (higher than structural concrete). Samples of inherently weaker material (as could be estimated from scratchtesting) or containing vugs or other flaws, will fail at much lower strengths. A typical range of data is given in Figure B5-1.4. If smaller intact pieces of dolomitised limestone are point load tested selectively, they will, of course, err towards the higher strength of the rock mass. As a consequence, conversion factors from point load test to UCS for this rock are usually taken empirically as 8 to 9 (Khalaf, personal communication). Data converted in this way are included in Figure B5-1.4. For more uniform rocks elsewhere in the world, conversion factors of about 22 are more commonly applied (Brook, 1993). If such a factor were to be used for the Simsima Limestone, then it would imply strength for the intact limestone, without flaws, up to about 200 MPa.

Given this very wide range of possible strengths, it would seem unwise simply to rely on a statistical testing campaign for characterising the rock mass. Far better to try first to characterise the rock geologically into units based on the strength of rock materials and then mass characteristics including flaws, degree of cementation and degree of fracturing. In this case, index tests (hammer, knife), combined with visual logging and selective testing of typical facies, are likely to give a far better indication of mass properties than UCS testing alone. To obtain parameters for the large scale (say foundations) then *in situ* tests such as plate loading and perhaps seismic tests would help, as would full-scale instrumented pile testing. Where rock mass strength is very important, as for the selection of dredging equipment, then it would be very unwise to take UCS data at face value (as a statistical distribution). As for many tests, there are numerous reasons why values measured in the laboratory might be unrepresentative of conditions *in situ*, often too low, and considerable judgement is required if the parameters are critically important.

Lesson: compressive strength of most rocks can often be estimated adequately by hitting with a hammer and the use of other index tests; if a hard blow by a hammer cannot break the material, then its strength probably exceeds that of any concrete structure to be built upon it. Where strength is critical, as in the selection of a tunnelling machine or choice of dredging equipment, then any test data must be examined

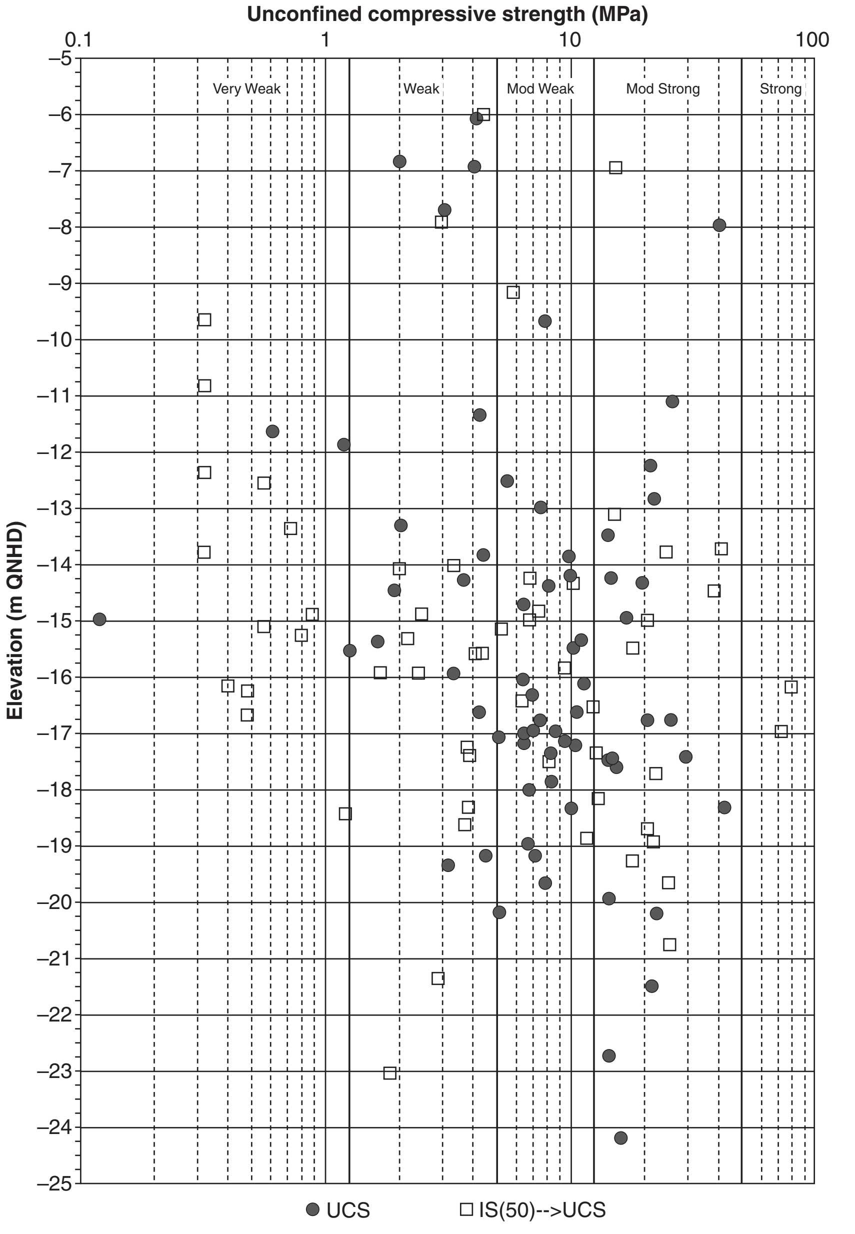


Figure B5-1.4 Typical UCS data from Simsima Limestone (courtesy of Karim Khalaf, Fugro, Middle East).

critically. If laboratory test samples contain flaws such as discontinuities, then measured intact strength may be too low. Of course, at the mass scale, the flaws and joints will be extremely important but their contribution cannot be properly assessed by their random occurrence and influence on laboratory test results.

5.4.2 Tensile strength

Although rocks actually usually fail in tension rather than compression, tensile strength is rarely measured directly or used in analysis or design, compressive strength being the preferred parameter for rock mass classifications and empirical strength criteria (see later). Tensile strength of rock and concrete is relatively low, typically about 1/10th of UCS. It is because of the weakness of concrete in tension that reinforcing steel needs to be used wherever tensile stresses are anticipated within an engineering structure.

5.4.3 Shear strength

Shear strength is a very important consideration for many geotechnical problems, most obviously in landslides where a volume of soil or rock shears on a slip surface out of a hillside. It is also important for the design of foundations and in tunnelling (Chapter 6). There are two main types of test used to measure shear strength in the laboratory – direct shear and triaxial testing. There are also many other *in situ* tests used to measure shear strength parameters, either directly (e.g. vane test) or indirectly (e.g. SPT and static cone penetrometer tests), and these have been introduced in Chapter 4.

For persistent (continuous) rock discontinuities, direct shear testing is the most appropriate way of measuring shear strength. Details of testing and interpretation are given in Hencher & Richards (1989) and Hencher (1995). Because of the inherently variable roughness of different natural samples, dilation needs to be measured and results normalised, as discussed later. If this is not done then, in the author's opinion, the tests are usually a total waste of time. The details of a shear box capable of testing rock discontinuities and weak rocks with controlled pore pressures is described by Barla *et al.* (2007).

Direct shear tests are also carried out on soil and are much easier to prepare and conduct than tests on rock discontinuities, although the stress conditions are not fully defined in the test, which can cause some difficulties in interpretation (Atkinson, 2007). This is one reason why triaxial testing is preferred for most testing of soils. Other advantages are that factors like drainage and pore pressure measurement can be carefully controlled. A disadvantage is that the soil may well become disturbed during trimming and preparation for the test as well as during back saturation and loading/unloading, but that is a problem for all testing. In a triaxial test, the cylindrical sample is placed inside a cell and then an all-around fluid pressure applied (σ_3). This remains the constant minimum principal stress throughout the test. Some tests are carried out drained, in that water is allowed to seep out of the sample as it is compressed; in others drainage is prevented, the water pressure changes as the sample is loaded and can be measured. In some tests the sample is initially loaded and consolidated to a required effective stress in an attempt to simulate the field condition. Once the sample is in equilibrium, it is gradually compressed axially whilst the confining stress remains constant. The process is illustrated graphically using Mohr stress circles in Figure 5.13. Note that within the sample, the angle between σ_1 and σ_3 is 90 degrees, but in the Mohr circle presentation, this stress field is expressed as a hemisphere (180 degrees). The hemisphere represents the stress state on any plane drawn through the sample. The test proceeds from the state where $\sigma_1 = \sigma_3$, then σ_1 is increased (hemispheres grow towards the right) until the sample eventually fails. Normal stress on any plane through the sample is measured on the horizontal axis, shear stress on the vertical axis. The stress normal to a vertical plane through the sample is σ_3 and the shear stress is zero; the normal stress on a horizontal plane through the sample is σ_1, the shear stress zero. These planes are known as principal planes. For a plane inclined at 10 degrees (shown as 20 degrees graphically within the Mohr circle) the normal stress on that plane is σ_{10} and at 45 degrees it is σ_{45}, with the corresponding shear stress (τ), as indicated. At failure, the shear plane through the sample will be developed at some angle (θ/2 degrees) to the horizontal, expressed as θ in the Mohr circle graph. The Mohr stress circle representing the stress state at that stage is shown in Figure 5.14 for a single test. Further tests would be carried out on other similar samples at different confining stresses and used to define a strength envelope (a line joining the stress states at which all samples failed). Usually the envelope for a set of samples can be defined in terms of friction (gradient of line) and apparent cohesion, c, which is the intercept on the shear stress axis at zero normal stress (Figure 5.4).

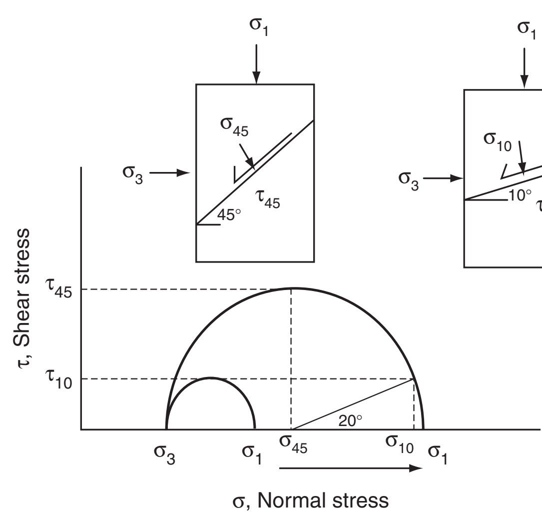


Figure 5.13 General representation of stress conditions in an individual sample.

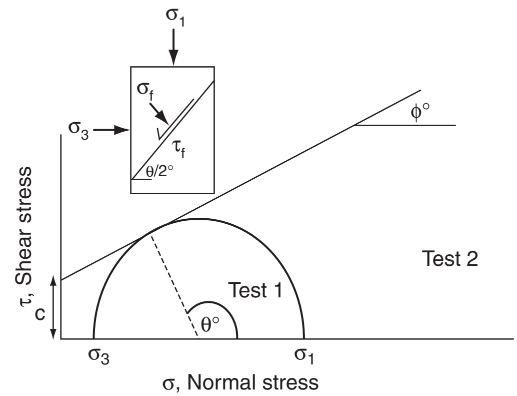


Figure 5.14
Mohr circle at shear failure.

5.4.3.1 True cohesion

The nature, origin and even existence of cohesion – strength at zero normal load – causes considerable debate and confusion. This is partly because it can be either apparent (the result of dilation during a test and varying with confining stress) or a real physical entity and due to cementation, grain bonding or impersistence of discontinuities in the rock mass. Quite often both factors contribute to the measured strength in the same test, for example, if shearing intact rock. In artificially prepared samples of remoulded soil there is no true cohesion and apparent cohesion is a function of the density of packing of the soil grains relative to the confining stress. A theory of critical state soil mechanics has been developed for such soil that links shear strength to deformation characteristics (Roscoe *et al.*, 1958; Schofield, 2006). Burland (2008) however notes the importance of geological history to natural soils, with the development of bonding and fabric leading to true cohesional, non-dilational and stress-independent strength. While Burland was really discussing relatively young soils, it has been demonstrated earlier (Figure 5.1) how, with time, true cohesion can become very high and far outweigh the contribution of friction to shear strength. Conversely, as rock is gradually weathered it is primarily the cohesional strength that is lost – friction remains essentially constant.

5.4.3.2 Residual strength

After high shear displacement, cohesion is lost, and shearing continues at a residual friction level. This is non-dilational friction but in nature can be lower than the critical state – also non-dilational – because of change in structure with, for example, flattening and alignment of particles in a clay or the development of highly polished shear surfaces. Such strengths can be very low (sometimes of the order of 7 degrees for montmorillonite clay-rich rocks) and very significant, especially for landslides (see discussion of Carsington Dam failure in Chapter 7). To test residual strength, torsional ring shear boxes are used, in which an annulus-shaped sample is prepared and then rotated until a constant low strength is obtained.

5.4.4 Deformability

Young's Modulus (E) is expressed as stress/strain (with units of stress) and is a key parameter for predicting settlement of a structure or deformation in a tunnel and needs to be defined at a mass scale. For soil, samples are consolidated in oedometers and measurements taken of deformation against time. The main derived parameters are m_v, which is an inversion of E, i.e. strain/stress, and C_c, which is a measure of rate of consolidation. For normally consolidated clay that has been simply buried by overlying sediment, there will be a gradual improvement in strength and stiffness with depth, as illustrated for natural soils in Figure 5.9. Soil that has been overconsolidated because of its geological history will exhibit relatively high stiffness up to the loading level corresponding to its earlier pre-consolidation stress state. Once that pressure is exceeded, the stiffness will revert to the natural consolidation curve. At very small strains, overconsolidated clay can be much stiffer than at higher strain levels, and this can be important for realistic modelling of excavations (Jardine *et al.*, 1984; Clayton, 2011). Geophysical testing can be used to interpret stiffness parameters from velocities of wave propagation through soil, and values are again on the high side compared to static tests at relatively high strains (Mathews *et al.*, 2000). The same is true of rock masses – interpretation of compressional or shear velocities tend to give higher stiffness values than do static loading tests, and this probably reflects the low strain nature of loading from transient dynamic waves (Ambraseys & Hendron, 1968). Because of the difficulties in determining E at the rock mass scale from first principles or testing, it is common to rely on empirical published data as discussed at 5.6.3.

5.4.5 Permeability

Permeability is an intrinsic parameter of soil and rock, relating to rates of fluid flow through the material and strictly varies according to the fluid concerned – e.g. oil, water or gas. It has dimensions of area (L^2). In hydrogeology and geotechnical engineering, the term permeability is generally used interchangeably with hydraulic conductivity and is the volume of water (m^3) passing through a unit area (m^2) under unit hydraulic gradient (1 m head over 1 m length) in a unit of time (per second). This reduces to m/s. For low permeability rock suitable for a nuclear waste repository, the permeability, k, might be 10^{-11} m/s. For an aquifer of sandstone suitable for water extraction, it might be 10^{-6} m/s and for clean gravel 10^{-1} m/s. Typical values for other soils are given in BS 8004 (BSI, 1986).

In some soil such as alluvial sand, the material permeability could be similar to that of the mass, so laboratory testing might be relevant, but for many ground profiles water flow will be localised and involve natural pipes, fissures and open joints or faults. Field tests are then generally necessary to measure mass-scale permeability, as outlined in Chapter 4. Large-scale pumping tests from wells with observational boreholes at various distances can give reliable parameters for aquifer behaviour but localised testing in boreholes, as specified in BS 5930 (BSI, 1999), can be unreliable (Black, 2010). As discussed in Chapters 3 and 6 and illustrated in examples in Chapter 7, localised geological features often control fluid flow through the soil or rock mass, so testing must be linked to relevant geological and hydrogeological models.

5.5 Soil properties

5.5.1 Clay soils

As Skempton (1970) showed (Figure 5.9), for clay soil deposited offshore at rates of perhaps 2m per thousand years, consolidation behaviour due to self-weight is fairly well defined. As the porosity diminishes and water is squeezed out, so strength increases and deformability reduces, even in the absence of other diagenetic processes. Hawkins *et al.* (1989), for example, show a consistent linear increase in shear strength with depth over 20m at a test site in Bothkennar, Scotland, based on vane tests. Cone test data from the same site are very similar to other sites in the UK, confirming the trend. Similar results have been achieved from other sites worldwide, with a typical relationship:

$$s_u = 10 + 2.0d$$

Where s_u = undrained shear strength, kPa and d = depth below ground, m.

Elsewhere, values can be somewhat lower; for the Busan Clay in Korea, the gradient is closer to 1.0 times depth (Chung *et al.*, 2007). Nevertheless the trend is similar so for design in soft to firm clay it is usual practice to carry out a series of vane tests down boreholes or cone penetrometer soundings, and then try to define a relationship of increasing strength with depth that can easily be input to numerical simulations. Relationships are published both for shear strength and modulus of clay interpreted from SPT tests, and these are reviewed in Clayton (1995) although the SPT is less appropriate for clay than for granular soils. Most of the values obtained from field tests are necessarily undrained and expressed as a value of apparent cohesion with no frictional component. Undrained shear strength of clay can also be obtained from undrained tests in the laboratory and is estimated during field description using index tests like resistance to finger pressure or in a rather more controlled way using a hand penetrometer, as discussed in Chapter 4. Undrained strength is useful for assessing the fundamental behaviour of clay empirically, for example, in designing foundations (Table 6.1). It is also used for numerical analysis in soils of low permeability immediately after or during construction. Conversely drained conditions apply where excess pore pressures have dissipated following construction or where they dissipate relatively rapidly during construction. For design of structures in clay under drained conditions, effective stress parameters are required – friction and possibly some cohesion where there has been some geological bonding. These parameters are generally obtained from triaxial testing, in which pore pressures are monitored and corrected for throughout the test (e.g. Craig, 1992). Effective stress parameters can also be interpreted from *in situ* piezocone penetrometer soundings (Chapter 4).

Laboratory tests are relied upon for characterising natural clay far more than for any other soils, because reasonably undisturbed samples can be taken and the small grain size relative to testing apparatus means that scale effects are not evident. An exception is in settlement analysis, where it is found that standard oedometer tests give lower stiffness than larger-scale plate load tests or are evident from back analysis of the construction of a structure. Specialised testing is necessary to simulate low strain deformation (e.g. Atkinson, 2000).

As noted earlier, for some active and ancient landslides, the strength along the slip plane through clay/mudstone is reduced below the critical state friction angle to a residual friction angle well below 20 degrees, even for clay of relatively low plasticity such as kaolinite or illite (Skempton *et al.*, 1989). Such low values can be measured in the laboratory using ring shear boxes and back-analysed from landslide case histories.

Clays include some groups of very problematical soils. Quick clays are clay and silt size but mostly detrital materials (rock flour produced by glacial scour), weakly cemented by salt, which can become disturbed and then flow, sometimes to disastrous effect. The Rissa, Norway, landslide in 1978 was filmed, flowing rapidly across flat ground, indicating the sensitivity of such materials. Other clays such as black cotton soils swell and shrink dramatically with changes in moisture, which causes damage to roads and other structures. The clay mineral group smectite (montmorillonite/bentonite) is most commonly associated with volume change and is typically identified by X-ray testing. Its presence is also indicated from high liquid limits and high plasticity indices in Atterberg limit tests (Chapter 3). These clay minerals can have very low shear strengths. Starr *et al.* (2010) describe a creeping major rock slope failure where the rock is smectite-rich and for which the operating residual friction angle was only about 7 degrees as established by numerical back-analysis and confirmed from laboratory tests.

5.5.2 Granular soil

The behaviour of granular soil such as silt, sand and gravel can be examined in the laboratory but for design, geotechnical parameters are generally determined by *in situ* testing, because of the difficulties of a) obtaining and transporting undisturbed samples and b) the problems of scale effects in testing samples of large grain size.

The most common test for characterising silt, sand and gravel is the SPT, as discussed in Chapter 4. Measured resistance needs to be corrected for various influences, including overburden pressure and the silt content of sand. Resistance may be affected by water softening in the base of a borehole. Details are given in Clayton (1995). SPT N-values are used to infer a range of properties, including density (unit weight), friction angle and deformability which are then used for the design of many types of structure, including foundations, retaining walls and slopes. CPT tests can also be used in this way and are particularly useful for the design of offshore structures.

5.5.3 Soil mass properties

Usually, properties of intact soils of sedimentary origin are assumed to be representative of the larger soil mass layer or unit. This can be an over-simplification in that even quite recent soils can contain fractures and systematic joints and many are layered with different layers having different properties. In the latter case, permeability parallel to bedding might be orders of magnitude higher than at right angles to bedding, and there are many geotechnical situations where such a condition would be important. McGown *et al*. (1980) discusses origins of fractures in soil and how they might be dealt with when assessing geotechnical properties. London Clay, for example, contains many fissures that can be interpreted using structural geological techniques (Fookes & Parrish, 1969). Chandler (2000) describes the significance of bedding parallel flexural-slip surfaces extending at least 300 m in London Clay. Similar features are discussed by Hutchinson (2001).

5.6 Rock properties

5.6.1 Intact rock

5.6.1.1 Fresh to moderately weathered rock

Fresh to moderately weathered rock, by the definitions adopted here (Chapter 3 and Appendix C), cannot be broken by hand at the intact sample scale, as in a piece of core. That being so, it has an unconfined compressive strength of at least 12.5 MPa and is definitely rock-like in that it could carry most structures without failing (presumed bearing capacity of at least 1 MPa according to Table 6.1) and will not fail in a man-made slope, in the absence of discontinuities, almost irrespective of height and steepness.

The strength of fresh rock is a function of its mineralogy, internal structure of those minerals (cleavage), grain size, shape and degree of interlocking, strength of mineral bonds, degree of cementation and porosity. Some rocks have intact strength approaching 400 MPa – these might include quartzite, welded tuffs and fine- and medium-grained igneous rocks such as basalt and dolerite. Corresponding intact moduli can be as high as 1×10^6 MPa (1×10^3 GPa) (Deere, 1968).

Compressive strength is measured most accurately using very stiff servo-controlled loading frames, whereby, as the rock begins to fail, so the loading is paused to limit the chance of explosive brittle failure. Such test set-ups allow the full failure path to be explored, which can be important in underground mine pillars where, despite initial failure in one pillar, there may be sufficient remnant strength after load is transferred to adjacent pillars, so that overall failure of the mine level does not occur. For most civil engineering works, UCS values measured by less sophisticated set-ups are adequate. Nevertheless, the specification for UCS testing is onerous, particularly regarding test dimensions and flatness of the ends of samples. If these requirements are not followed, local stress concentrations can cause early failure. If samples are too short, then shear failure might be inhibited. As noted in Box 5-1, there are alternative ways of estimating UCS that might be adequate for the task at hand.

UCS is the starting point for many different empirical assessments of rock masses, including excavatability by machinery such as tunnel boring machines. Other parameters that might need to be quantified include abrasivity and durability. Appropriate tests are specified in the ISRM series of recommended methods (Ulusay & Hudson, 2006).

Intact rock modulus is rarely measured for projects and is not usually an important parameter for design. An exception is in numerical modelling of fractured rock mass, e.g. using UDEC (Itasca), where this parameter is required, but for this purpose, values are typically estimated from published charts or even selected to allow the model to come to a solution within a reasonable time. Models tend to be insensitive to the chosen parameter.

5.6.1.2 Weathered rock

Intact weathered rock has true cohesion from relict mineral bonding. In some cases there may be secondary cementation, especially from iron oxides and the redistribution of weathering products within the rock framework. At the strong end of grade IV where it can just be broken by hand UCS might be about 12.5 MPa and cohesion of about 3 MPa might then be anticipated (Hencher, 2006). In practice, such high values have never been reported. Ebuk, who tested a range of

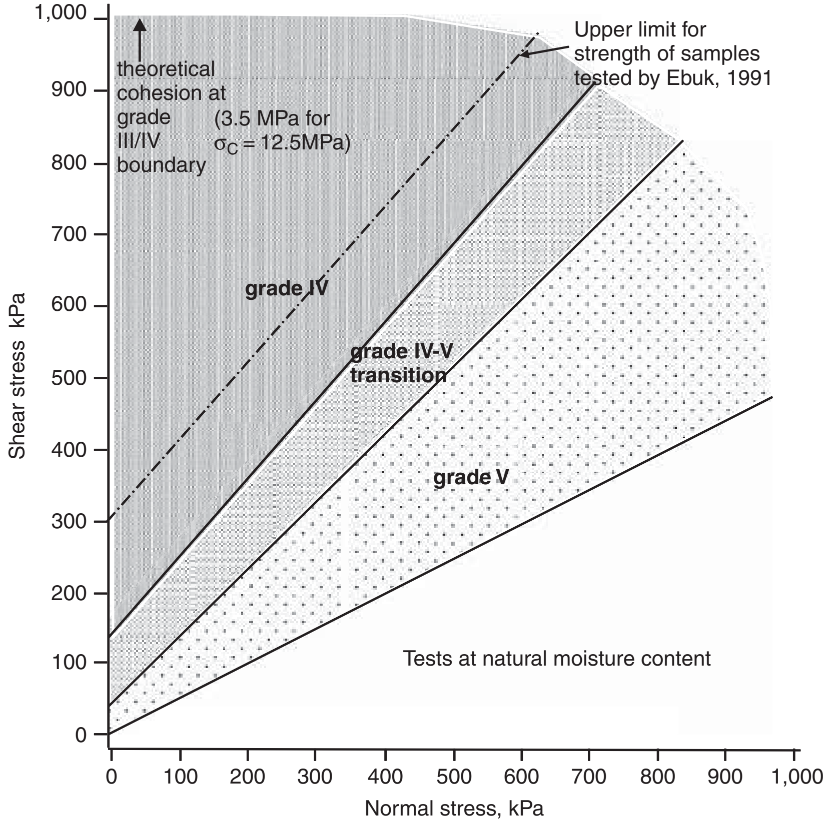


Figure 5.15 Peak strength envelopes for grades IV, V and VI granite (based on Ebuk, 1991). It is highly likely that Ebuk (and others) have not carried out or reported tests on stronger grade IV materials (or if so, the author has not seen them).

weathered rocks in direct shear, measured a maximum cohesion of 300 kPa for grade IV samples (Figure 5.15) but may have only been testing the weaker range of grade IV.

For design, parameters for weathered rock are often estimated from SPT N-value data. Tests are often continued to 100 or even 200 blows, which is questionable practice for many reasons, not least damage to equipment. In terms of rock mass modulus, E, a typical relationship adopted for design is:

$$E = 1.0 \text{ to } 1.2\text{N MPa} \quad \text{(Hencher \& McNicholl, 1995)}$$

For foundation design, parameters such as side friction and end bearing are also often estimated from empirical relationships linked to SPT data. Full discussion of practice in Hong Kong is given in GEO (2006).

5.6.2 Rock mass strength

The presence of discontinuities in many rocks means that intact rock parameters from the laboratory are inappropriate at the field scale.

Therefore, many attempts have been made to represent the overall strength of the rock mass using simple Mohr-Coulomb parameters, friction and cohesion, based on overall rock quality, using classifications such as those presented in Appendix C. For example, using the Rock Mass Rating (RMR) of Bienawski (1989), 'poor rock' would be assigned cohesion 1–200 kPa with friction angle 15–25 degrees, and 'good rock' would be assigned cohesion 3–400 kPa with friction angle 35–45 degrees.

A rather more flexible and geologically realistic approach is to use the Hoek-Brown criteria (Hoek & Brown, 1997; Brown, 2008), which is linked to a Geological Strength Index (GSI) for rating overall rock mass conditions such as 'blockiness' and the roughness or otherwise of discontinuities. The GSI chart is presented and discussed in Appendix C. Given a GSI estimate, the uniaxial compressive strength for the rock blocks and a constant, m_i, which differs for different rock types and has been derived empirically from review of numerous test data (Hoek & Brown, 1980), one can calculate a full strength envelope for the rock mass. A program, RocLab, is downloadable from https://www.rocscience.com and allows values for cohesion and friction to be calculated but it needs to be checked that these relate to the appropriate stress level for the problem at hand. For example, Figure 5.16 shows a steep cut slope in weathered tuff. The question is whether it needs to be cut back or otherwise reinforced or supported. The rock mass is severely

Figure 5.16 Cut slope through weathered volcanic tuff.

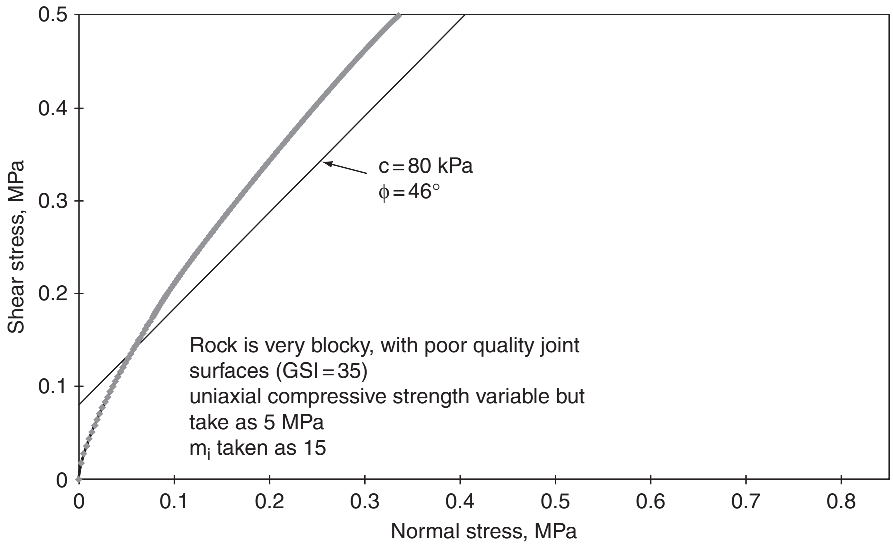


Figure 5.17 Strength envelope for slope in Figure 5.16, based on Hoek-Brown criteria (see text).

weathered. There are corestones of very strong tuff but these are separated and surrounded by highly and completely weathered materials that are much weaker. There are many joints and some of these have kaolin infill. In this case, there are no structural mechanisms for translational failure along daylighting joints, and it is a clear candidate for where a Hoek-Brown/GSI approach might help the assessment. From the GSI chart, one might best characterise the mass as 'very blocky' with 'poor' joint surfaces. The rock type is tuff, so the m_i value is 15 (for granite it would be 33). The difficult parameter is intact strength. In this case, the corestones have UCS values in excess of 100 MPa, but for this assessment I have taken into account the strength of the weakest material making up this slope and, on balance, an average of 5 MPa is considered conservative. Using a spreadsheet from Hoek *et al.* (1995) modified for low stress conditions, the curve shown in Figure 5.17 is obtained. On that basis, for a potential slip surface at a depth of about 10m (vertical stress say 0.27 MPa), appropriate strength parameters might be *c* = 80 kPa and phi = 46 degrees, as shown. Carvalho *et al.* (2007) discuss the assessment of rock mass strength where the intact rock has relatively low uniaxial compressive strength in more detail.

5.6.3 Rock mass deformability

Rock mass modulus is very difficult to predict with any accuracy, and measurements in boreholes or even by large *in situ* tests need to be considered critically and certainly should not be used directly in design without due consideration of the rock qualities of the zone tested (including relaxation) vs. the larger mass volume. Back calculations have been made from large projects, including dams and tunnels, and these data provide the main database for prediction (e.g. Gioda & Sakurai, 2005). Generally, poor quality, highly fractured rock (up to RMR = 50) will have a rock mass modulus increasing from soil-type values of perhaps 500 MPa to about 20 GPa with decreasing fracture spacing and increasing intact compressive strength. As the rock mass quality improves, so the modulus increases markedly, up to values of 60 GPa or so for good-quality rock with RMR = 80. Many authors have attempted correlations between a variety of rock mass classifications (RMR and Q especially) and rock mass modulus, but with considerable scatter. This is perhaps not surprising given the inherent difficulties of 1) trying to represent an often complex, heterogeneous geological situation as a single quality number and 2) the non-uniform loading conditions of any project vs. the measurement system (deficiencies of data).

Hoek & Diederichs (2006) carried out a detailed review and proposed optimised equations linked to the GSI classification. The best-fit equation obtained was:

$$E_{mass}(\mathrm{MPa}) = 10^5(1-D/2)/\left(1 + e^{((75+25D-GSI)/11)}\right)$$

where GSI is as taken from the chart in Appendix C (Table C11). The factor $D = 0$ for undisturbed masses, 0.5 for partially disturbed and 1.0 if fully disturbed. Hoek & Diederichs present a more refined version of this equation using site-specific data for intact strength and modulus, but in many situations the rock mass will not be uniform, so considerable judgement is necessary anyway. Richards & Read (2007) tried applying the Hoek-Diederichs equations to the Waitacki Dam in New Zealand, which was founded on greywacke, and found that the mass modulus was considerably underestimated for a judged GSI of 20, but examination of their data shows how sensitive any prediction is on the GSI adopted. As discussed elsewhere, features like joint spacing and continuity are extremely difficult to measure and characterise and very risky to extrapolate from field exposures because of variations with weathering and the structural regime. This all reinforces the need for considerable judgement and engineering geological expertise in establishing ground models, and caution when applying any empirical relationships.

Large-scale pile loading tests can provide data on rock mass deformation (Hill & Wallace, 2001). They found that published correlations based on RMR and Q classifications overestimated the *in situ* modulus for deep foundation design by up to one order of magnitude, but this was only a significant consideration where the Rock Mass Rating was below 40 (poor and very poor rock masses), and in such cases site-specific testing might be required. As discussed in Chapter 6, the increasing use of Osterberg-type jacks embedded in large-diameter bored piles will no doubt provide very useful data in the future for assessing deformability of rock masses and this, combined with sophisticated numerical modelling, is allowing refinements to the empirical approaches currently in use.

5.7 Rock discontinuity properties

5.7.1 General

The majority of rocks, and some soil masses near the Earth's surface, contain many discontinuities and these dominate mass properties, including strength, deformability and permeability. Discontinuities include bedding planes, cleavage, lithological boundaries, faults and joints. The origins, nature and development of discontinuities are discussed in detail in Chapter 3. For the rest of this discussion, I will discuss joints but this is generally relevant to other discontinuities. Many joints are initiated geologically as incipient weakness directions and only with time do they develop as full mechanical discontinuities, as illustrated in Figure 5.18 and discussed by Hencher & Knipe (2007). In this figure, the incipient cleavage in the slate below the unconformity with the Carboniferous Limestone generally has cohesion almost as high as the rock orthogonal to that cleavage direction. Nearby, however, cleavage and bedding has opened up due to exposure and

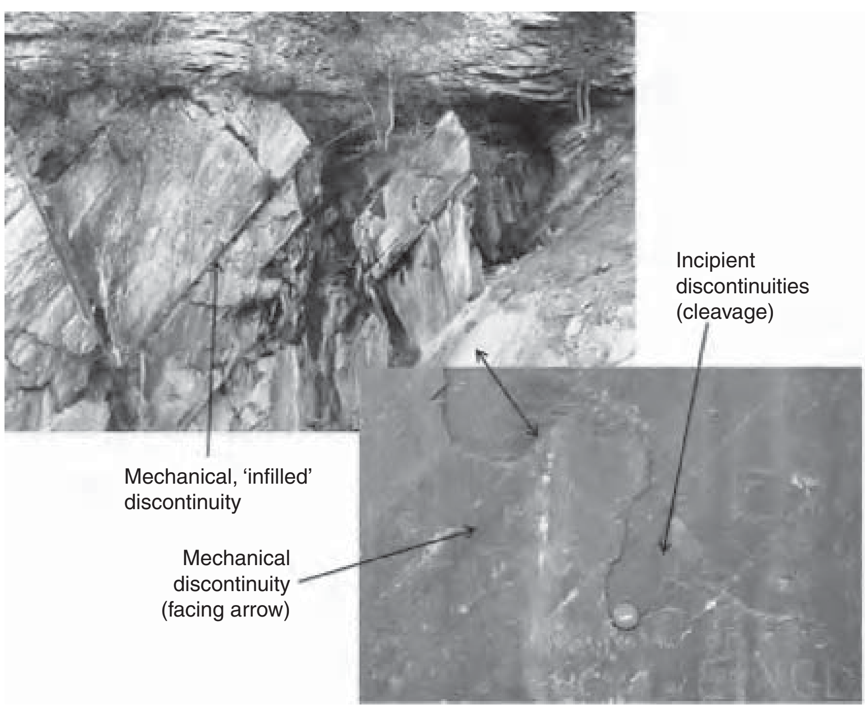


Figure 5.18 Variable development of cleavage and bedding features as mechanical discontinuities. Horton in Ribblesdale, West Yorkshire, UK.

Figure 5.19 Well-defined daylighting discontinuities, clearly only stable due to impersistence (cohesion), Taiwan.

weathering to form persistent joints with zero cohesion. Also shown is a bedding-parallel surface that is infilled with soil – actually a sedimentary feature. At intermediate stages, before rock joints become full mechanical fractures, sections of incipient fractures are cohesive and will contribute strongly to shear strength and shear stiffness along the discontinuity plane. This is illustrated in Figure 5.19. The persistence and shape of rock joints are very challenging parameters to measure or even estimate. Rawnsley (1990) tried to relate joint properties such as style and persistence to geological origin. He concluded, after studying numerous rock outcrops of wide geological age, that whilst persistence can be typified at the scale of joint sets, it is far less predictable at smaller scales (Rawnsley *et al.*, 1990). Zhang & Einstein (2010) review the situation and make some suggestions based on measurement, modelling and theory (see also discussion of DFN modelling in Chapter 3).

5.7.2 Parameters

The main properties of rock joints that need to be measured or estimated are shear strength, normal and shear stiffness and permeability/hydraulic conductivity. These properties depend on the geometry of the joints, including roughness, the nature, strength and frictional properties of the wall rock and any infill between the walls, and their tightness or openness.

Shear and normal stiffness of rock joints are not parameters that are normally required for civil engineering design but are needed as inputs when carrying out numerical simulations of jointed rock masses where each joint is modelled discretely using software such as UDEC. Guidance is given in the UDEC manuals (Itasca, 2004). Permeability of joints depends on their openness, tortuosity and connectivity. It is a very difficult but important subject area, especially for nuclear waste disposal considerations and tunnel inflow assessments (Black *et al.*, 2007).

5.7.3 Shear strength of rock joints

When dealing with rock slopes, often any discontinuity that appears that could be persistent, is treated as so (ignoring potential cohesion from rock bridges). This is a conservative thing to do (see discussion in Chapter 6) but there is little alternative. It is generally agreed that the shear strength of persistent joints is derived from some basic frictional resistance offered by an effectively planar natural joint, plus the work done in overriding the roughness features on that joint. This is expressed by the following equation (after Patton, 1966):

$$\tau = \sigma \tan(\phi_b^\circ + i^\circ)$$

where τ is shear strength, σ is normal stress, $\phi_b°$ is a basic friction angle for a planar joint and $i°$ is a dilation angle that the centre of gravity of the sliding slab follows during shear, i.e. the deviation from the direction that the shearing would have followed if the plane had been flat and sliding had occurred along the mean dip direction of the joint. Despite the apparent simplicity of the Patton equation, derivation of the parameters can be difficult, especially for judging the effective roughness angle. The available international standards and codes deal with this inadequately.

5.7.3.1 Basic friction, ϕ_b

Basic friction of natural joints can be measured by direct shear testing on rock joint samples, but samples taken from different sections of the same joint and joint set can be highly variable, particularly in terms of roughness. Furthermore, it is found that any rough rock joint sample will give different values for peak strength, depending on the direction of shear under the same normal load. Tests need very careful set up, instrumentation, analysis and interpretation, if they are to make sense. A series of tests on different samples of a joint will often yield very wide scatter, which is meaningless without correcting for sample-specific dilation, as described by Hencher & Richards (1989) and Hencher (1995). Dilation reflects work being done in overriding asperities. The dilation angle measured during a shear test will vary, especially according to the original roughness of the sample and the stress level. It is test-specific, will vary throughout a test and with direction of testing. It is not the same as the dilation angle, *i*°, which needs to be assessed at field scale, although the mechanics are the same. To avoid confusion, the laboratory-scale dilation angle measured during a test is here designated ψ°, whereas the field-scale dilation angle to be judged and allowed for in design is *i*°, as defined by Patton (1966).

Typically, because of the complex nature of shearing, with damage being caused to some roughness asperities whist others are overridden, the dilation angle, ψ°, is difficult to predict for an irregularly rough sample, although numerous efforts have been made to do so with some limited success (e.g. Kulatilake *et al*., 1995; Archambault *et al*., 1999). In practice, rather than trying to predict dilation, which will be unique to each sample, stress level and testing direction, it is a parameter that needs to be measured carefully during direct shear tests so that corrections can be made to derive a normalised basic friction angle for use in design. Figure 5.20 shows the result from the well-instrumented first stage of a direct shear test on a rough interlocking joint through limestone. The measured strength throughout the test includes the effect of the upper block having to override the roughness as the joint dilates and work is done against the confining pressure. The dilation curve in Figure 5.20 superficially appears fairly consistent, but if one calculates the dilation angle over short horizontal increments, from the same data set, it is seen to be much more variable and strongly reflects the peaks and troughs of the measured shear strength throughout the test (compare Figure 5.21 with Figure 5.20).

These instantaneous dilation angles can be used to correct (normalise) the shear strength incrementally throughout the test, using the following equations:

$$\tau_{\psi} = (\tau \cos\psi - \sigma \sin \psi)\cos \psi$$

$$\sigma_{\psi} = (\sigma \cos \psi + \tau \sin \psi)\cos \psi$$

where τ_{ψ} and σ_{ψ} are the shear and normal stresses corrected for positive dilation caused by sample roughness. The signs are reversed where compression takes place. By making such corrections, the basic friction angle can be determined for the natural joint surface. In practice, experience shows that for a system measuring to an accuracy of about ±0.005 mm, analysis over horizontal displacement increments of about 0.2 mm generally gives accurate dilation angles, even for a rough tensile fracture (Hencher, 1995). By comparison, if one were to use the average dilation angle throughout the test, as implied in the ISRM Suggested Method (ISRM, 1974), this would not allow the variable shear strength to be understood and might lead to serious errors in determining basic friction values.

Tests can be run multi-stage, in which the same sample is used for tests at different confining stresses, which is very cost-effective, given

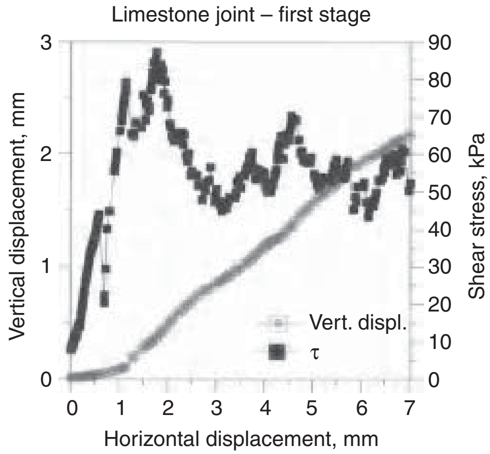


Figure 5.20 Results from single stage of direct shear test on rough induced tensile fracture through limestone. Upper curve shows very spiky shear stress against displacement. The lower line shows vertical vs. horizontal displacement (dilation) throughout this stage of the test. The line has a fairly consistent gradient.

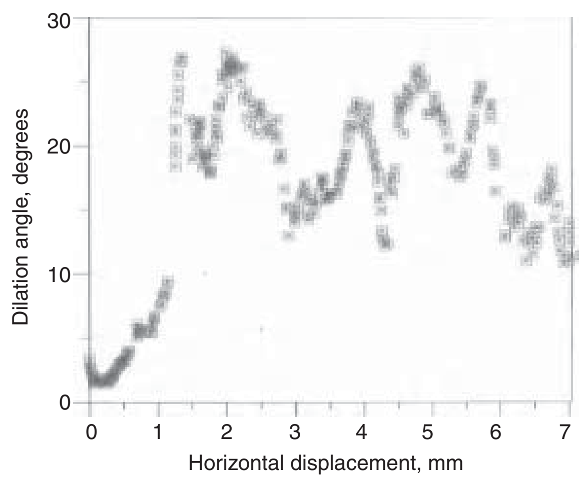


Figure 5.21 Detailed analysis of dilation curve from Figure 5.20 calculated over 0.2 mm horizontal increments. The revealed underlying spikiness in the dilation curve matches that of the shear strength curve in 5.20 and is clearly the cause of variable strength. Details of how the dilation can be corrected for to reveal the underlying basic friction are given in the text.

the difficulties of obtaining and setting up samples. At each stage, the normal load is generally increased (or decreased for experimental reasons) and then the sample sheared until peak strength plus a few mm. Tests must be properly documented, however, with photographs, sketches and profiles, so that any variable data can be explained rationally (Hencher & Richards, 1989). Generally, it is found that

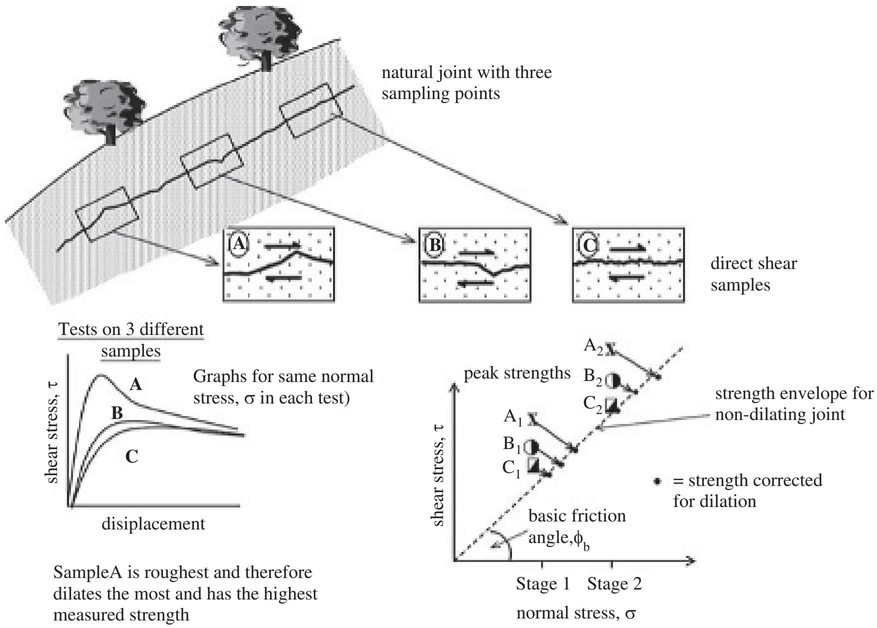


Figure 5.22 Methodology for selecting a series of samples of rock joint, testing and correcting to yield a basic friction angle for the naturally textured rock joint (after Hencher *et al.*, 2011).

tests on a series of samples from the same joint set (with similar surface mineralogy and textures) provide a reasonably well-defined dilation-corrected strength envelope, as illustrated in Figure 5.22. That strength is frictional (obeys Amonton's laws) and comprises an adhesional component plus a non-dilational damage component that varies with texture and roughness.

Barton (1990) suggested that the dilation-corrected basic friction angle might be partly scale-dependent, as assumed for the asperity damage component in the Barton-Bandis model (Bandis *et al.*, 1981), but further research using the same testing equipment as Bandis but with better instrumentation, indicates that this is unlikely (Hencher *et al.*, 1993; Papaliangas *et al.*, 1994). Dilation-corrected basic friction is independent of the length of the sample. Scale effects do need to be taken into account in design but as a geometrical consideration when deciding on an appropriate field scale $i°$ value.

Many silicate rocks are found to have a basic friction $\phi_b \approx 40$ degrees (Papaliangas *et al.*, 1995), and Byerlee (1978) found the same strength envelope ($\tau = 0.85\sigma$) for a large number of direct shear tests on various rock types where dilation was constrained by using high confining stresses. Empirically, it seems to be about the highest value for basic friction achievable for natural joints in many silicate rocks and applicable specifically to joints that are forced to dilate during shear or where dilation is suppressed because of the high normal load. Conversely, much lower friction angles can be measured for natural joints

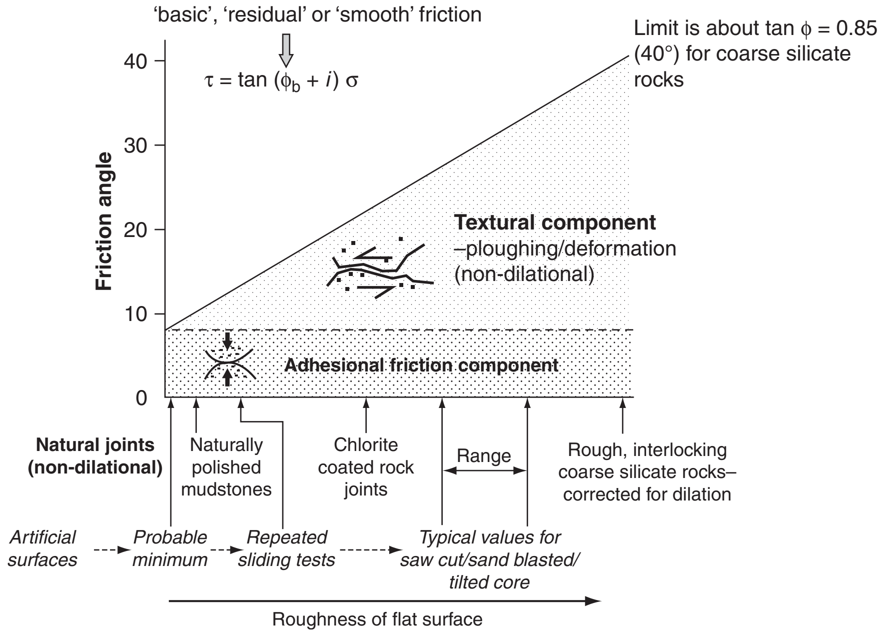


Figure 5.23 Concept of basic friction for a rock joint (after Hencher *et al*., 2011).

where they are planar and where the surface texture is very fine, polished or coated with low-friction minerals, as illustrated by a case example in Box 5-2. The author has measured values of only 10 to 15 degrees for naturally polished joint surfaces through Coal Measures mudstones of South Wales, UK, and such low values are lower than measured for saw-cut surfaces through the parent rock. The range of variation for basic friction, measured for natural joints with different surface textures and for artificially prepared (saw-cut and lapped) joints, is indicated in Figure 5.23.

Box 5-2 Yip Kan Street landslide – an example of use of direct shear testing.

The Yip Kan Street landslide occurred in July 1981 on a dry Sunday night. It mainly involved large blocks of rock of up to 10 m^3, which slid on persistent joint planes dipping at only 22 degrees out of the slope (Hencher, 1981b). The total failure volume was estimated to be 1,235 m^3. The 8 m high, near-vertical slope was cut in very strong, slightly decomposed, coarse-grained igneous rock (quartz-syenite). The upper part of the slope was in saprolite. The failure occurred next to a construction site where blasting had been carried out recently, before the failure but not over the weekend. There had been intense rainfall a week before the failure. The slope had been deteriorating in the days preceding the failure, with cracks in chunam cover in the weathered part having been repaired five days before failure.

Figure B5-2.1 Failure plane with debris cleared off.

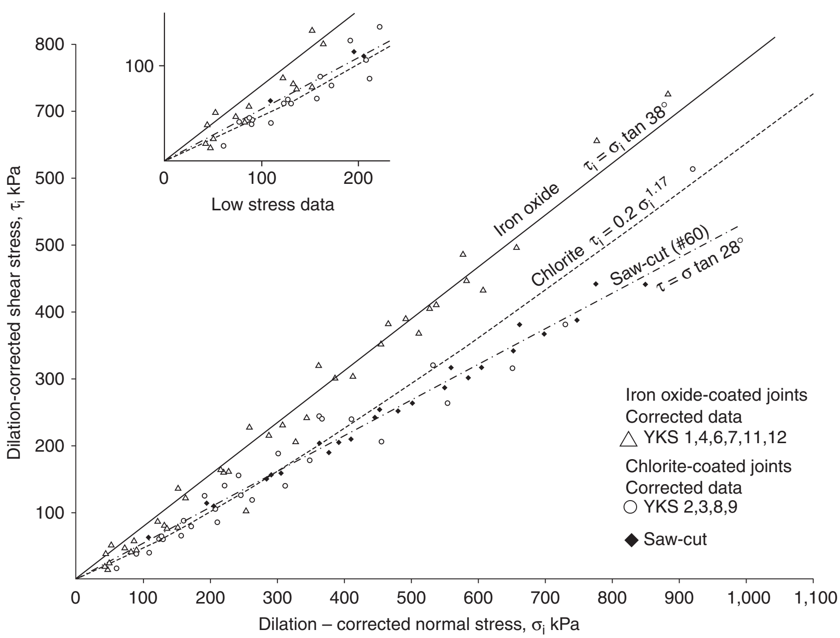


Figure B5-2.2 Shear strength data, Yip Kan Street rock slope failure.

Because of the low angle of sliding, it was decided to investigate in some detail. Blocks were collected – both matching discontinuities and mismatched. It was noted that some blocks were coated with red iron oxides and others with green chlorite (a hard, thin coating). Each sample was carefully described and then tested multi-stage in a Golder Associates direct shear box. At each stage, the test proceeded until peak strength was reached and then for another mm or two, following which the normal stress was increased, without resetting the sample in some cases. For some tests, complete runs of about 15 mm shear displacement were conducted and in one test the sample was tested at the highest stress level first, which was then reduced in stages incrementally. Samples were photographed, roughness measured and damage described carefully. For reference, a series of tests were conducted on saw-cut samples, ground with grade 60 carborundum powder.

Results from the tests are presented in Figure B5-2.2. Tests on natural joint surfaces were corrected for dilation incrementally. It can be seen that the saw-cut surfaces gave a friction angle of about 28 degrees, which is about what might be expected.

The tests from natural joints fall into distinct groups. The data from joints coated with iron oxides define a friction angle of 38 degrees, which is the same as one finds for many weathered rocks (Hencher *et al*., 2011). The data for the chlorite-coated joints were much lower, however, and unexpectedly so. At low stress levels especially, values were very low, below that of the saw-cut joints, as can be seen from the inset figure and about the same as the angle of dip of the planes along which the failure took place ($\phi \cong 20$ degrees at the lowest stress levels). Field-scale roughness was measured at 5 degrees using a 420 mm diameter plate and 9 degrees using a base plate of 80 mm. It was concluded that the failure was progressive, probably having been exacerbated by blasting and previous rainfall and that the initial movements overcame the field-scale roughness. The eventual failure was explained by the presence of persistent chlorite-coated joints with inherently low frictional resistance (Brand *et al*., 1983).

5.7.3.2 Roughness

Roughness at the field scale will often be the controlling factor for the stability of rough or wavy persistent joints and for engineering design must be added to the basic friction, ϕ_b, of the effectively planar rock joint, as determined from corrected shear tests. Roughness is expressed as an anticipated dilation angle, $i°$, which accounts for the likely geometrical path for the sliding slab during failure (deviation from mean dip). There are two main tasks for the geotechnical engineer in analysing the roughness component: firstly, to determine the actual geometry of the surface along the direction of likely sliding at all scales (Figure 5.24) and secondly to judge which of those roughness features along the failure path will survive during shear and force the joint or joints to deviate from the mean dip angle. This is the most difficult part of the shear strength assessment, not least because it is impossible to establish the detailed roughness of surfaces that are hidden in the rock mass. Considerable judgement is required and has to be balanced against the risk involved. Hack (1998) gives a good review of the options, and the difficulties in exercising engineering judgement are discussed in an insightful way by Baecher & Christian (2003).

In practice, the best way of characterising roughness is by measurement on a grid pattern in the way originally described by Fecker & Rengers (1971), adopted in the ISRM Suggested Methods (1978) and described in Richards & Cowland (1982), although spatial variability may be an important issue; the important first-order roughness represented by major wave features may vary considerably

Figure 5.24 Characterising discontinuity roughness using plates of different diameter. Skipton Quarry, West Yorkshire, UK.

from one area to another, as of course also might the mean dip of the plane. At one location, a block might be prevented from sliding by a wave in the joint surface causing a reduction in the effective down-dip angle along the sliding direction; elsewhere, a slab of perhaps several metres length may have a dip angle steeper than the mean angle for the joint as a whole because it sits on the down-slope section of one of the major waves. Defining the scale at which roughness will force dilation during sliding, rather than being sheared through, requires considerable judgement. Some assistance is provided by Schneider (1976) and by Goodman (1980) who indicate that for typical rough joint surfaces, where slabs are free to rotate during shear, as the length of the slab increases (at field scale), the dilation angle controlling lifting of the centre of gravity of the upper block will reduce. The problem cannot be finessed by improved analytical methodology. There is no substitution to careful engineering geological inspection, investigation, characterisation of the ground model and judgement based on experience of similar joints and geological settings, and an appreciation of the fundamental mechanics controlling the potential failure.

5.7.4 Infilled joints

The two walls of a joint might be separated by a layer or pockets of weaker material which may reduce shear strength. A similar situation arises from preferential weathering along a persistent

Figure 5.25 Cut slope at Rhuallt, North Wales, UK. Traversing the slope is a very persistent narrow stratum of weak clay which, combined with cross-cutting faults, provided the mechanism for major rock failure in this otherwise excellent-quality rock mass.

joint. The effect of the infill is a function of the relative height of roughness asperities in the wall rock vs. the thickness of weaker material (Papaliangas *et al*., 1990). If persistent and the infill is of low strength, the consequences can be serious. Cut slopes on the A55 at Rhuallt, North Wales (Figure 5.25), failed by sliding on bedding-parallel thin clay infilled discontinuities with faults acting as release surfaces (Gordon *et al*., 1996). The mechanism had not been anticipated from ground investigation prior to the failure, which involved more than 185,000 m^3 of rock.

In some slopes, incremental movement may take place over many years before final detachment of a landslide and, following each movement, sediment may be washed in to accumulate in dilated hollows on the joint (Figure 5.26). The presence of such infill might cause alarm during ground investigation but in many cases is confined to local down-warps and probably plays little part in decreasing shear strength, other than in restricting drainage (Halcrow Asia Partnership, 1998b). It may, however, be taken as a warning that the slope is deteriorating and approaching failure.

5.7.5 Estimating shear strength using empirical methods

Various empirical criteria have been proposed for estimating shear strength of rock joints, based on index tests and idealised joint shapes. The most widely used is that proposed by Barton (1973). Frictional resistance for saw-cut or other artificially prepared planar surfaces is taken as a lower bound. The limiting value is typically 28.5 to 31.5 degrees according to Barton & Bandis (1990). An additional

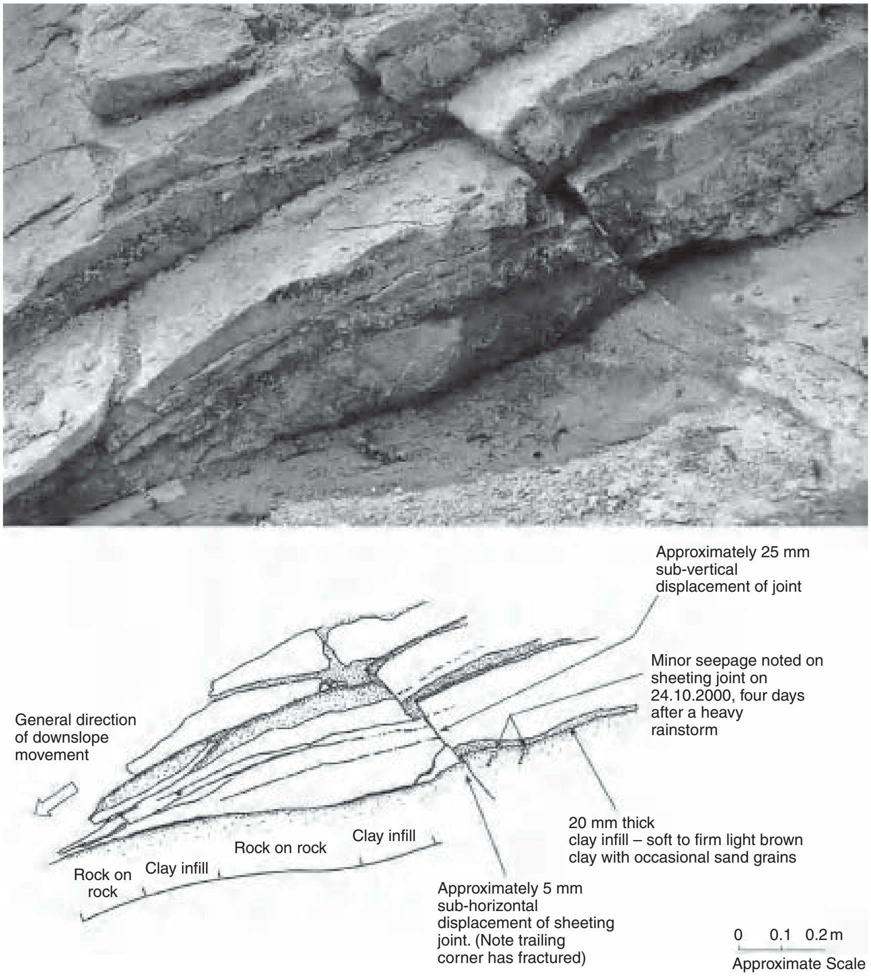


Figure 5.26 Patchily infilled sheeting joints following intermittent displacement prior to failure. Details are given in Hencher (2006).

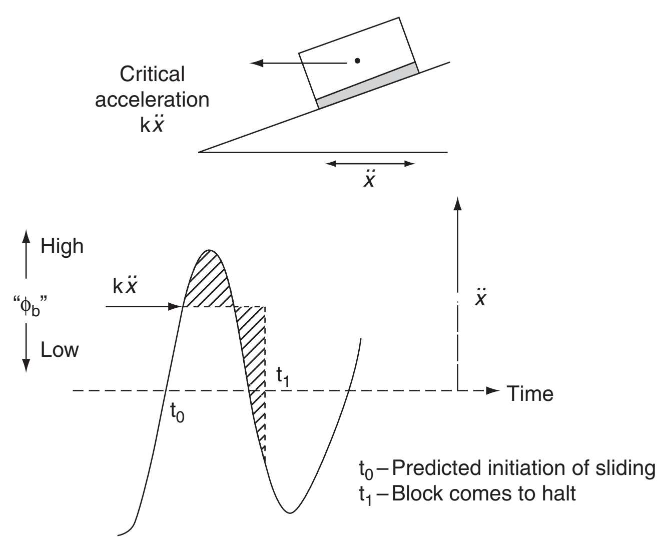
component is then added to account for roughness using a Joint Roughness Coefficient (JRC) usually judged from standard profiles and ranging from 0 to 20. This can be difficult in practice (Beer *et al.*, 2002). JRC is then adjusted for the strength of the rock asperities vs. stress conditions and for scale. Details are given in Brady & Brown (2004) and Wyllie & Mah (2004).The criterion can be incorporated within numerical software for modelling rock mass behaviour such as UDEC (Itasca, 2004). The contribution to shear strength from small-scale roughness is measured or estimated from standard shape profiles (Joint Roughness Coefficient), although this can be difficult in practice (Beer *et al.*, 2002). Larger-scale roughness (waviness) then must be accounted for, over and above JRC, and scale corrections applied.

An important point that needs to be emphasised is that dilation-corrected basic friction parameters from direct shear tests on natural joints should not be used interchangeably in empirical equations as this could lead to an overestimation of field scale strength by perhaps 10 degrees in many cases.

5.7.6 Dynamic shear strength of rock joints

There is some evidence that frictional resistance for rock joints is dependent on loading rate, and this may be significant for aseismic design and for understanding response to blasting. For a block of rock sitting on an inclined plane, given a value for static friction, one can calculate the horizontal acceleration necessary to initiate movement and when the block should stop, given a particular acceleration time history, as illustrated in Figure 5.27. This type of calculation is the basis of the Newmark (1965) method of dynamic slope stability analysis, which is used to calculate the distance travelled, as discussed in more detail in Chapter 6. Hencher (1977) carried out a series of experiments and found that initiation of movement was generally later than anticipated (or did not occur), implying greater peak frictional resistance than predicted from static tests. The effective friction for initiation increased with the rate of loading (Figure 5.28). The implication is that if the loading is very rapid and reversed quickly (as in blast vibrations), shear displacement might not occur, despite the supposed critical acceleration being exceeded. However, once movement was initiated, Hencher found that the distance travelled was higher than anticipated from static strength measurements and interpreted this as reflecting rolling friction and the inability of strong frictional contacts to form during rapid sliding. Hencher (1981a) suggested that for Newmark-type analysis, residual strength should

Figure 5.27
Transient loading of block on a plane.



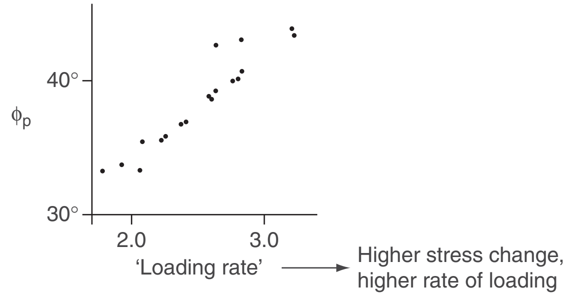


Figure 5.28 Relationship between phi (dynamic) and rate of loading. The higher the rate of application of load (frequency), the greater the initial strength. Data from Hencher (1977, 1981a).

be used for calculating displacements. Recent work confirms low sliding friction angles post-failure (Lee *et al*., 2010).

5.8 Rock-soil mixes

It has long been recognised that mixes of soil and rock, such as illustrated in Figure 5.29, can often stand safely at steeper angles than if the slope were comprised only of the soil fraction. From testing on soils together with theoretical studies, the point at which the hard inclusions start to have a strengthening effect is about 30% by volume.

Figure 5.29 Cutting through boulder colluvium. East of Cape Town, South Africa.

5.8.1 Theoretical effect on shear strength of included boulders

Hencher (1983d) and Hencher *et al.* (1985) report on the back-analysis of a landslide involving colluvium containing a high percentage of boulders, in which an attempt was made to estimate dilation angles on the basis of the coarse fraction percentage estimated in the field and measurements taken from idealised drawings. These estimated field dilation angles were added to the strength for the matrix, determined from laboratory testing. West *et al.* (1992) took this further and identified several ways that included boulders might influence shear strength, based on physical modelling and back analysis of slopes (these are illustrated in Figure 5.30). Factors envisaged included: boulders preventing failure along an otherwise preferred failure path, failure surface forced to deviate around a boulder, and a failure zone incorporating the boulder. Triaxial tests reported by Lindquist & Goodman (1994) similarly concluded that boulders increase the mass strength. Additional review is provided by Irfan & Tang (1993).

Practical methods for addressing the strength of mixed soils and rocks remain difficult. One of the main problems is that such masses can be highly heterogeneous and difficult to characterise realistically. The other is that whilst trends of increasing mass strength with percentage of rock clasts and boulders are clear, general rules have not yet been formulated. Further advances will probably be by numerical

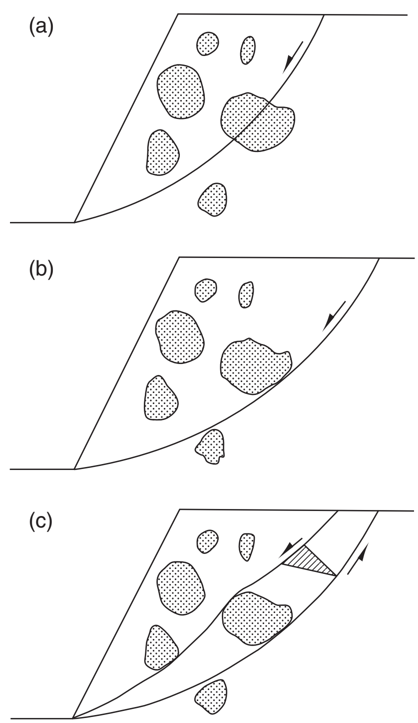


Figure 5.30 Mechanisms of failure through a mixed rock and soil slope. After West *et al.* (1992).

modelling and could be done using PFC3D (Itasca). Whilst the largely intractable geological characterisation nature of the problem would remain, the problem could probably be resolved parametrically in a similar way and with a similar level of success for prediction as the Hoek-Brown model for fractured rock masses.

5.8.2 Bearing capacity of mixed soil and rock

Mixed soil and rock deposits include sedimentary deposits like colluvium and glacial boulder clay, but also some weathered rocks. As for assessing shear strength, there are considerable difficulties for sampling and testing and there can also be significant problems for construction (e.g. Weltman & Healy, 1978). The conservative position for design is to take the strength and deformability of the matrix as representative of the mass, but allowance might be made for the included stiffer and stronger clasts by rational analysis, perhaps backed up by numerical modelling.

5.9 Rock used in construction

Crushed rock and quarried or dredged sand and gravel are important materials used in making concrete and construction generally, perhaps as fill. Rock is also used as armourstone, for example, in protecting earth dams from wave action or for forming harbours. It is also cut and polished as dimension stone to be used as kitchen work surfaces or as cladding on the outside of prestigious buildings. Engineering geologists are often required to identify sources of aggregate, either from existing quarries but sometimes from new borrow areas in the case of sand for reclamation or new quarries for a remote project such as a road. Some of the properties that are important for their use are the same as in much of geotechnical design: strength, unit weight and porosity, but there are other properties that need to be tested specifically.

5.9.1 Concrete aggregate

For concrete, the aggregate must be sound, durable and chemically stable. Materials to be avoided include sulphates (e.g. gypsum and pyrites), clay and some silicate minerals such as opal and volcanic glass, which can cause a severe reaction and deterioration of the concrete if present in the wrong proportions (see case example of Pracana Dam in Chapter 7). Tests are available and should be used to ensure that the aggregate being sourced is suitable. These include mortar bar tests whereby a test mix of concrete is formed and observed to see if it expands with time. Other factors might include the need for light- or heavy-weight concrete, fire resistance and overall strength. Concrete mix design for a large project may require a research programme to optimise the aggregate specification and type of cement to use. For smaller projects or where the demands are less onerous, then cost may be the controlling factor; aggregates and quarries have place-value, which is a matter of the quality of aggregate at a particular quarry together with the costs of transport to the project site. A useful review of the factors to be considered in specifying concrete aggregate is given by Smith & Collis (2001).

5.9.2 Armourstone

Armourstone is used to protect structures primarily from wave action and is often made up of blocks of rock of several tonnes. Generally, the rock must be durable and massive. If it softens or discontinuities open up with time, then the function is lost. Massive crystalline limestone often works well, as do many igneous and metamorphic rocks. Usually, durability (and availability and cost) is all-important but see the case history of Carsington Dam in Chapter 7 where the choice of limestone as riprap contributed to adverse chemical reactions and environmental damage. Weak or fractured rocks are obviously not appropriate. For many coastal defence works in the east of England, large rock blocks are brought by barge from Scandinavia because of a lack of suitable local rock. CIRIA (2005) provides useful guidance. Where suitable rock is not available then concrete tetrapod structures known as dolosse are used in the same way, piled on top of one another and interlocked, to protect coasts and structures by dissipating wave energy.

5.9.3 Road stone

Aggregate is used in road construction in many different ways – as general fill or in the sub-base, as drainage material and in the wearing course. There are many different standard tests to be applied in road construction, and these are described in Smith & Collis (2001). The most demanding specification is for wearing course material. Rock must be strong and durable but also must resist polishing as it is worn by traffic. This requires the rock used to comprise a range of different minerals that are strongly bonded but wear irregularly. Rocks like limestone are generally unsuitable (the polished stone value, PSV, is too low). Rocks like Ingleton granite, which is really an arkose, have excellent properties and therefore very high place values – worth quarrying and transporting large distances – even from a National Park.

5.9.4 Dimension stone

Dimension stone is quarried to be used directly in building, construction or even sculptures. Typical rocks quarried in this way include marble, granite and slate for roofs. Rocks are generally chosen for their colour and appearance – the quarry at St Bees headland, Cumbria, UK (a fairly ordinary sandstone), was re-opened temporarily in the 1990s to provide rock for shipping to New York to repair buildings faced with sandstone carried by ships as ballast in the 19th Century – because of its appearance. Dimension stone must also be resistant to wear, frost and chemical attack. This can be difficult to determine from direct testing, so experience of the long-term performance of a particular rock from a particular quarry may be the best clue.