Chapter 1
Properties of Unsaturated Soils

1.1 Nature and genesis of unsaturated soils

The term *soil* as used in geotechnical engineering encompasses a wide spectrum of particulate materials. In the saturated state all the void spaces between the particles are filled with water, but in the unsaturated state a proportion of the void spaces is filled with air. We will primarily be concerned with the analysis of fine-grained soils comprising clays and silts, or soils containing coarser sand and gravel that have a significant percentage of fines. As shown in Figure 1.1, which illustrates the gradings of a number of soils of different lithologies, the finer clay soils have particle sizes less than 2 μm that are not visible to the naked eye, while the coarser gravels range up to 60 mm. The following can serve as a yardstick to the range of particle sizes: if a clay platelet was expanded to the size of a saucer, a coarse gravel particle would have a proportional dimension of around 10 km. This wide range of particle sizes and the inherent variability of soils give rise to behavioural characteristics not readily amenable to rigorous analysis. Stress history, particle shape and time-dependent characteristics also influence their multi-faceted behaviour, generally requiring simplifications and generalisations in formulating solutions to geotechnical problems.

The solid particles, water and air are the phases making up a soil mass. Classically, researchers have achieved recognisable success in developing an understanding of the behaviour of saturated, fine-grained soils and the behaviour of dry, coarse materials, such as sand and gravel. Extending our understanding to the behaviour of unsaturated soils, particularly those with a significant percentage of fines, has proved problematic. This is principally because of the additional fluid phase of air or other gases in the void spaces, which complicates the thorny issue of the controlling stress regime. Interpretation of the behaviour of unsaturated soils requires the differences in the air and water pressures, the phase compressibilities and their interactions, as well as chemical effects, to be taken into account. The interactions include the contractile skin between the fluid phases, which gives rise to a surface tension effect that is particularly influential in creating the characteristic aggregated structure of fine-grained soils. The significances of the phases and their interactions in influencing soil behaviour are investigated in later chapters.

Both natural soils and engineered ground are liable to be in an unsaturated condition. Natural soils in an unsaturated state are common in arid or semi-arid areas where the groundwater table is often many metres deep. Around one-third of the earth's surface is
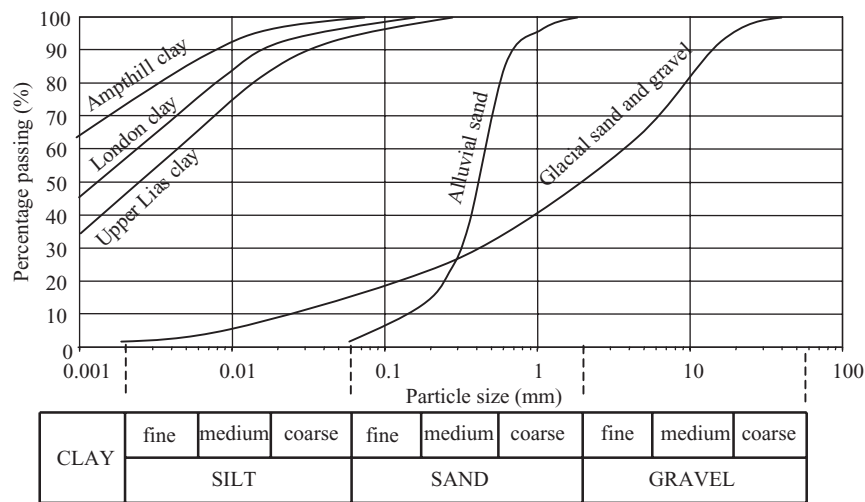
Figure 1.1   Particle size distribution.

situated in arid or semi-arid regions where the potential evaporation exceeds the precipitation (Barbour, 1999). However, any soil near the ground surface in a relatively dry environment is liable to have a negative pore water pressure (water pressure relative to a datum of atmospheric air pressure) and could experience de-saturation or air entry into the pore spaces. Though the soil may be saturated for some height above the water table, if the pore water pressure drops sufficiently, air will enter the pore spaces. Figure 1.2 illustrates the change from a positive pore water pressure below the water table to a negative pore water pressure above the water table. While the plot indicates a reduction in negative pore water pressure close to the ground surface, where precipitation would increase the degree of saturation, increased desiccation due to evaporation can be expected to occur in a hot environment. Compacted fills such as in earth dams, road subgrades and embankments are
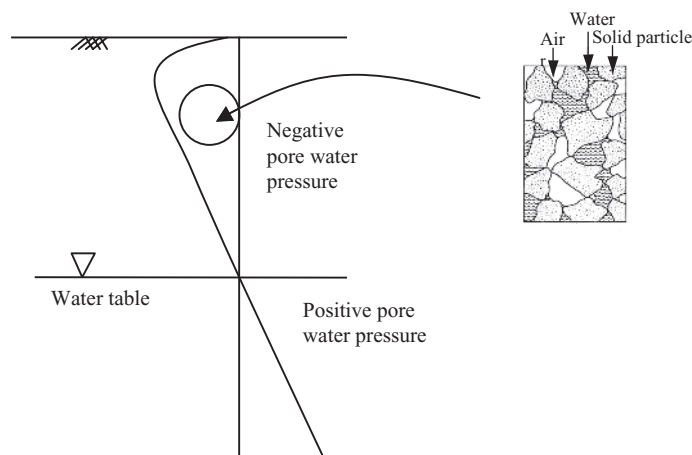

Figure 1.2   Unsaturated soils in the ground.

also usually placed in an unsaturated state as complete compaction and closure of all air voids is generally impractical. Negative pore water pressure[1] is the key to understanding unsaturated soil behaviour and in interpreting the significance to engineering structures.

Climate plays an important role in the formation of unsaturated soils (Lu and Likos, 2004), with evaporation in hot weather drying out the ground leading to shrinkage of fine-grained soils and, ultimately, to shrinkage cracking. Subsequent wetting, following rain, leads to swelling and closure of cracks, but not necessarily to total eradication of suction-induced soil structure. Aggregation of fine particles following drying affects subsequent behaviour characteristics. Future climate changes due to global warming could potentially cause significant changes in the soil moisture regime, and thus in soil conditions over large areas of the world. Uptake of water by vegetation can also lead to significant ground de-saturation due to evapotranspiration, while removal of vegetation can lead to subsequent re-saturation leading to potential stability problems, notably instability of hillsides due to deforestation. Where unsaturated soils comprise *highly* plastic clays, large swelling and shrinkage phenomenon can occur due to uptake of water or reduction in water content, resulting in ground movements capable of causing severe structural damage. The primary cause of expansive clays is the presence of swelling clay minerals such as montmorillonite. In temperate zones such as Britain, seasonal volume change in clay is generally restricted to the upper 1.0–1.5 m (Bell and Culshaw, 2001). Nevertheless, in the South and South Midlands regions of England, the presence of clay formations such as the London clay, Oxford clay, Kimmeridge clay and Lias clay has, in dry summers, particularly in conjunction with tree root action, led to large numbers of insurance claims for cracking of domestic properties as a result of excessive ground shrinkage.

Collapsible soils represent an important phenomenon. Loess or loosely compacted fills in an unsaturated state can experience large collapse settlements if they are wetted, bringing about damage to overlying structures. In developing areas of the world, notably East Asia, residual soils from decomposed rock exhibit complex behaviour characteristics, attributable in part to partial saturation (e.g. Lee and Coop, 1995; Ng and Chiu, 2003). The rapid structure and infrastructure development in areas such as the tropics is liable to lead to enhanced awareness of potential problems with unsaturated soils. Further reading on the wide range of geotechnical problems, where an understanding of unsaturated soil behaviour is a prerequisite to determining engineering solutions, is given by Alonso and Olivella (2006).

Within the context of the foregoing discussion, the remainder of this chapter deals with the phases and their interactions and addresses some of the basic principles governing the behaviour of soils, particularly those soils in an unsaturated state. First, however, it is necessary to define some of the variables used to describe soils.

1.2 Soil variables

Subsequent sections of this book employ the following variables. It is these tools that are used to form the basic equations governing the behaviour of unsaturated soils.

1.2.1 Volume relationships

An important theme within this book is the thermodynamic link between the pressures (and stresses) and the volumes of the phases in a soil at equilibrium, and the link between
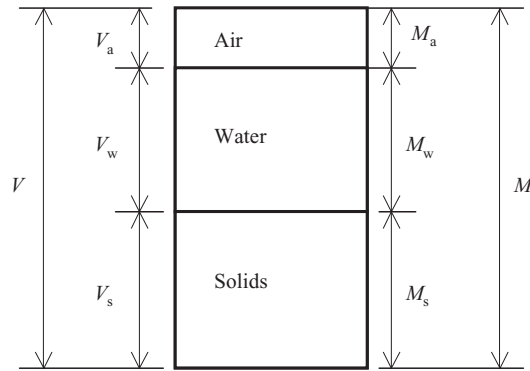
4   Unsaturated Soils

Figure 1.3 Phase diagram.

the pressures (and stresses) and specific strain increments in analysing changes between equilibrium states. These 'conjugate' variables are particularly relevant in unsaturated soils. However, we are racing ahead of ourselves and it is first necessary to introduce a number of simple variables for the relative volumes of the phases that are illustrated in Figure 1.3. The first are the porosity terms that relate the volume of the voids (V_v) and the volumes of the phases V_s, V_w and V_a for the solid particles, the water and the air respectively to the total volume of the soil V:

$$n = \frac{V_v}{V} \qquad n_s = \frac{V_s}{V} \qquad n_w = \frac{V_w}{V} \qquad n_a = \frac{V_a}{V} \tag{1.1}$$

where n is porosity and the suffices 's', 'w' and 'a' attached to the volume and porosity terms relate to the solid particles, the water and the air phases respectively.

The porosity term for the solid particles does not fit well with the idea of solid particles but reflects the relative volume filled by the solid particles and is convenient as it allows us to write the following equation:

$$n_s + n = n_s + n_w + n_a = 1 \tag{1.2}$$

The porosity terms are expressed here as ratios but are often expressed as a percentage by multiplying by 100. The air porosity n_a is more frequently referred to as the air voids content and is particularly useful in expressing the adequacy of compaction in earthworks operations where effort is usually made to reduce the air voids content to a low level.

Voids ratio e is an alternative representation of the volumetric variables and is conveniently used to express volumetric changes in consolidation or compression tests, particularly for saturated soils. The voids ratio expresses the ratio of the volume of the voids to the volume of the solids:

$$e = \frac{V_v}{V_s} \tag{1.3}$$

The relationship between porosity and voids ratio is given by:

$$n = \frac{e}{1+e} \tag{1.4}$$

The degree of saturation S_r is also a commonly used variable in unsaturated soils and is expressed in percentage terms as:

$$S_r = \frac{V_w}{V_v} \times 100\% \qquad [1.5]$$

For dry soils $S_r = 0$ and for saturated soils $S_r = 100\%$. Between these two extremes the degree of saturation has an intermediate value. The degree of saturation is an important term where there is fluid movement as it expresses the relative volumes of the water and air in the pore spaces. However, the controlling stresses and the strength of unsaturated soils are often correlated with the degree of saturation. The use of the degree of saturation, on its own, is considered a poor choice of variable to which to relate stresses and strength. There is no term for the volume of the solids in the definition of S_r, yet it is the solid particles that give a soil its strength. Far better variables and ones that emerge naturally from thermodynamic analysis later in this book are the specific volume v, the specific water volume v_w and the specific air volume v_a defined as:

$$v = 1 + e = \frac{V}{V_s} \quad v_w = 1 + eS_r = \frac{V_s + V_w}{V_s} \quad v_a = e\,(1 - S_r) = \frac{V_a}{V_s} \qquad [1.6]$$

The specific volume terms are additive in a similar manner to the porosity terms other than the term for the volume of solids which is included within the specific water volume; thus:

$$v = v_w + v_a \qquad [1.7]$$

The specific volume v is defined as the total volume of soil per unit volume of solids, and the difference $(v - v_w)$ is the volume of air voids per unit volume of solids v_a, frequently defined in the literature as the air voids ratio e_a. In fine-grained soils in an unsaturated state, the soil particles have a tendency to collect together as *aggregates* of particles, typically of silt or sand size, with relatively large inter-aggregate void spaces. The water phase and the soil particles have a close affinity, and where the aggregates of soil particles in an unsaturated soil contain all the water, v_w can be defined as the volume of the aggregates per unit volume of solids. Accordingly, the ratio of the specific water volume to the specific volume (Equation 1.8) expresses the volume of the aggregates per unit volume of soil:

$$\frac{v_w}{v} = \frac{V_s + V_w}{V} \qquad [1.8]$$

This equation contains a term for the volume of the solid particles and a term for the volume of water phase, and because there is a term for the total volume, there is inbuilt into the formulation a term for the air phase. The equation presents the volumetric terms for all the phases in an unsaturated soil in a single formulation. The ratio v_w/v is shown in the following chapters to be a far better variable than S_r to which to relate the controlling stresses and the strength of unsaturated soils. While there would be some merit in replacing v_w/v with a unifying symbol, this is resisted as it would mask the importance of the volumetric terms in the subsequent analysis.

1.2.2 Water content relationships

The amount of water in a soil is generally expressed in one of two ways. The water content w (frequently termed the gravimetric water content) is expressed as:

$$w = \frac{M_{\rm w}}{M_{\rm s}} \times 100\% \qquad [1.9]$$

where $M_{\rm w}$ is the mass of water and $M_{\rm s}$ is the mass of solid particles.

An alternative definition is the volumetric water content $\theta_{\rm w}$:

$$\theta_{\rm w} = \frac{V_{\rm w}}{V} \qquad [1.10]$$

This is synonymous with the definition of $n_{\rm w}$ in Equation 1.1.

1.2.3 Density relationships

There are two important density variables for a soil mass: the bulk density and the dry density. The bulk density $\rho_{\rm b}$ is the ratio of the total mass $M_{\rm t}$ to the total volume, where the total mass and the total volume are the sums of the masses and volumes of the phases respectively, as illustrated in Figure 1.3:

$$\rho_{\rm b} = \frac{M_{\rm t}}{V} \qquad [1.11]$$

The dry density $\rho_{\rm d}$ is the ratio of the mass of the solid particles $M_{\rm s}$ to the total volume:

$$\rho_{\rm d} = \frac{M_{\rm s}}{V} \qquad [1.12]$$

The density of the solid particles $\rho_{\rm s}$ is often related to the density of water $\rho_{\rm w}$ in a dimensionless term variously referred to as the relative density, specific weight or specific gravity $G_{\rm s}$. The relative density is the density of the soil particles divided by the density of water at a temperature of $4°C$ under normal atmospheric pressure:

$$G_{\rm s} = \frac{\rho_{\rm s}}{\rho_{\rm w}} \qquad [1.13]$$

In earthworks operations such as in road embankments and earth dams, soils are compacted to improve their strength and compression properties. For a given compaction effort, there is a water content, termed the optimum water content (OWC), at which a soil achieves a maximum dry density. A typical laboratory dry density – water content relationship is illustrated in Figure 1.4. Greater compactive effort leads to an increase in the maximum dry density and a reduction in the OWC. It is important to carefully select the appropriate laboratory compaction test to replicate as closely as possible the compaction conditions achievable on site. Standard Proctor compaction is normally considered achievable under reasonably controlled site operations. Modified Proctor compaction, using greater compactive energy, can be achieved under carefully controlled site conditions using modern compaction plant[2]. Wet of optimum, compaction plots lie close to the zero air voids line, where the air voids content is defined as $n_{\rm a}$ in Equation 1.1. The zero air voids line represents the maximum achievable dry density for given water content, and on this line all the air voids are eradicated. Dry of optimum, the soil becomes progressively less saturated, suction increases and there is increased evidence of a fissured soil structure.
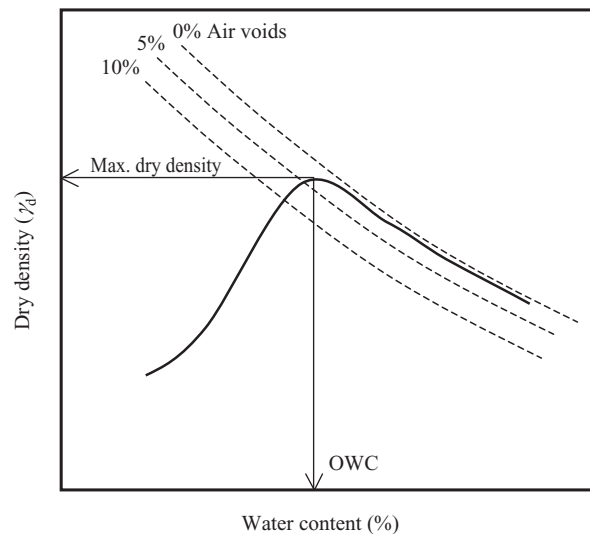
Figure 1.4   Compaction dry density – water content relationship.

1.3 Particle properties

Destructive processes such as chemical and physical weathering lead to the formation of soils from rock. Clay soils are generally the products of chemical weathering. The chemical processes due to the actions of water (especially if it is acidic or alkaline), oxygen and carbon dioxide (Craig, 1997) change the mineral form of the rock, leading to the formation of crystalline particles of colloidal sizes known as clay minerals. Natural clays generally contain a range of clay minerals, the most widely encountered minerals being kaolinite, calcite, illite, dolomite and smectite (the last clay mineral species including montmorillonite).

The types and proportions of minerals present in clay significantly influence its index properties. McLean and Gribble (1988) reported a range of liquid limits (w_L) and plastic limits (w_P) for the clay minerals kaolinite, illite and montmorillonite, as shown in Table 1.1. Also included are the limits for sodium-rich bentonite (impure clay consisting mostly of montmorillonite) and some common British soils.

We will not describe the tests to determine the limits (w_L) and (w_P) for fine-grained soils as these are provided in many textbooks and standards, but we will describe their significance in general terms based on Figure 1.5. The liquid limit is the water content above which a disturbed soil starts to flow and to behave as a liquid, and the plastic limit is the water content below which a disturbed unconfined soil fissures and behaves as a brittle solid. Between these limits the soil is said to behave as a plastic material.

Activity, defined as the ratio of plasticity index I_P (where $I_P = w_L − w_P$) to the percentage clay content, describes the sensitivity of clay (Skempton, 1953). The activity of London clay is typically within the range 0.5–0.83 (Gasparre *et al.*, 2007; Monroy *et al.*, 2010) while for Gault clay it is approximately 0.6 and for Belfast Upper Boulder clay approximately 0.5 (Sivakumar, 2005). Much of the experimental data reported later in this book uses kaolin with an activity of approximately 0.4. Skempton (1953) quoted an activity for sodium-rich montmorillonite of 7.2, while Blatz and Graham (2003) reported an activity

Table 1.1   Typical clay mineral and clay soil properties.

	(w_L) (%)	(w_P) (%)
Kaolinite	60–73	26–37
Illite (or hydrous mica)	63–120	34–60
Montmorillonite	108–700	51–60
Sodium-rich bentonite[a]	354	27
Ampthill clay	66–89	23–30
Mercia mudstone[b]	25–60	17–33
Etruria marl[c]	35–52	14–21
Belfast Upper Boulder Clay	59–65	27–31
Upper Lias clay[d]	57–65	24–31
London clay[e]	60–83	21–32

[a]After Stewart *et al.* (2001).
[b]After Chandler and Davis (1973).
[c]After Hutchison *et al.* (1973).
[d]After Chandler (1974).
[e]After Ward *et al.* (1959), Gasparre *et al.* (2007) and Monroy *et al.* (2010).

for sodium-rich bentonite of around 2. Clays are generally categorised into three groups in terms of their activity:

Inactive clay: activity <0.75
Normal clay: activity 0.75–1.25
Active clay: activity >1.25

In a relatively dry, fine-grained soil the particles are held together by a relict water phase, under a tensile suction force, in the fine intra-aggregate pore spaces. On wetting, the suction is reduced and the soil expands. This inter-particle swelling takes place in all clay soils. However, some clay minerals also exhibit significant intra-crystalline swelling. This is a particular characteristic of smectitic clay minerals such as montmorillonite. The
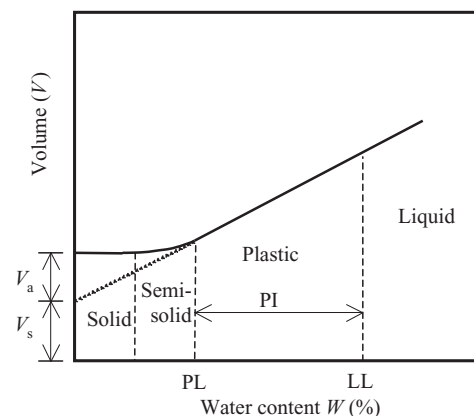

Figure 1.5   Plasticity characteristics.

individual molecular layers that make up the crystal structure are weakly bonded so that on wetting water enters not only between crystals but also between the layers that comprise the crystals. The adsorption of water into clay particle surfaces and the crystalline swelling of some clay minerals are discussed further in Section 1.4.5.

Generally kaolinite has the smallest swelling capacity of the clay minerals with nearly all the swelling occurring as inter-particle swelling. Larger swelling potential, in order of increasing magnitude, is exhibited by illite, calcium-montmorillonite and sodium-montmorillonite. Natural soils comprise mixtures of clay minerals and exhibit the net effect of the mineral composition. Increasing plasticity index and activity give a general guide to increasing swelling potential and the significance of intra-crystalline swelling (Grim, 1962; Bell and Culshaw, 2001).

1.4 Phase properties and interactions

The proportions of the three phases making up unsaturated soils have a significant influence on their behaviour as do the interactions within and between the phases. The interactions are particularly influential in the perception of equilibrium and the rates of mass change and redistribution. The interactions need careful appraisal to appreciate their significance.

At any point in a fluid under equilibrium conditions, the pressure is equal in all directions. However, gravitational effects give rise to greater pressures with depth and closeness to the earth's centre. In a soil the particles interact and the stresses at any point are, in general, different in different directions. Thus, soils exhibit a heterogeneous stress regime with stress levels generally increasing with depth. In laboratory soil tests the size of the specimen[3] tested is usually sufficiently small that the effect of the change in gravitational effects between the top and bottom of the specimen can be ignored without any significant loss of accuracy. This is the case for specimens tested in triaxial cells, the results from which are usually considered to represent the conditions at a point, or at least a localised volume.

For a soil system in equilibrium, an applied external pressure (or stress regime) is balanced by the components of pressure and stress arising from the phases and their interactions, and given by:

- The water pressure acting through the volume of the water;
- The air pressure acting through the volume of air;
- Any chemical imbalance leading to osmotic suction effects
- The water vapour in air;
- The dissolved air in water;
- The contractile skin formed at the water interface where it meets the air;
- The adsorbed double layer of water on the soil particle surfaces and absorbed crystalline water;
- The pressure acting through the solid phase because of the surrounding fluid(s);
- The interaction between the soil particles, which can be viewed on a micro-mechanical scale as the interaction stresses between the soil particles, or on the macro-mechanical scale as the stresses acting over a total planar area.

While some of the interactions between the phases contribute very little to resisting applied forces, at least at equilibrium, and can be ignored, it is important to understand the contributions of the phases and the interactions to overall soil behaviour.

1.4.1 Soil suction

Soil suction is a major factor affecting the behaviour of unsaturated soils. Total suction Ψ has two components, namely matric suction and osmotic suction. This can be expressed as:

$$\Psi = (u_a - u_w) + \phi_s \qquad [1.14]$$

where $(u_a - u_w)$ is the matric suction, being the difference between the pore air pressure u_a and the pore water pressure u_w, and ϕ_s is the osmotic suction, being the result of chemical imbalance between the pore water in the soil volume under consideration and an external source of water.

The significance of suction in soil has been recognised for many decades. The ability of a soil to absorb additional water, whether it is fully saturated or unsaturated, is termed soil suction and can be defined as the free energy state of soil water (Edlefsen and Anderson, 1943). The partial vapour pressure of the soil water can be used to measure the free energy state of the soil water (Richards, 1965), and the following thermodynamic relationship exists between soil suction and the partial vapour pressure:

$$\Psi = -\frac{R_u T \rho_w}{\omega_v} \ln\left(\frac{u_v}{u_{vo}}\right) \qquad [1.15]$$

where R_u is the universal gas constant, T is absolute temperature, u_v is the partial pressure of pore water vapour, u_{vo} is the saturation pressure of pore water over a flat surface of pure water at the same temperature (u_v/u_{vo} is the relative humidity R_h) and ω_v is the molecular mass of water vapour.

While matric suction will be of prime interest in this book, it is important to recognise the role of osmotic suction. This results from retention energy due to the presence of salts in the pore water, or more precisely a difference in salt concentrations in the pore water in the system being analysed, and the surrounding water. Osmotic suction can attract or remove water from a system. The osmotic suction represents the ionic potential of the pore fluid in a soil (Fredlund and Rahardjo, 1993). Osmotic suction can be altered by either changing the mass of water or the amount of ions in solution. However, the strength of an unsaturated soil is principally controlled by the matric suction, even though the presence of salts within the soil water can give rise to some fundamental changes in mechanical behaviour (Alonso *et al.*, 1987).

Matric suction is a result primarily of the phenomenon of capillarity, but is also influenced by surface adsorption effects. The capillarity phenomenon is directly related to the surface tension of water and results, for example, in water rising up thin capillary tubes, as illustrated in Figure 1.6, and forming a curved surface between the water and air known as a *meniscus*. In unsaturated soil mechanics, the water–air interface is often referred to as the *contractile skin*. For equilibrium at the air–water interface in the capillary tube, the pressure difference across the meniscus $(u_a - u_w)$ is given by:

$$(u_a - u_w) = \frac{2T_c}{R} \qquad [1.16]$$

where $(u_a - u_w) = \rho_w g h_c$, h_c is the capillary rise, R is the radius of curvature of the meniscus (where $R = r_t/\cos\theta$), r_t is the radius of the capillary tube, T_c is the surface tension of the air–water interface (contractile skin) with units of force per unit length or energy per unit area and θ is the contact angle of the air–water interface with the wall of the capillary tube.
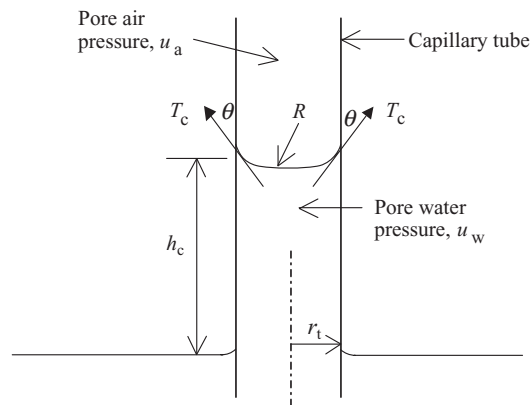
Figure 1.6   Capillary model.

In soils, the pores act as tortuous capillary tubes and result in the soil water rising above the water table. The finer the pores, the greater the meniscus curvature and the higher the water is elevated. The capillary water has a negative water pressure with respect to the air pressure and its magnitude is inversely proportional to the radius of curvature of the meniscus. In other words, negative pore water pressure, or matric suction, increases as the radius of the meniscus decreases. Accordingly, fine-grained soils normally experience greater capillary rise than coarse-grained soils where the pore spaces are larger, though there are recognisable effects on the overall capillary rise from changes in pore diameter within soils, as soils comprise discrete particles not uniform tubes.

Surface adsorption phenomena also influence the matric suction and are particularly important to clay minerals. Surface adsorption results from the negatively charged surfaces of clay particles (Figure 1.7) (Mitchell, 1993). The relatively powerful electrical force
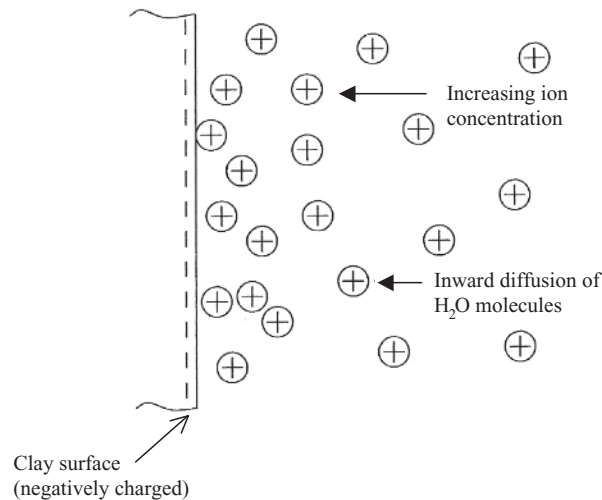

Figure 1.7   Water adsorption by a clay particle (after Mitchell, 1993).

around clay particles strongly attracts water molecules leading to an adsorbed water layer that effectively reduces the inter-particle pore spaces. The clay mineralogy plays an important role in the thickness of the adsorption layer and those soils with high-activity clay minerals, such as montmorillonite, are able to retain a greater amount of adsorbed water around their particles.

In unsaturated soils, the pore water has a pressure less than the air pressure and the contractile skin that forms at the interface between the phases is concave to the water phase, as illustrated in Figure 1.6. The water pressure deficiency in the pore spaces may have a magnitude many hundreds of times the atmospheric pressure, and the volume changes associated with the development or equalisation of the pressure difference can prove detrimental to engineering structures. The matric suction, that is the difference between the air and water pressures, is often abbreviated as s and is given by:

$$s = (u_a - u_w) \qquad [1.17]$$

As described in Section 1.1, the term negative pore water pressure $-u_w$ is equivalent to s when u_a in Equation 1.17 is atmospheric pressure and is adopted as the datum for pressure measurement.

Suction effects are generally associated with unsaturated soils and it is less well appreciated that the effect is also important for saturated soils. This has reduced the emphasis on the study and measurement of soil suction, and in design situations the soil is often assumed to be saturated with zero suction, though this may not necessarily be the case. Neglecting negative pore water pressure in assessing in situ soil strength usually represents a worst-case scenario for soils and allows for the possibilities of infiltration after rain and flooding. However, in many situations this is a simplification not reflective of site conditions.

The soil–water characteristic curve (SWCC) for a soil is the relationship between the water content and the soil suction (or negative pore water pressure). The water content variable can be defined in a number of ways: volumetric water content, gravimetric water content, degree of saturation or specific water volume, and is a measure of the amount of water in the soil pores. The amount of water can also be referenced to the residual water content as in Equation 1.18:

$$\Theta = \frac{\theta_w - \theta_r}{\theta_s - \theta_r} \qquad [1.18]$$

where Θ is the normalised volumetric water content, θ_r is the residual volumetric water content and θ_s is the saturation volumetric water content.

When the reference volumetric water content θ_r is taken as zero, $\Theta = \theta_w/\theta_s$. The normalised volumetric water content is used in Equations 4.16 and 4.17 to relate the strength of unsaturated soils to the SWCC.

In plotting the SWCC, the suction can be defined as either the matric suction $(u_a - u_w)$ or the total suction Ψ. At elevated suctions (e.g. >3000 kPa), the matric suction is, in practice, often significantly larger than the osmotic suction and assumed to be broadly equal to the total suction. The total suction corresponding to zero water content (close to 10^6 kPa) appears to be essentially the same for all soil types (Vanapelli *et al.*, 1996).

Figure 1.8 shows typical SWCC plots of volumetric water content θ_w against suction $s = (u_a - u_w)$ for a range of soils from Holland (Koorevaar *et al.*, 1983).

The shrinkage and swelling behaviour for London clay from the data of Croney and Coleman (1960) is presented in Figure 1.9. The plots are presented as negative pore water pressure to a log scale against specific volume v and specific water volume v_w (Murray
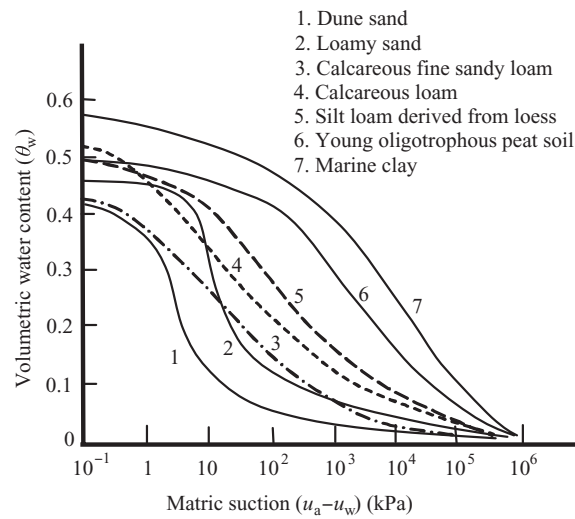
Figure 1.8 Soil–water characteristic curves (SWCC) for some Dutch soils (after Koorevaar *et al.*, 1983).
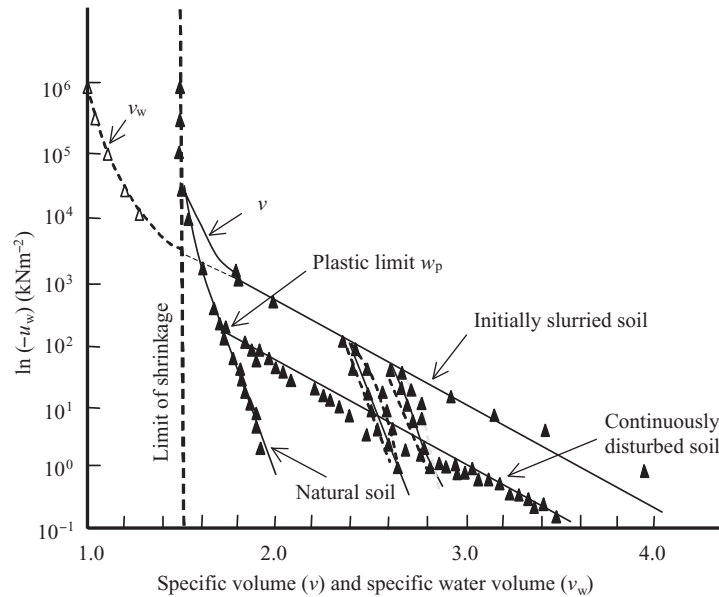

Figure 1.9 Specific volume v and specific water volume v_w against negative pore water pressure $-u_w$ to a log scale for London clay from the data of Croney and Coleman (1960) (after Murray and Geddes, 1995). © 2008 NRC Canadaor its licensors. Reproduced with permission.

and Geddes, 1995). Figure 1.9 shows plots for the soil dried from slurry. While saturated, the plots of v_w and v are the same and can be reasonably represented by a straight line up to a critical value of suction, known as the air entry value, where the soil ceases to be saturated. The data indicate that this change point is close to the plastic limit. The plot for v_w approaches a value $v_w = 1.0$ and v approaches the shrinkage limit of the soil as suction increases. Also shown in Figure 1.9 is a line for 'continuous disturbed soil' which represents the final suction on being sheared using a miniature vane apparatus. This again can be reasonably represented by a straight line parallel to the 'initially slurried line' for most of its length. Murray and Geddes (1995) argued that this represents the critical state strength line (*csl*) which forms an integral part of discussions in later chapters.

Mass constitutes 'energy' and the phases and their interactions in an unsaturated soil can be viewed as components of the soil energy, with changing conditions synonymous with exchange of energy. It is important to realise also that pressure is a manifestation of molecular activity, and thus the pressure of a liquid or gas in a closed system increases with temperature because of increase in molecular energy. Solids also generate large expansive or contractile forces due to temperature change though the shape of the solid mass is more rigidly defined. Consistent with this are the definitions by Aitchison (1965) of total suction as the free energy of the soil water, matric suction as the capillary component of free energy and osmotic suction as the solute component of free energy.

1.4.2 Water vapour in air

Vapour movement, in addition to mass water movement, is an important means of transport of water within an unsaturated soil and assists in the equalisation of water pressure. In laboratory tests in a closed system, vapour exchange assists in the unification of water pressure throughout a soil mass. Under equilibrium conditions the vapour pressure is defined as the saturated vapour pressure above the contractile skin. This is a special case not readily justifiable under normal conditions. At low degrees of saturation where continuous air passages in a soil are connected to the atmosphere, the vapour pressure will be less than the saturated vapour pressure and true equilibrium may not be established. There is always likely to be an imbalance and therefore ongoing mass transfer. Vapour pressure increases with temperature increase, decreases with increasing negative pore water pressure and is influenced by the solutes and the soil particles.

1.4.3 Contractile skin and air phase

A manifestation of the energy within the contractile skin is a surface tension effect that acts within a layer a few molecules thick. As illustrated in Figure 1.10, in three dimensions, equilibrium of the contractile skin is given by the following equation which includes a term for the vapour pressure u_v:

$$u_a + u_v - u_w = T_c \left[\frac{1}{R_1} + \frac{1}{R_2} \right] \qquad [1.19]$$

where R_1 and R_2 are the radii of curvature of the contractile skin for two perpendicular planes at right angles to the curvature of the interface.

As discussed by Childs (1969), the contractile skin may be wholly concave or a combination of concave and convex (in two orthogonal directions) to the higher pressure air.
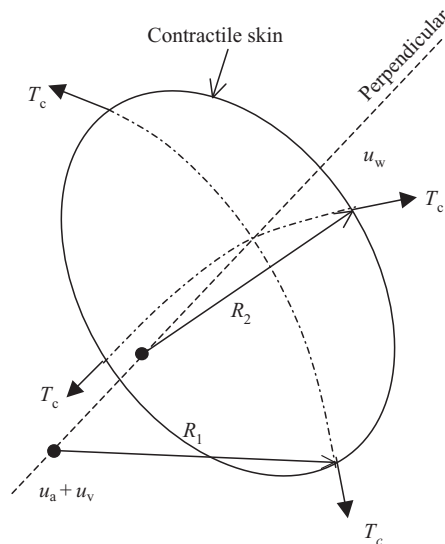
Figure 1.10   Contractile skin.

It is only necessary that the relative magnitudes of R_1 and R_2 are such so as to balance Equation 1.19. Indeed, where the contractile skin spans between collections of soil particles, dependent on the geometry, it appears necessary for continuity that the interface has a curvature both concave and convex to the air phase.

In idealised liquids containing no dissolved gases or impurities, it is the vapour pressure in relation to the liquid pressure that controls the development or the degradation of vapour bubbles. In natural systems, such as soils, there are dissolved gases and impurities in the liquid phase as well as the presence of the soil particles that influence the nucleation and bubble dynamics. Schuurman (1966) showed that for air–water mixtures the influence of the vapour pressure term u_v in Equation 1.15 is small and can be omitted for general purposes.

The idealised case of spherical occluded air bubbles ($R = R_1 = R_2$) does not represent a stable equilibrium condition and careful consideration needs to be given to the growth and decay of gas bubbles to appreciate their significance in the development of unsaturated conditions in soils. From Equation 1.19, for the case of a spherical bubble, ignoring the effect of the water vapour, the matric suction $s = (u_a - u_w)$ can be shown to be given by Equation 1.16, which is variously referred to as the Kelvin or Laplace equation. There is likely to be a relatively abrupt energy change in the formation of bubbles and the development of a contractile skin as the water ruptures. The growth and decay of gas bubbles in liquids is a dynamic process which can be described in terms of Kelvin's equation and the Rayleigh–Plesset equation (e.g. Keller, 1964; Brennen, 1995). A full treatment of this complex subject matter is beyond the scope of this text and we will deal with it in a descriptive manner in bringing out the significance to soils.

Nucleation is the seed to the development of bubbles. This can be divided into cavitation and boiling (Brennen, 1995). Cavitation is the process of nucleation in a liquid when the pressure falls below a critical value, and boiling is the process of nucleation that occurs when the temperature is raised above a critical value. Physically there is little difference between the two nucleation processes though the thermodynamic path followed

is different. In both cases dissolved gases and/or vapour form bubbles at nucleation points. The nucleation points may arise due to thermal motions within a liquid forming temporary microscopic voids that act as nuclei for the growth of bubbles; this is termed homogeneous nucleation. Heterogeneous nucleation is more common in practice where bubbles form spontaneously around nuclei such as a solid particle or an ion, or form on the surface of a containing vessel.

A bubble in the pore water in a soil can decay in size as the gas dissolves or grow if gas is liberated from the solution into the bubble. In discussing the general framework of bubble growth and decay it is important to distinguish between the different degrees of air saturation in water. In a saturated condition the free air pressure is taken as the pressure of the dissolved air, in a supersaturated condition the dissolved air pressure exceeds the free air pressure and in an undersaturated condition the dissolved air pressure is less than the free air pressure.

Consider the situation under confined conditions of water initially in an air-saturated condition. Decreasing the confining pressure will result in the water becoming supersaturated and the dissolved air will start to be more readily liberated, forming bubbles around nuclei. There must be a reduction in air pressure in the bubbles and of the dissolved air as air is liberated from the solution but the pressure inside the bubbles must exceed the water pressure outside the bubbles. For spherical bubbles, there is an equilibrium radius R_e based on Equation 1.16 which will be dictated by the equilibrium of the pressure of the dissolved air and the air in the bubbles. This represents an unstable equilibrium and any bubbles smaller than R_e will tend to decay, as the pressure inside the bubbles will be greater than the dissolved air pressure. Conversely, any bubbles with radius greater than R_e will tend to expand, as the pressure will be less than the dissolved air pressure.

In a supersaturated closed water system with no soil particles, it is necessary for continuity to conjecture the tendency for the free air to form in a single bubble[4] that gradually approaches equilibrium. This bubble is found at the top of a container of gassy liquid. In support of this, Keller (1964) points to the observation of the Russian astronaut Colonel Nikolayev, who while orbiting around the world in a satellite reported that in a closed bottle containing liquid and gas 'the gas formed a single spherical bubble near the centre of the bottle'. Another example is the cavitation in the water back-pressure leads of triaxial testing equipment. Bubbles form if the water pressure is too low and the bubbles gradually combine to form large isolated bubbles[5]. For a water–air solution under reduced confining pressure, where there is no influence from the sides of the container the dissolved air pressure reduces more rapidly than the free air pressure and equilibrium (an air-saturated condition) is gradually approached with a single bubble present. This is consistent with the minimisation of the thermodynamic potential as discussed later in the book.

In soils the situation is more complicated because of the soil particles. Consider again the case of decreasing external pressure under confined conditions. Under decreasing external pressure, the bubbles will expand until they achieve apparent equilibrium within the confines of the pore spaces. Within these spaces the bubbles will interact with the soil particles, with which they are likely to have a close affinity, and will be influenced by the adsorbed double layer and gravity, and are unlikely to be spherical. Under decreasing confining pressure these air 'pockets' will have a tendency to expand and link with others to form larger pockets until eventually continuous air passages are established.

Under the alternative of increasing confining pressure from an equilibrium condition, the water will be undersaturated, as the free air will have a pressure greater than the dissolved air. The free air will tend to dissolve to equalise conditions, although again this will be influenced by the presence of the soil particles.

The rate of expansion and decay of the volume of free air is likely to influence the perceived equilibrium conditions. Barden and Sides (1967) concluded that in unsaturated soils there is evidence that equilibrium in terms of Henry's law[6] may require a considerable time interval, far greater than in the absence of soil particles.

During wetting and drying of a soil, hysteresis can be expected: when a soil is wetted, surface tension forces resist the expansion of a soil; when a soil is dried surface tension forces attempt to keep as many voids filled with water as possible. During wetting of a soil, partly because of the variation in pore dimensions, the water reluctantly fills the void spaces. Conversely, during drying of a soil the water resists being drawn into the smaller pore spaces. This hydraulic hysteresis is discussed further in Sections 4.7.4 and 7.10.

1.4.4 Air phase and dissolved air in water

In most analyses and tests, other than where fines are removed during fluid flow, the volume of solid particles remains unchanged. It is the volume of water and air in the voids that changes and gives rise to overall volume change. Water is usually considered incompressible and the volume of water change is due to exchange with the surrounding environment, although a small volume change may result from changes in water vapour in the air.

Air cannot be said to be incompressible and the change in the volume of air within a soil can be a result of the following effects:

- Interchange of air with the external surroundings;
- Compression or expansion of the free air in the pores;
- Air being dissolved or liberated from the pore water.

Water molecules form a lattice-like structure with the dissolved gases occupying the spaces within the lattice. The volume available for dissolved air is approximately 2% of the total volume. Water is of very low compressibility and at equilibrium, when in contact with free air, the volume of air dissolved in water is essentially independent of air and water pressures. This can be demonstrated by an examination of the ideal gas laws and Henry's law (Fredlund and Rahardjo, 1993). It is the liberation of the dissolved air under decreasing external pressure and corresponding increasing suction that leads to the formation of air bubbles. As shown by Dorsey (1940), however, dissolved air produces an insignificant difference between the compressibility of de-aired water and air-saturated water.

Strictly speaking, the dissolved and undissolved gases in a soil may comprise mixtures of gases at odds with the proportions of gases normally found in free air. In fact, monitoring of ground gases rarely indicates gas concentrations in the soil, precisely those in free air. Even well away from sites influenced by factors such as landfills, elevated carbon dioxide and depleted oxygen are frequently detected. The ground gases are in a more restrained state than free air and segregation of gases and the influence of soil chemical reactions can give rise to gas concentrations far removed from a breathable environment. We will assume, however, that this does not influence the general conclusions in the following and when reference is made to dissolved and liberated gases that this is air and the terms gases and air are interchangeable.

1.4.5 Adsorbed double layer, crystalline swelling and soil particle interactions

Clay particles less than around 1 μm are often referred to as colloids. Colloids are small particles with irregular plate-like geometry, the behaviour of which is influenced greatly by surface forces. The surface area of clay particles for a given mass is large and the smaller the particles, the larger the *specific surface* (surface area per unit mass) and the influence of surface forces. Kaolinite has a specific surface of around 10–20 m^2/g and the influence of surface forces is significantly less than that of montmorillonite with a specific surface of around 800 m^2/g (Lambe and Whitman, 1969). Consequently, kaolinite particles are far less water-sensitive than the finer montmorillonite particles. The surface of clay particles carries a small electrical charge, generally considered to be negative, which depends on the soil mineral and may be affected by an electrolyte in the pore water. The charge gives rise to forces between fine soil particles in addition to self-weight effects. The charge is balanced by exchangeable ions on the particle surface, which along with the soil particle surface attracts water into the so-called adsorbed double layer. Under equilibrium conditions, isolated particles under these conditions have no net charge. However, the surface charge comes into play when:

- The double layers of particles come into contact;
- There is a reduction of the cations within the adsorbed layer, as would be expected when air comes into contact with the double layer or there is a change in pore fluid chemistry.

In these cases a net repulsive force exists. The particles also experience attractive forces, notably van der Waals force, which affects all adjacent pieces of matter and is essentially independent of the characteristics of the fluid between the particles (Sposito, 1989). The repulsive and attractive forces are of significance and play a role in the strength of soils. The physical interpretation of the stresses in fine-grained particulate soils is closely related to the stresses transmitted through the particles, but is not merely the contact stresses between the particles. Indeed in highly plastic, saturated, dispersed clay there may be little or no physical contact between the particles (Lambe and Whitman, 1969).

As a soil compresses or expands, the particles move closer together or further apart and this influences the 'effective stress[7]', which is a function of the resulting interactions. It is this stress that controls the strength and compressibility of a soil. The 'effective' stress is a net stress that includes the influence of the contact stresses and other interactions between the particles and the adsorbed water. In an unsaturated soil, where there are menisci between particles, there must also be interaction between the contractile skin and the adsorbed double layer and a reduction in the thickness of the double layer. This is unlikely to be a static environment, though a quasi-static equilibrium between surface tension forces, water vapour and the adsorbed water is likely to be established with time.

With a decrease in moisture content, however, there will be an increase in the repulsive forces between closely spaced particles, which along with the influence of the contractile skin can be perceived as helping to propagate air-filled void spaces and the creation of aggregated packets containing the soil particles and water.

While all clay soils can be expected to change volume on change of water content, and thus suction, the term *swelling clay* is often used to describe those clays containing a relatively high proportion of clay minerals with the potential for significant crystalline swelling. This is particularly evident with the colloidal clay minerals of the smectite group, which includes montmorillonite, that exhibit high activity (see Section 1.3). Increasing
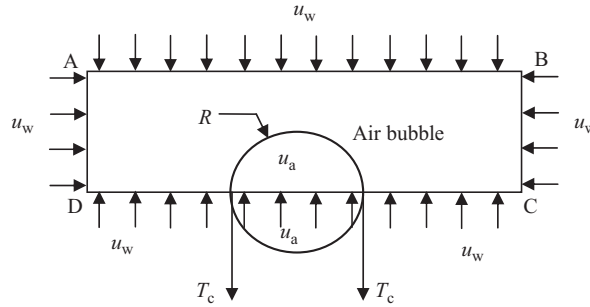
Figure 1.11 Equilibrium of element ABCD with air bubble.

suction draws water from the hydration layers of the clay minerals and the interlayer spaces contract resulting in particle shrinkage in addition to reduction in the void spaces between particles. Reducing suction reverses the phenomenon but hysteresis in volume change can be expected to occur. The change in particle size due to variation in water content obviously influences the mechanical response of swelling clays to external loading. Fityus and Buzzi (2009) have reviewed the place of expansive clays in the framework of unsaturated soil mechanics.

1.4.6 Soil particles and surrounding fluid pressure

All soil particles are surrounded by fluid(s), usually water and air. In a saturated soil, water is the only fluid and the water pressure acts not just through the volume of the water, but also through the volume of the soil particles. This important point is often missed in an analysis and we make no excuse for explaining this simple fact in detail. First, however, consider the spherical air bubble of radius R in Figure 1.11 as it is useful in understanding the significance of fluid pressure on a soil particle. The air bubble is intersected by the small element ABCD with horizontal plane area A_p at right angles to the plane of the paper. The element is considered to be sufficiently small that the influence of the change of gravitational field from top to bottom can be ignored, which is a reasonable assumption for small bubbles. For vertical equilibrium of the element, equating those forces acting downwards to those acting upwards:

$$u_w A_p + 2\pi R T_c = u_w (A_p - \pi R^2) + u_a \pi R^2 \qquad [1.20]$$

This reduces to Equation 1.16 for a spherical air bubble as A_p tends to πR^2.

Now consider the case of a soil particle of cross-sectional area A_s intersected by the small element ABCD in Figure 1.12. For vertical equilibrium of the element:

$$u_w A_p = u_w (A_p - A_s) + u_s A_s \qquad [1.21]$$

where u_s is the pressure in the soil particle. As A_p tends to A_s, u_w tends to u_s, indicating that the water pressure acts through the soil particle. This is taken into account in Section 1.4.7 to establish Terzaghi's equation of effective stress.

Gravity dictates that the air bubble in Figure 1.11, being less dense than water, rises through the water unless acted upon by other forces. In a soil, the confining spaces between particles and the interaction of air bubble with the particles inhibit bubble migration. For
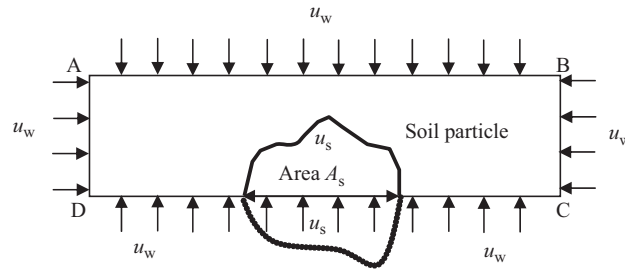
Figure 1.12 Equilibrium of element ABCD with soil particle.

the soil particle in Figure 1.12, gravity dictates that the particle will sink if unrestrained and denser than water. This is the converse of the behaviour of the air bubble. Both the air bubble and solid particle would move to minimise of the thermodynamic potential if free to do so.

1.4.7 'Effective' stress and stress state variables

In any analysis on a flat plane cut through a soil, the plane must cut through soil particles. In a saturated soil, the water pressure acting through the soil particles must be taken into account. Consider Figure 1.13 where the plane AB of area A_p cuts through soil particles and water.

   The total upwards force is given by the total stress σ acting over the area A_p. This is balanced by the water pressure, acting through the volume of water and soil particles, and the effective stress. The effective stress is a consequence of the soil particle interactions and acts over the total planar area consistent with the definition of effective stress. Thus, the balance of forces gives $\sigma A_p = u_w A_w + u_w A_s + \sigma' A_p$. Since $A_p = A_w + A_s$, this reduces to Terzaghi's effective stress equation for saturated soil:

$$\sigma' = \sigma - u_w \qquad [1.22]$$

Equation 1.22 does not emerge from the analysis unless the water pressure acting through the volume of the soil particles is taken into account. An alternative presentation can be found in some textbooks and assumes that points of contact between particles are small and analysis is made of equilibrium on a 'wavy' surface passing through only the points of contact. In this case the water pressure acts over the total area of the wavy plane and
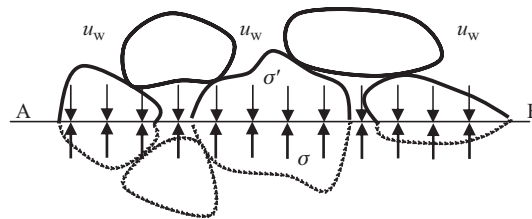

Figure 1.13 Equilibrium on plane AB cut through a saturated soil.

$\sigma A_p = u_w A_p + \sigma' A_p$. This again reduces to Equation 1.17, but the analysis masks the influence of the fluid pressure (in this case water) acting through the soil particles, which is important in the analysis of unsaturated soils presented in Chapter 8.

In a perfectly dry soil the surrounding fluid is air and it is the air pressure that acts through the soil particles. The equivalent effective stress (net stress) for a perfectly dry soil is given by:

$$\bar{\sigma} = \sigma - u_a \qquad [1.23]$$

This assumes no 'bonding' between soil grains and no influence from soil suction.

In unsaturated soils the situation is more complex because two fluids are present. In Chapter 5 current thinking around the use of independent stress state variables for unsaturated soils is discussed. The stress state variables are defined as the two, effective, stresses $(\sigma - u_a)$ and $(\sigma - u_w)$, along with the matric suction $(u_a - u_w)$. The suction variable is the difference between the other two stress state variables and thus only two of the variables can be considered independent.

We will deal with the state of the soil under triaxial stress conditions in later chapters but it is appropriate here to define the mean stresses. A schematic of triaxial stress conditions is presented in Figure 1.14. In the apparatus the total axial stress is given by the principal stress σ_{11} and the total radial or lateral stress (the cell pressure acting all around the soil
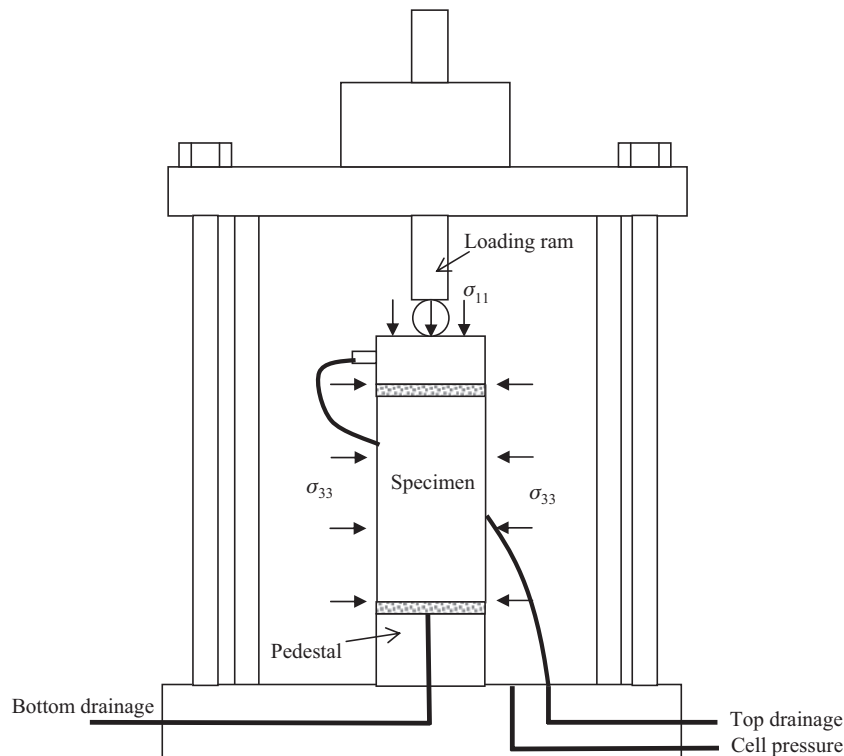

Figure 1.14   Schematic of a triaxial cell.

specimen) is given by the principal stress σ_{33}. Thus, for a saturated soil:

$$p' = p - u_{\mathrm{w}} \qquad\qquad [1.24]$$

and for a perfectly dry soil:

$$\overline{p} = p - u_{\mathrm{a}} \qquad\qquad [1.25]$$

where $p = \frac{1}{3}(\sigma_{11} + 2\sigma_{33})$ is the mean total stress, $p' = \frac{1}{3}(\sigma'_{11} + 2\sigma'_{33})$ is the mean effective stress, $\overline{p} = \frac{1}{3}(\overline{O}_{11} + 2\overline{O}_{33})$ is the mean net stress, $\overline{O}_{11}$ is the axial net principal stress and $\overline{O}_{33}$ is the radial net principal stress. σ'_{11} is the axial effective principal stress, σ'_{33} is the radial effective principal stress.

1.5 Soil structure

The fabric or structure of soil is derived from the arrangement of the particles and the presence of mass structural effects. Soil fabric significantly influences the mechanical and hydromechanical behaviour (Casagrande, 1932; Lambe, 1951; Leroueil and Vaughan, 1990; Mitchell, 1993). Structure is particularly important in fine-grained clayey soils, which comprise an abundance of fine, plate-like particles. Though also influential in coarser-grained sands and gravels, the effects are less pronounced. The presence of mass discontinuities such as fissures, laminations and bedding planes form part of the macro-fabric of a soil and are a consequence of deposition and past stress history. Such characteristics significantly influence behaviour in the field, not least the permeability (Hossain, 1992; Leroueil *et al.*, 1992; Little *et al.*, 1992). Small specimens of either natural soil or compacted soil may not contain a representative distribution of the macro-features, and the structural hierarchy in situ is rarely as uniform as in laboratory-prepared specimens. It is important to recognise this in transferring laboratory results to the prediction of behaviour in the field. However, the micro-fabric, or distribution of the soil particles, also plays a significant part in soil behaviour and is usually considered adequately represented in small-scale laboratory specimens.

The early descriptions of the structure of compacted clays was based on individual particles with the structure dry of OWC described as flocculated and the structure wet of OWC described as more dispersed with a face-to-face, parallel particle arrangement (Lambe, 1951). The generalised horizontal alignment of clay platelets in soil compacted wet of optimum is consistent with the description given by Lambe (1951) of a dispersed structure. However, an ordered soil structure of this kind can also result from natural deposition or be a consequence of particle reorientation on deformation planes. In fine-grained cohesive soil, preferential orientation of clay platelets leads to pronounced anisotropic behaviour characteristics. At right angles to the direction of particle alignment the permeability can be expected to be several orders of magnitude less than the permeability in the direction of particle alignment because of the greater tortuosity of fluid flow (Lambe, 1955; Witt and Brauns, 1983; Arch *et al.*, 1993). Strength, deformation and consolidation characteristics can also be expected to be influenced by particle alignment. It may be that research into fractal geometry is able to shed further light on the distribution of different levels of soil structure.

The importance of soil structure and its dependence, amongst other factors, on water content, stress history and method of compaction have generated much research, with progress through the years relying on advances in technology. While there are several methods available to examine the pore size distribution of soils, the most productive have

been the scanning electron microscope (SEM) and mercury intrusion porosimetry (MIP). These techniques are outlined in Section 1.6. Interpretation of the structure in unsaturated soils dry of OWC is now more refined and takes account of the aggregation of soil particles. Aggregation leads to a soil with a bi-modal pore size distribution comprising relatively small intra-aggregate void spaces within the particle aggregations and larger inter-aggregate void spaces between the aggregates (Barden and Sides, 1970; Ahmed *et al.*, 1974; Wan *et al.*, 1990; Alonso *et al.*, 1995; Gens *et al.*, 1995; Delage *et al.*, 1996; Lloret *et al.*, 2003; Romero *et al.*, 2003; Thom *et al.*, 2007; Sivakumar *et al.*, 2010b).

1.6 Experimental techniques for examining pore size distribution

The importance of soil fabric to behaviour cannot be overemphasised. Consequently, techniques that allow an examination of the microstructural arrangement of soil particles, soil structural features such as strain localisation and a determination of the variation of pore sizes within a soil are fundamental to characterising soil behaviour. The SEM allows a visual image of the structure of fine-grained soils to be obtained and is the most widely used surface diagnostic tool. Much research has also centred on MIP, which allows measurement of the pore size distribution and clearly illustrates a bi-modal distribution of pore sizes in most fine-grained unsaturated soils. There continue to be advancements in these approaches as well as the introduction of newer techniques such as the tomography methods: X-ray computed tomography, neutron tomography and electrical impedance tomography. These approaches are briefly discussed in the following sections. Other techniques referenced in the literature – time-domain reflectometry, near-infrared spectroscopy, the dual-energy gamma-ray technique, along with more powerful X-ray techniques provided by synchrotron radiation and micro-focus X-ray systems – are also finding increasing usage (see Romero and Simms, 2008).

1.6.1 SEM method

This technique employs a narrowly focused beam of high-energy electrons to scan across the surface of a specimen. The beam interaction with the surface generates a shower of secondary and backscattered electrons which are collected by a detector. The intensity of the emitted electrons varies according to differences in surface topography and/or material composition. A flat specimen is usually needed in order to discriminate between the topographic information and the compositional information, as well as phase identification. The specimens are carefully glued into plugs using epoxy or silpaste and are dried overnight in a low-temperature oven. The specimens are then coated with a conductive metal, normally gold, and placed inside the SEM unit from which images are obtained at the required magnifications.

Romero and Simms (2008) described the environmental scanning electron microscope (ESEM) technique, which is a promising advancement from SEM testing. The approach requires no conductive coating on the specimen, making it possible to examine wet soil specimens and to preserve the natural specimen characteristics. Amongst others who have reported testing using ESEM are Komine and Ogata (1999) and Villar and Lloret (2001), who have reported on the swelling characteristics of bentonite, and Monroy *et al.* (2010), who have reported on the structure of London clay.

1.6.2 MIP method

MIP is perhaps the most widely used method for measuring pore sizes of a soil or rock fragment. The general procedure for MIP was first suggested by Washburn (1921). Normally, mercury does not intrude pores in a specimen because the free energy of the soil–mercury interface is greater than the free energy of the soil–gas interface. However, the application of pressure can force mercury into the pores of a soil. The differential pressure required to intrude the pores is given by:

$$p_d = \frac{-4 T_m \cos \theta_m}{d} \qquad [1.26]$$

where p_d is the differential pressure, T_m is the surface tension of mercury (0.484 N/m at 25°C), d is the pore diameter, θ_m is the contact angle of the air-mercury with the walls of the capillary (usually taken as between 139° and 147° (Diamond, 1970), although Penumandu and Dean (2000) suggest higher values of 162° for the advancing angle and 158° for the receding angle for kaolin using the sessile drop technique).

The use of mercury porosimetry requires that all of the pores be free of interfering liquids at the start of the intrusion measurement (Sridharan *et al.*, 1971). The interfering liquids can be removed by either air drying or oven drying. However, in the case of soils, these approaches can lead to soil shrinkage (Diamond, 1970). In order to overcome the problem, freeze drying is used to remove the water by sublimation and desorption (Ahmed *et al.*, 1974; Delage *et al.*, 1996). Romero and Simms (2008) provide further discussion on the methodology and the use of MIP.

1.6.3 Tomography methods

Sun *et al.* (2004) described the development and use of real-time, X-ray computed tomography to the observation of density changes in both soils and rocks, and reported results from a conventional triaxial compression test on silty clay. This non-destructive method was used to provide cross-sectional images of the attenuation of an X-ray beam through the soil specimen, as the specimen was sheared under increasing deviator stress up to and beyond peak strength. The images provided evidence of the stages in development of defects from initial specimen conditions through to post-failure. The cross-sectional images appear to indicate the development of, or tendency towards, a complex pattern of fissuring, interpreted as radial, axial fissures, though more laterally inclined fissures are unlikely to be adequately represented on the images.

Koliji *et al.* (2006) described the use of neutron tomography followed by image processing to measure the macro-pore evolution during compaction. Neutron tomography is a non-destructive technique for investigating the distribution of neutron attenuating materials (Degueldre *et al.*, 1996). Using the principles of computer tomography, a 3-D reconstruction of a soil specimen from a set of radiographies can be obtained by rotating the specimen.

Electrical impedance tomography can be used in the laboratory as well as in geophysical investigations in the field. The technique allows the estimation of the spatial distribution of the electrical conductivity within a soil to be determined from impedance measurements. Electrodes are applied to the surface of the soil and electric currents applied to selected electrodes while the resulting voltage is measured at the remaining electrodes. The distribution of conductivity within the specimen can then be estimated. Abu-Hassanein *et al.* (1996) and Borsic *et al.* (2005) described the use of electrical conductivity measurements

in the laboratory to determine heterogeneities in soil properties such as porosity, degree of saturation and hydraulic conductivity.

1.7 Pore size distribution

There is a considerable body of research identifying the bi-modal pore size distribution of unsaturated fine-grained soils based primarily on SEM and MIP analyses of laboratory-prepared specimens. It is now widely accepted that the simplest representation of un-saturated soils is given by two levels of structure (Diamond, 1970; Ahmed *et al.*, 1974; Gens and Alonso, 1992; Al-Mukhtar, 1995; Al-Mukhtar *et al.*, 1996; Delage *et al.*, 1996; Cuisinier and Laloui, 2004; Koliji *et al.*, 2006). Figure 1.15 illustrates a compacted soil that possesses a bi-modal pore size distribution as a result of aggregation of the soil parti-cles. Within the aggregates are the fine intra-aggregate pores, while between the aggregates are the larger inter-aggregate pores. The general conclusion from analysis of specimens compacted wet and dry of Proctor OWC is of an aggregated soil structure dry of optimum and a more dispersed, unimodal pore size distribution wet of optimum.

The dual-porosity model is a result of the micro-structural arrangement of the soil parti-cles within the aggregations, and the macro-structural arrangement of the aggregates and their relation to mass characteristics. Within both natural and compacted clayey soils dry of optimum, the aggregation of soil particles results in what is often referred to as an 'open' soil structure. The drier a soil, the more likely the inter-aggregate pore spaces will develop into open fissures. Griffiths and Joshi (1989) found that volume change of unsaturated soils under increasing consolidation stress was due to deformation of inter-aggregate pore spaces and that intra-aggregate pore spaces remained largely unchanged. Gens and Alonso (1992) reached a similar conclusion from an examination of the deformation character-istics of unsaturated expansive clay, while Toll (1990) recognised the significance of an aggregated structure to the strength of unsaturated soils.

As a soil dries, the water phase is drawn into the smaller intra-aggregate pores and the suction increases, while the air phase fills the surrounding larger inter-aggregate void spaces (Alonso *et al.*, 1990; Delage and Graham, 1995). The concept of a bi-modal structure and the dual stress regime implied by this model is used in examining the strength and compression behaviour of unsaturated soils in later chapters. Consistent with this, in describing unsaturated soils, Barden and Sides (1970) carried out a series of SEM tests and referred to the creation of saturated packets, comprising the soil particles and water, surrounded by air-filled voids. While this represents an idealised view of unsaturated soils, theoretical justification for the close association of the water and soil particles is presented in Chapters 7 and 8. There is also increasing experimental evidence to support the view that saturated aggregates of particles provide a reasonable basis for
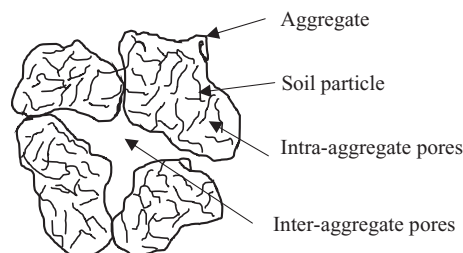

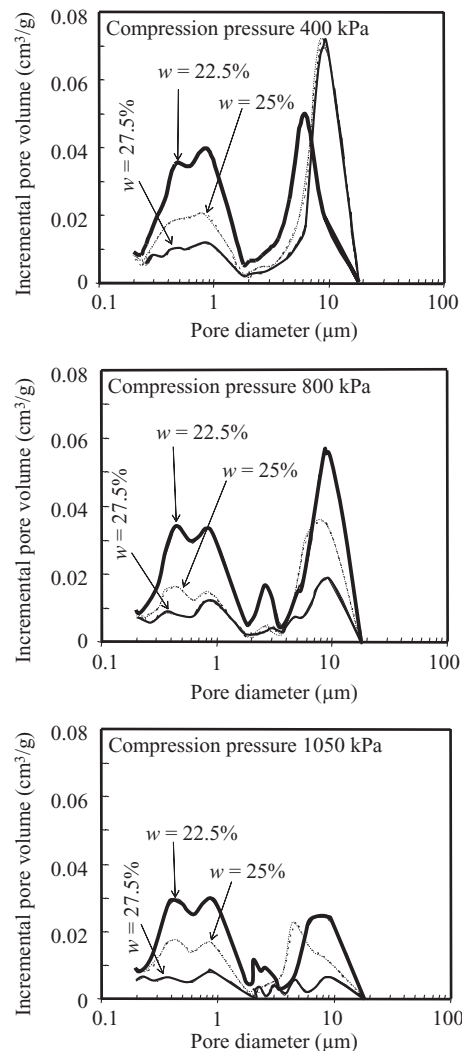**Figure 1.15**   Illustration of aggregated soil structure.

Figure 1.16 Pore size distribution of specimens of kaolin isotropically compressed to various pressures at water contents of 22.5, 25 and 27.5% (after Thom *et al.*, 2007).

the interpretation of the structure of unsaturated soils over a wide range of suction. Tang and Cui (2009) made this assumption in their analysis, arguing that it was supported by the experimental observations of Saiyouri *et al.* (2000). Monroy *et al.* (2010) described a bi-modal structure for unsaturated London clay in which measurements indicated the aggregates were saturated. Further justification is given by the MIP analyses discussed later in the chapter.

Thom *et al.* (2007) reported on the results of an investigation using MIP into the effect of post-compaction wetting, compaction water content, method of compaction and compactive effort on the existence of a bi-modal pore distribution in unsaturated specimens of speswhite kaolin. Figures 1.16–1.18 illustrate the pore size distributions based on the
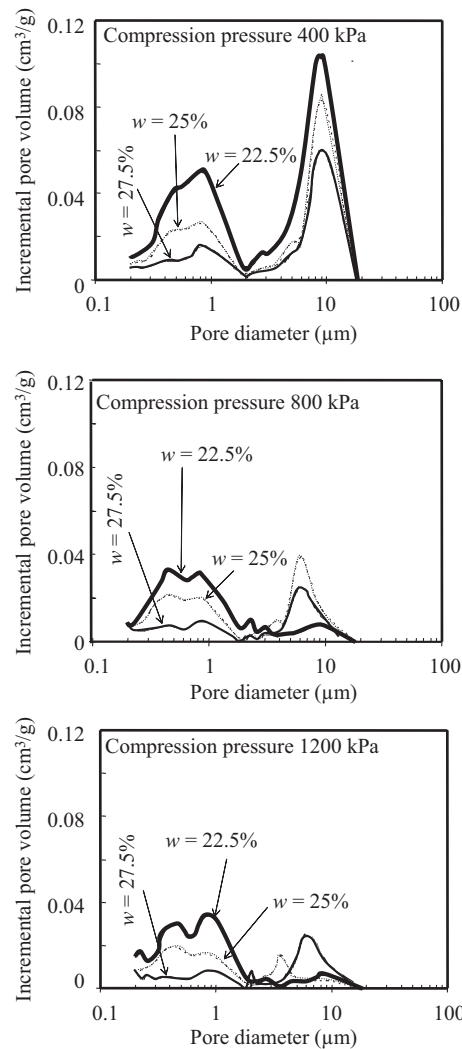
Figure 1.17   Pore size distribution of specimens of kaolin statically compressed to various pressures at water contents of 22.5, 25 and 27.5% (after Thom *et al.*, 2007).

MIP tests on specimens of kaolin isotropically compressed, statically compressed and dynamically compacted at water contents of 22.5, 25 and 27.5%. The plots indicate the incremental pore volume for 1 g of dry kaolin plotted against the pore diameter. Incremental pore volume refers to the volume of mercury that intruded into the pores between pressure increments. Irrespective of the method of specimen preparation, there is clear indication that the kaolin specimens possessed a bi-modal pore size distribution. The SEM image in Figure 1.19, taken of one of the specimens isotropically compressed, provides a pictorial illustration.

   The plots of Figures 1.16–1.18 indicate a distinct division between those smaller voids, interpreted as comprising the intra-aggregate pore spaces, and the larger inter-aggregate voids. The intra-aggregate voids are shown as typically between 0.3 and 2 μm, with the
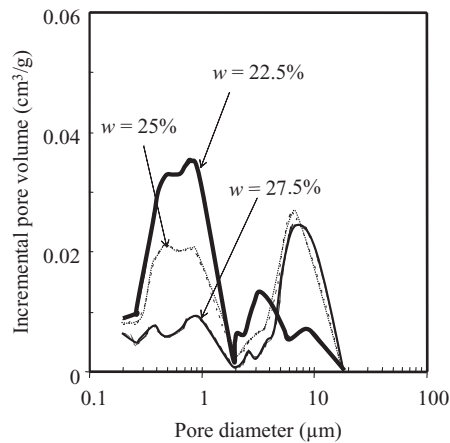
Figure 1.18 Pore size distribution of specimens of kaolin dynamically compacted at water contents of 22.5, 25 and 27.5% (after Thom *et al.*, 2007).

larger inter-aggregate voids >3 μm and typically around 10 μm. The difference between the two sets of void spaces of 2–3 μm corresponds approximately to the particle size of kaolinite. The inter-aggregate voids are shown as larger than those voids within the aggregations by around 10–20 times. In general, the plots indicate that the lower the compaction water content, the greater the percentage of the intra-aggregate pores and the lower the percentage of the inter-aggregate pores. However, a word of caution is necessary as the relative volume of inter-aggregate pore spaces for very dry soils may be influenced in the field by macro-fissures not being represented in the small specimens tested in the laboratory.

The plots clearly indicate that the form of compression and the water content at the time of compression influenced the determined pore size distribution of the unsaturated kaolin. For given water content, the increase in compression pressure in Figures 1.16 and 1.17 is shown to have had a far more pronounced influence on the inter-aggregate pores than on
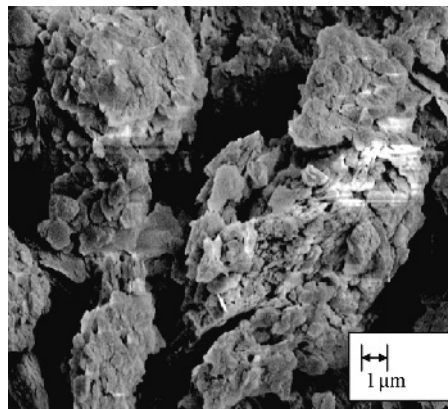

Figure 1.19 SEM image of specimen of compacted kaolin (static compression to 800 kPa at water content of 25%) (after Thom *et al.*, 2007).
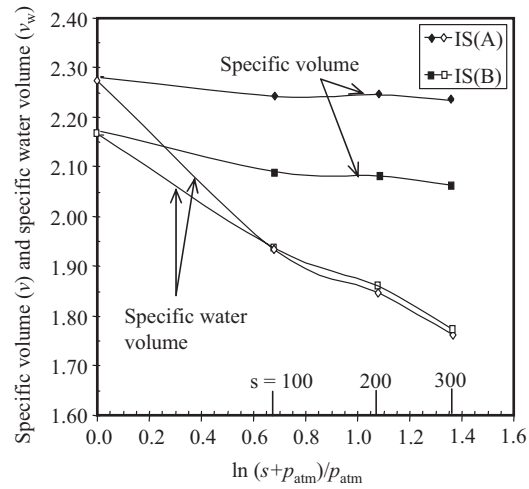
Figure 1.20   Suction-volume and suction-water volume relationship (after Sivakumar *et al.*, 2010b).

the intra-aggregate pores. This is consistent with the findings of Griffiths and Joshi (1989) and Gens and Alonso (1992).

It is also important to note that the bi-modal structure was not broken down by compaction even at water contents close to the optimum of 29% (based on standard Proctor compaction).

Figures 1.20 and 1.21 after Sivakumar *et al.* (2010b) and Sivakumar (2005) respectively present the results of swelling tests on specimens of kaolin. The specimens, IS(A) and IS(B), were prepared isotropically to two different initial specific volumes (IS(A) at
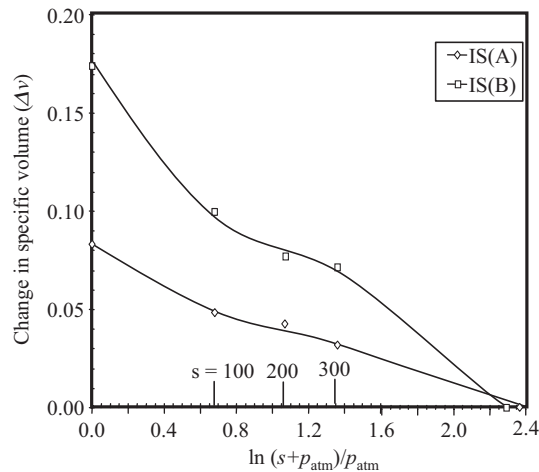

Figure 1.21   Wetting-induced volume change of specimens IS(A) and IS(B) (after Sivakumar, 2005).
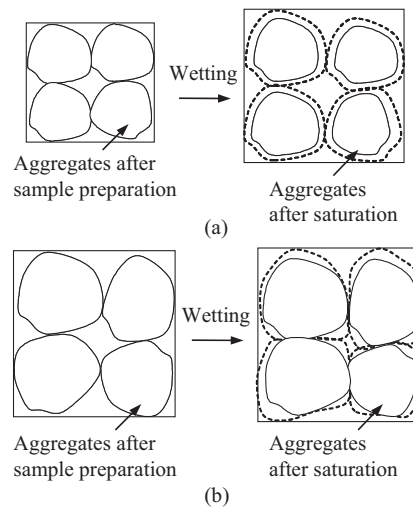
Figure 1.22 (a) Simple model and (b) actual model for aggregates expanding into macro-voids (after Sivakumar *et al.*, 2010b).

2.19 and IS(B) more compact at 1.99), but to similar specific water volumes corresponding to a water content of 25%. The method of isotropic soil preparation is detailed in Chapter 3. Figure 1.20 shows the specific volume and specific water volume against suction, and Figure 1.21 shows the change in specific volume against suction of the specimens from the initially prepared conditions. The inclusion of p_{atm} (atmospheric pressure) in the suction term avoids the natural logarithm of suction becoming indeterminate when the suction falls to zero. The results for specimens of both IS(A) and IS(B) show that significant swelling occurred during the wetting process, but the more compact specimens of IS(B) swelled twice as much as those of IS(A). The simple scenario of swelling of the clay specimens generated solely by the mechanism of expansion of the aggregates is illustrated in Figure 1.22(a). If expansion of the aggregates was the only mechanism for volume change, the volume change for specimens IS(A) and IS(B) would have been expected to be broadly similar. However, this was not the case and the wetting tests on kaolin confirm expansion of the individual aggregates into the inter-aggregate void spaces on uptake of water by the aggregates, reducing the overall potential expansion of a specimen (Thom *et al.*, 2007; Sivakumar *et al.*, 2010b). The conclusion is that the greater the initial compaction, the denser the specimen, the less the closure of the inter-aggregate voids and the greater the overall specimen expansion for a given increase in water content. The MIP results of Figure 1.24 indicate a significant reduction in the inter-aggregate pore volume. This can be explained only by inferred distortion and expansion of the aggregates into the inter-aggregate pore spaces during wetting, as illustrated in Figure 1.22(b). The aggregates in the tests thus acted as deformable structures in which particle rearrangement took place under changing conditions.

According to Alonso *et al.* (1995) and Sivakumar *et al.* (2006b), there are three mechanisms involved in the volume change characteristics of unsaturated soils: (a) swelling of individual aggregates due to water uptake; (b) aggregate slippage at the inter-aggregate contacts due to lack of strength to support the externally applied load, leading to collapse settlement; and (c) distortion of aggregates into the inter-aggregate pore spaces. The
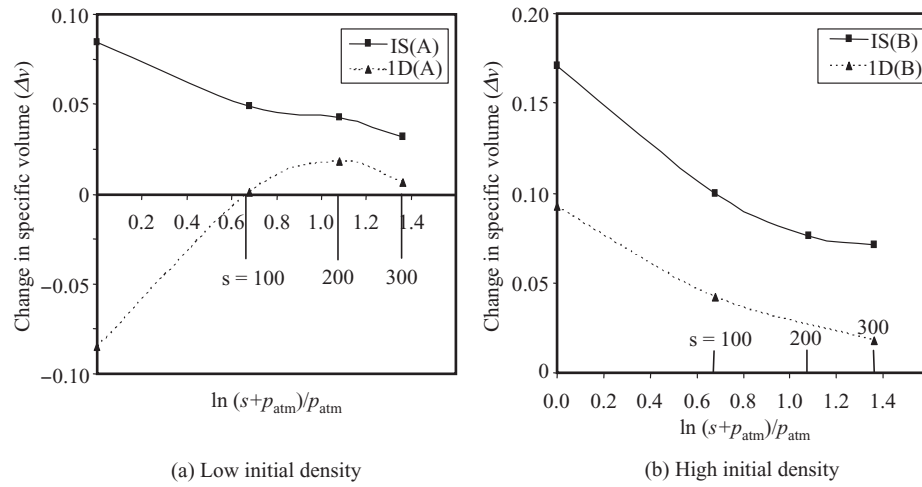
Figure 1.23 Effects of stress-induced anisotropy on the specific volume during wetting (after Sivakumar *et al.*, 2010b).

relative influence of these controlling mechanism determines the overall response of clays during wetting.

Further important insight can be gained into the aggregated structure of kaolin from the wetting results of Figure 1.23(a) and (b) and the corresponding MIP results of Figure 1.24(a) and (b). Cumulative mercury intrusion measurements for the specimens before wetting indicated that the volumes of mercury necessary to fill the inter-aggregate voids were approximately 0.18 cm^3/g and 0.125 cm^3/g for specimens IS(A) and IS(B) respectively. These volumes correspond to a macro-voids ratio of approximately 0.493 and 0.331 for IS(A) and IS(B). Since the respective overall voids
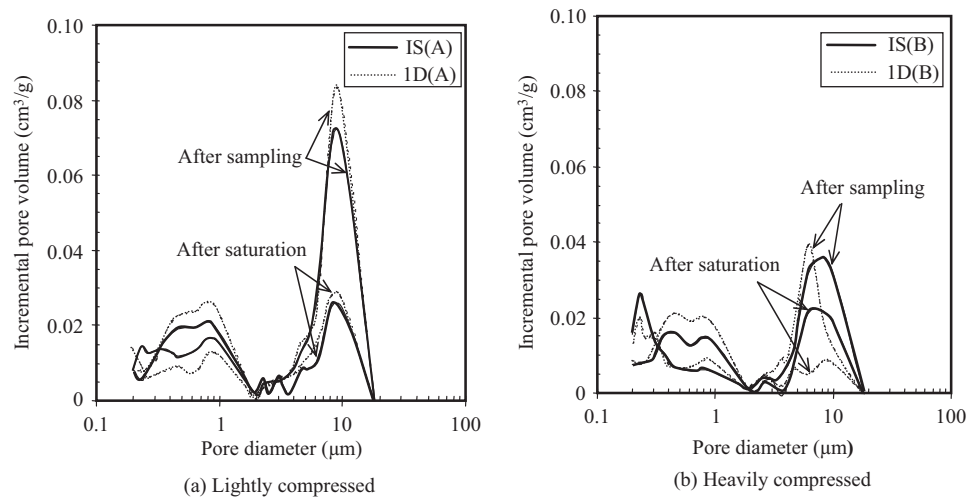

Figure 1.24 Effects of stress-induced anisotropy on the pore size distribution (after Sivakumar *et al.*, 2010b).

ratios of these specimens were 1.193 and 0.989, the magnitude of intra-aggregate voids ratios of the specimens of IS(A) and IS(B) were 0.700 and 0.658 respectively. The kaolin aggregates were prepared at a water content of 25%. If the aggregates were saturated and the inter-aggregates pores free of water, then, taking the specific gravity of kaolin as 2.65, the intra-aggregate voids ratios of the specimens would be approximately 0.663. This value agrees favourably with the measured intra-aggregate voids ratio from MIP results of IS(A) and IS(B), confirming the existence of saturated aggregates for the specimens with suctions up to at least 1000 kPa.

Figure 1.23(a) and (b) compares the changes in specific volumes from initial conditions of isotropically compressed and one-dimensionally compressed specimens subject to reductions in suction on wetting. The IS(A) and 1D(A) specimens of Figure 1.23(a) had an initial specific volume of 2.19 and the IS(B) and 1D(B) specimens of Figure 1.23(b) an initial specific volume of 1.99. Although the paired specimens had identical initial specific volumes and water volumes, the differences in the responses of the specimens to wetting (i.e. the reductions in suction) are shown as significant. In particular, the results for the specimen with isotropic previous stress history and relatively low initial density IS(A) show significant swelling during the wetting process, whereas the paired specimen with one-dimensional previous stress history 1D(A) exhibits significant collapse compression on reduction in suction from 200 kPa to zero. The collapse compression was a result of reduction of the larger inter-aggregate voids associated with a reduction in the resistance to compression instigated by the wetting process. For the denser one-dimensionally prepared specimen 1D(B) of Figure 1.23(b), no collapse is evident as the resistance to collapse was greater. Both IS(B) and 1D(B) experienced swelling throughout the equalisation process though the plots differ significantly. The magnitude of swelling from the initial conditions is shown as 75% greater for the isotropically compressed specimen than the one-dimensionally compressed specimen.

The MIP results of Figure 1.24(a) and (b) indicate the general trend of reduction in the larger inter-aggregate voids and increase in the smaller intra-aggregate voids following saturation from the as-prepared conditions. Despite the identical initial specific volumes and specific water volumes, the specimens with isotropic and one-dimensional previous stress history exhibit slightly different initial bi-modal pore size distributions, represented in the figure as 'after sampling'. The specimens prepared using one-dimensional (1D) compression are shown to exhibit a slightly more 'open' macro-structure at both levels of initial compression effort (i.e. light 1D(A) and heavy 1D(B) compression). The open structure exhibited by the statically compressed specimen of 1D(A) may have contributed to unstable inter-aggregate contacts susceptible to collapse during wetting. However, the observed small differences in the pore size distributions do not explain satisfactorily the disparity in the volume change behaviour reported in Figure 1.23(a) and (b), other than to suggest that small differences in structure may result in significantly different behaviour characteristics consistent with unstable or meta-stable conditions and a chaotic response to wetting.

1.8 Conclusions

The following outlines the main conclusions:

- Soils encompass a wide spectrum of particulate materials, which in the saturated state have all the voids between the particles filled with water and in the unsaturated state have a percentage of the void spaces filled with air. The wide range of particle sizes and the inherent variability of soils give rise to behavioural characteristics not readily

amenable to analysis. Stress history and time also influence the soil's multi-faceted behaviour requiring simplifications and generalisations in formulating solutions to problems.

- For a soil system in equilibrium, an applied external pressure (or stress regime) is balanced by the components of pressure and stress associated with the phases and their interactions.
- The fabric or structure of soil is derived from the arrangement of the particles and can significantly affect the overall mechanical and hydromechanical properties.
- Unsaturated fine-grained soils can be viewed as having a bi-modal pore size distribution due to aggregation of particles, with a clear division between those smaller voids comprising the intra-aggregate pore spaces and the larger inter-aggregate voids.
- The bi-modal structure does not appear to be readily broken down by compaction even at water contents close to the optimum.
- The wetting of unsaturated specimens results in individual aggregates expanding into the inter-aggregate void spaces, reducing the overall potential expansion of the specimen. The aggregates act as deformable structures in which particle rearrangement takes place under changing conditions.
- Over a wide range of suctions, there is experimental evidence to suggest that the aggregates remain saturated with water, with air restricted to the larger inter-aggregate voids.

Notes

1. In later sections we will more frequently refer to matric suction of which negative pore water pressure may be considered a special case. Suction is the water pressure relative to an elevated datum air pressure and is frequently used in overcoming cavitation in laboratory testing.
2. B.S. 1377 (1990) 2.5 kg rammer compaction is frequently referred to as Standard Proctor compaction and 4.5 kg rammer compaction as the heavy or Modified Proctor compaction. The equivalent ASTM compaction tests are the Standard AASHTO and Modified AASHTO.
3. Throughout the book, the term *specimen* is used to describe soils carefully prepared for testing, while the term *sample* is used to describe volumes of material not yet prepared as specimens.
4. In some disciplines a bubble is defined as having two liquid surfaces, as for a soap bubble, one on the inside and one on the outside of the bubble. Where there is one surface, such as a gas bubble in a liquid, this is referred to as a cavity.
5. This is an example of Ostwald's ripening. Bubbles or cavities with relatively large curvature are less stable and larger bubbles will grow at the expense of smaller ones.
6. Henry's law is a physical principle which states that at a constant temperature, the solubility of a gas in a liquid varies directly with the partial pressure of the gas. Accordingly, as the partial pressure increases, the solubility also increases. The constant of proportionality is called the Henry's law constant and provides a measure of the *solubility* of a compound.
7. While the term effective stress is strictly applicable only to saturated soils or perfectly dry soils with no suction, we will from time to time refer to 'effective' stress as a generic term for the net inter-particle and interactive stresses internal to both saturated and unsaturated soils.