

Chapter 1

Basic characteristics of soils

Learning outcomes

After working through the material in this chapter, you should be able to:

- 1 Understand how soil deposits are formed and the basic composition and structure of soils at the level of the micro-fabric (Sections 1.1 and 1.2);
- 2 Describe (Sections 1.3 and 1.4) and classify (Section 1.5) soils based on their basic physical characteristics;
- 3 Determine the basic physical characteristics of a soil continuum (i.e. at the level of the macro-fabric, Section 1.6);
- 4 Specify compaction required to produce engineered fill materials with desired continuum properties for use in geotechnical constructions (Section 1.7).

1.1 The origin of soils

To the civil engineer, soil is any uncemented or weakly cemented accumulation of mineral particles formed by the weathering of rocks as part of the rock cycle (Figure 1.1), the void space between the particles containing water and/or air. Weak cementation can be due to carbonates or oxides precipitated between the particles, or due to organic matter. Subsequent deposition and compression of soils, combined with cementation between particles, transforms soils into sedimentary rocks (a process known as **lithification**). If the products of weathering remain at their original location they constitute a **residual soil**. If the products are transported and deposited in a different location they constitute a **transported soil**, the agents of transportation being gravity, wind, water and glaciers. During transportation, the size and shape of particles can undergo change and the particles can be sorted into specific size ranges. Particle sizes in soils can vary from over 100 mm to less than 0.001 mm. In the UK, the size ranges are described as shown in Figure 1.2. In Figure 1.2, the terms ‘clay’, ‘silt’ etc. are used to describe only the sizes of particles between specified limits. However, the same terms are also used to describe particular types of soil, classified according to their mechanical behaviour (see Section 1.5).

The type of transportation and subsequent deposition of soil particles has a strong influence on the distribution of particle sizes at a particular location. Some common depositional regimes are shown in Figure 1.3. In glacial regimes, soil material is eroded from underlying rock by the frictional and freeze-thaw action of glaciers. The material, which is typically very varied in particle size from clay to

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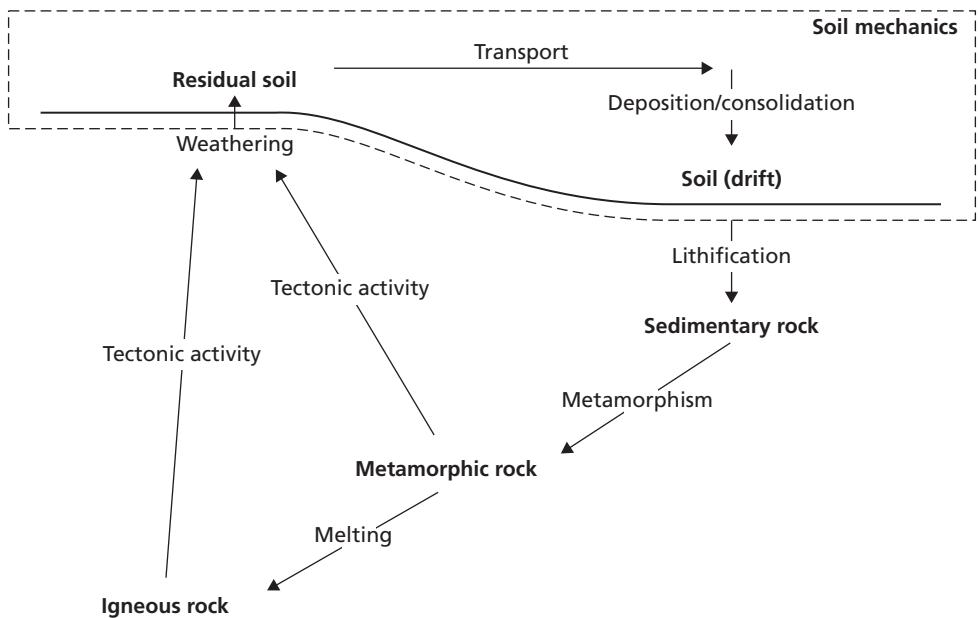


Figure 1.1 The rock cycle.

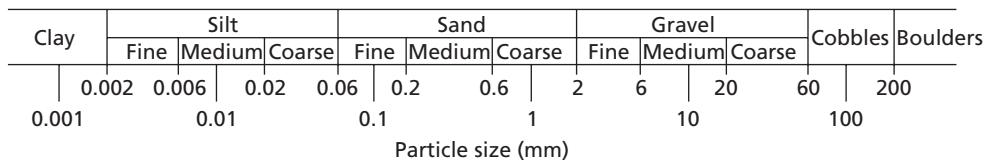


Figure 1.2 Particle size ranges.

boulder-sized particles, is carried along at the base of the glacier and deposited as the ice melts; the resulting material is known as (**glacial**) **till**. Similar material is also deposited as a terminal moraine at the edge of the glacier. As the glacier melts, moraine is transported in the outwash; it is easier for smaller, lighter particles to be carried in suspension, leading to a gradation in particle size with distance from the glacier as shown in Figure 1.3(a). In warmer temperate climates the chief transporting action is water (i.e. rivers and seas), as shown in Figure 1.3(b). The deposited material is known as **alluvium**, the composition of which depends on the speed of water flow. Faster-flowing rivers can carry larger particles in suspension, resulting in alluvium, which is a mixture of sand and gravel-sized particles, while slower-flowing water will tend to carry only smaller particles. At estuarine locations where rivers meet the sea, material may be deposited as a shelf or **delta**. In arid (desert) environments (Figure 1.3(c)) wind is the key agent of transportation, eroding rock outcrops and forming a **pediment** (the desert floor) of fine wind-blown sediment (**loess**). Towards the coast, a **playa** of temporary evaporating lakes, leaving salt deposits, may also be formed. The large temperature differences between night and day additionally cause thermal weathering of rock outcrops, producing **scree**. These surface processes are geologically very recent, and are referred to as **drift deposits** on geological maps. Soil which has undergone significant compression/consolidation following deposition is typically much older and is referred to as **solid**, alongside rocks, on geological maps.

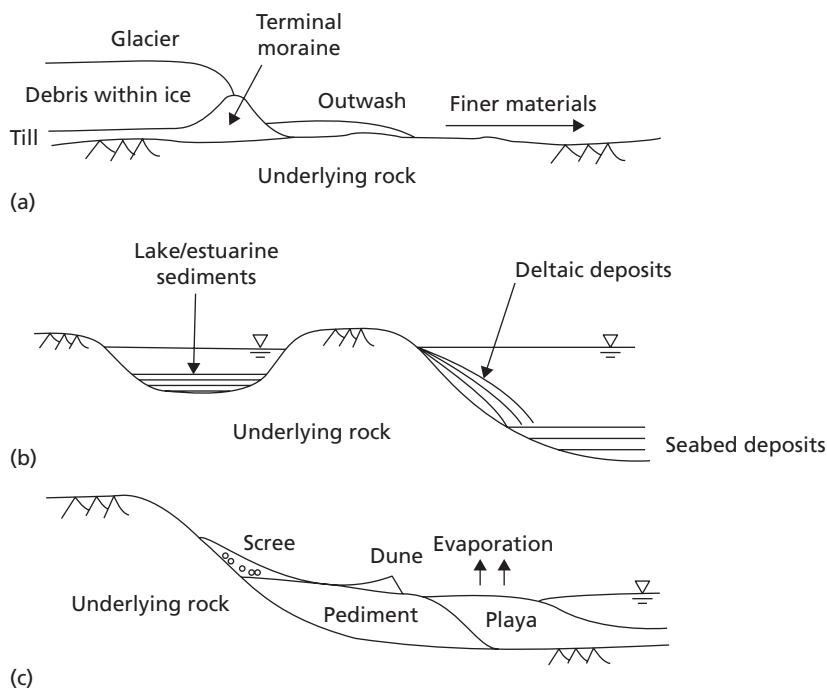


Figure 1.3 Common depositional environments: (a) glacial, (b) fluvial, (c) desert.

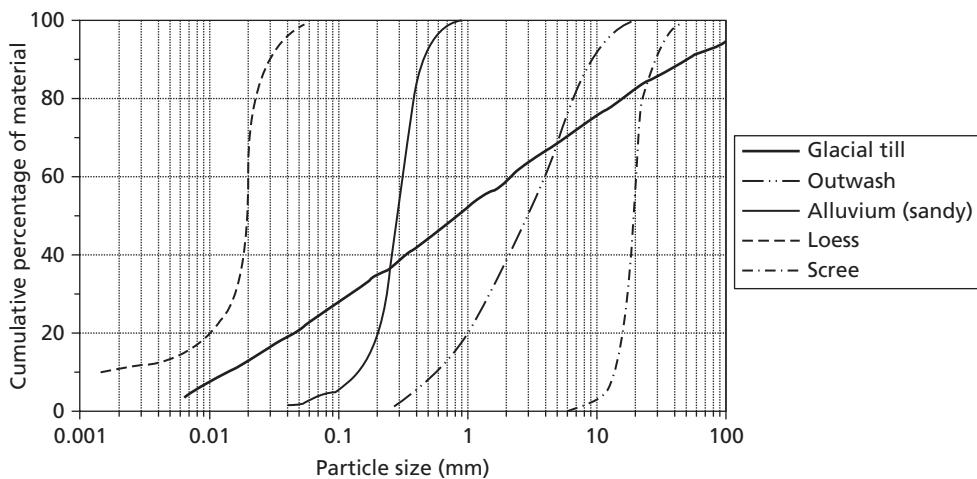


Figure 1.4 Particle size distributions of sediments from different depositional environments.

The relative proportions of different-sized particles within a soil are described as its **particle size distribution** (PSD), and typical curves for materials in different depositional environments are shown in Figure 1.4. The method of determining the PSD of a deposit and its subsequent use in soil classification is described in Sections 1.4 and 1.5.

At a given location, the subsurface materials will be a mixture of rocks and soils, stretching back many hundreds of millions of years in geological time. As a result, it is important to understand the full

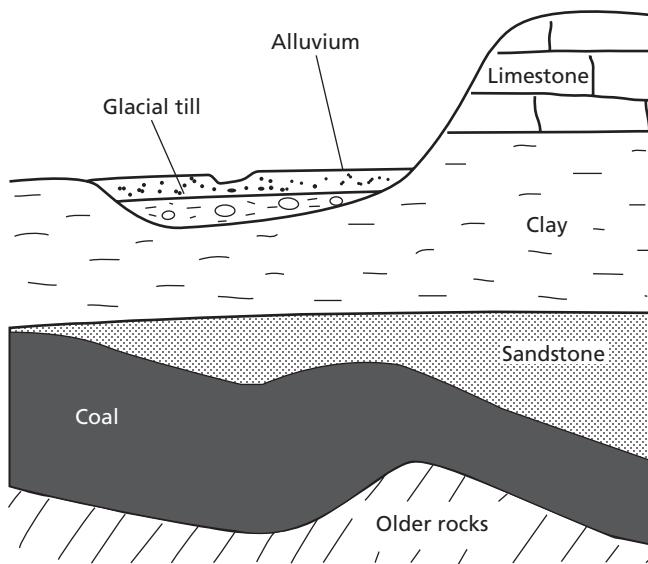


Figure 1.5 Typical ground profile in the West Midlands, UK.

geological history of an area to understand the likely characteristics of the deposits that will be present at the surface, as the depositional regime may have changed significantly over geological time. As an example, the West Midlands in the UK was deltaic in the Carboniferous period (~395–345 million years ago), depositing organic material which subsequently became coal measures. In the subsequent Triassic period (280–225 million years ago), due to a change in sea level sandy materials were deposited which were subsequently lithified to become Bunter sandstone. Mountain building during this period on what is now the European continent caused the existing rock layers to become folded. It was subsequently flooded by the North Sea during the Cretaceous/Jurassic periods (225–136 million years ago), depositing fine particles and carbonate material (Lias clay and Oolitic limestone). The Ice Ages in the Pleistocene period (1.5–2 million years ago) subsequently led to glaciation over all but the southernmost part of the UK, eroding some of the recently deposited softer rocks and depositing glacial till. The subsequent melting of the glaciers created river valleys, which deposited alluvium above the till. The geological history would therefore suggest that the surficial soil conditions are likely to consist of alluvium overlying till/clay overlying stronger rocks, as shown schematically in Figure 1.5. This example demonstrates the importance of engineering geology in understanding ground conditions. A thorough introduction to this topic can be found in Waltham (2002).

1.2 The nature of soils

The destructive process in the formation of soil from rock may be either physical or chemical. The physical process may be erosion by the action of wind, water or glaciers, or disintegration caused by cycles of freezing and thawing in cracks in the rock. The resultant soil particles retain the same mineralogical composition as that of the parent rock (a full description of this is beyond the scope of this text). Particles of this type are described as being of ‘bulky’ form, and their shape can be indicated by terms such as angular, rounded, flat and elongated. The particles occur in a wide range of sizes, from boulders, through gravels and sands, to the fine rock flour formed by the grinding action of glaciers. The structural arrangement of bulky particles (Figure 1.6) is described as **single grain**, each particle being in direct

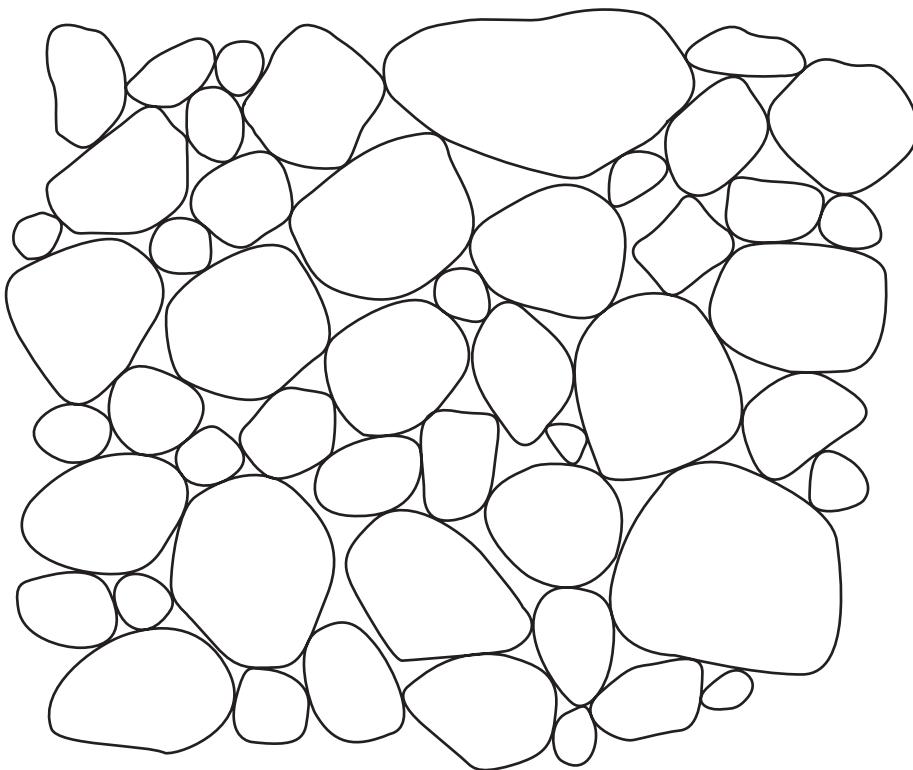


Figure 1.6 Single grain structure.

contact with adjoining particles without there being any bond between them. The state of the particles can be described as dense, medium dense or loose, depending on how they are packed together (see Section 1.5).

Chemical processes result in changes in the mineral form of the parent rock due to the action of water (especially if it contains traces of acid or alkali), oxygen and carbon dioxide. Chemical weathering results in the formation of groups of crystalline particles of **colloidal** size ($<0.002\text{ mm}$) known as clay minerals. The clay mineral kaolinite, for example, is formed by the breakdown of feldspar by the action of water and carbon dioxide. Most clay mineral particles are of ‘plate-like’ form, having a high specific surface (i.e. a high surface area to mass ratio), with the result that their structure is influenced significantly by surface forces. Long ‘needle-shaped’ particles can also occur, but are comparatively rare.

The basic structural units of most clay minerals are a silicon–oxygen tetrahedron and an aluminium–hydroxyl octahedron, as illustrated in Figure 1.7(a). There are valency imbalances in both units, resulting in net negative charges. The basic units therefore do not exist in isolation, but combine to form sheet structures. The tetrahedral units combine by the sharing of oxygen ions to form a silica sheet. The octahedral units combine through shared hydroxyl ions to form a gibbsite sheet. The silica sheet retains a net negative charge, but the gibbsite sheet is electrically neutral. Silicon and aluminium may be partially replaced by other elements, this being known as **isomorphous substitution**, resulting in further charge imbalance. The sheet structures are represented symbolically in Figure 1.7(b). Layer structures then form by the bonding of a silica sheet with either one or two gibbsite sheets. Clay mineral particles consist of stacks of these layers, with different forms of bonding between the layers.

The surfaces of clay mineral particles carry residual negative charges, mainly as a result of the isomorphous substitution of silicon or aluminium by ions of lower valency but also due to disassociation of

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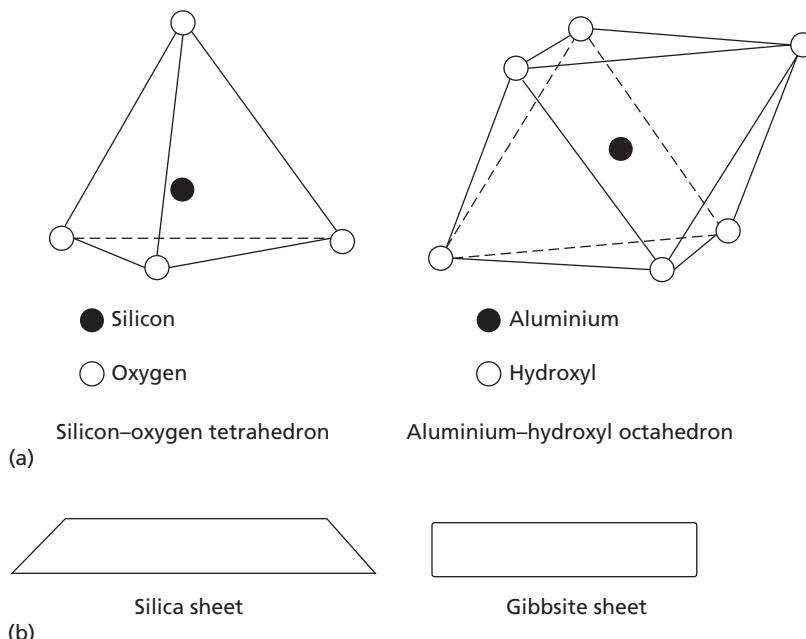


Figure 1.7 Clay minerals: basic units.

hydroxyl ions. Unsatisfied charges due to ‘broken bonds’ at the edges of particles also occur. The negative charges result in cations present in the water in the void space being attracted to the particles. The cations are not held strongly and, if the nature of the water changes, can be replaced by other cations, a phenomenon referred to as **base exchange**.

Cations are attracted to a clay mineral particle because of the negatively charged surface, but at the same time they tend to move away from each other because of their thermal energy. The net effect is that the cations form a dispersed layer adjacent to the particle, the cation concentration decreasing with increasing distance from the surface until the concentration becomes equal to that in the general mass of water in the void space of the soil as a whole. The term ‘double layer’ describes the negatively charged particle surface and the dispersed layer of cations. For a given particle, the thickness of the cation layer depends mainly on the valency and concentration of the cations: an increase in valency (due to cation exchange) or an increase in concentration will result in a decrease in layer thickness.

Layers of water molecules are held around a clay mineral particle by hydrogen bonding and (because water molecules are dipolar) by attraction to the negatively charged surfaces. In addition, the exchangeable cations attract water (i.e. they become hydrated). The particle is thus surrounded by a layer of adsorbed water. The water nearest to the particle is strongly held and appears to have a high viscosity, but the viscosity decreases with increasing distance from the particle surface to that of ‘free’ water at the boundary of the adsorbed layer. Adsorbed water molecules can move relatively freely parallel to the particle surface, but movement perpendicular to the surface is restricted.

The structures of the principal clay minerals are represented in Figure 1.8. Kaolinite consists of a structure based on a single sheet of silica combined with a single sheet of gibbsite. There is very limited isomorphous substitution. The combined silica–gibbsite sheets are held together relatively strongly by hydrogen bonding. A kaolinite particle may consist of over 100 stacks. Illite has a basic structure consisting of a sheet of gibbsite between and combined with two sheets of silica. In the silica sheet, there is partial substitution of silicon by aluminium. The combined sheets are linked together by relatively weak bonding due to non-exchangeable potassium ions held between them. Montmorillonite has the same

basic structure as illite. In the gibbsite sheet there is partial substitution of aluminium by magnesium and iron, and in the silica sheet there is again partial substitution of silicon by aluminium. The space between the combined sheets is occupied by water molecules and exchangeable cations other than potassium, resulting in a very weak bond. Considerable swelling of montmorillonite (and therefore of any soil of which it is a part) can occur due to additional water being adsorbed between the combined sheets. This demonstrates that understanding the basic composition of a soil in terms of its mineralogy can provide clues as to the geotechnical problems which may subsequently be encountered.

Forces of repulsion and attraction act between adjacent clay mineral particles. Repulsion occurs between the like charges of the double layers, the force of repulsion depending on the characteristics of the layers. An increase in cation valency or concentration will result in a decrease in repulsive force and vice versa. Attraction between particles is due to short-range van der Waals forces (electrical forces of attraction between neutral molecules), which are independent of the double-layer characteristics, that decrease rapidly with increasing distance between particles. The net inter-particle forces influence the structural form of clay mineral particles on deposition. If there is net repulsion the particles tend to assume a face-to-face orientation, this being referred to as a **dispersed** structure. If, on the other hand, there is net attraction the orientation of the particles tends to be edge-to-face or edge-to-edge, this being referred to as a **flocculated** structure. These structures, involving interaction between single clay mineral particles, are illustrated in Figures 1.9(a) and (b).

In natural clays, which normally contain a significant proportion of larger, bulky particles, the structural arrangement can be extremely complex. Interaction between single clay mineral particles is rare, the tendency being for the formation of elementary **aggregations** of particles with a face-to-face orientation. In turn, these elementary aggregations combine to form larger assemblages, the structure of which

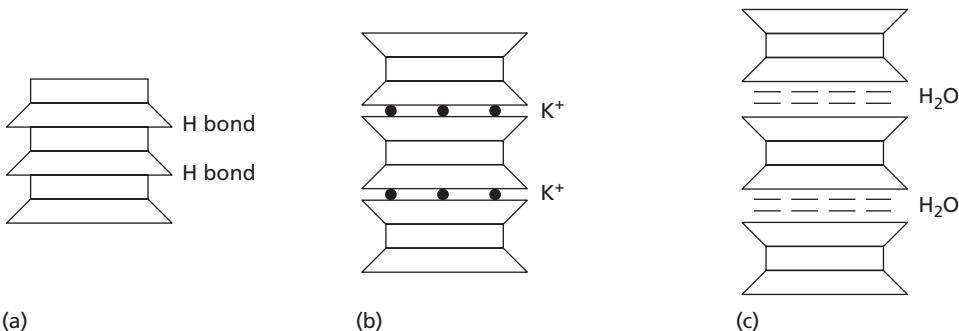


Figure 1.8 Clay minerals: (a) kaolinite, (b) illite, and (c) montmorillonite.

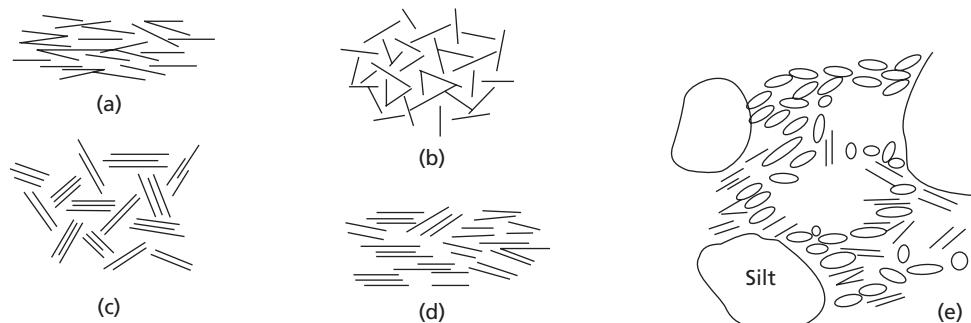


Figure 1.9 Clay structures: (a) dispersed, (b) flocculated, (c) bookhouse, (d) turbostratic, (e) example of a natural clay.

is influenced by the depositional environment. Two possible forms of particle assemblage, known as the bookhouse and turbostratic structures, are illustrated in Figures 1.9(c) and (d). Assemblages can also occur in the form of connectors or a matrix between larger particles. An example of the structure of a natural clay, in diagrammatical form, is shown in Figure 1.9(e).

If clay mineral particles are present they usually exert a considerable influence on the properties of a soil, an influence out of all proportion to their percentage by weight in the soil. Soils whose properties are influenced mainly by clay and silt size particles are commonly referred to as **fine-grained** (or fine) soils. Those whose properties are influenced mainly by sand and gravel size particles are referred to as **coarse-grained** (or coarse) soils.

1.3 Plasticity of fine-grained soils

Plasticity is an important characteristic in the case of fine-grained soils, the term ‘plasticity’ describing the ability of a soil to undergo irrecoverable deformation without cracking or crumbling. In general, depending on its **water content** (defined as the ratio of the mass of water in the soil to the mass of solid particles), a soil may exist in one of the liquid, plastic, semi-solid and solid states. If the water content of a soil initially in the liquid state is gradually reduced, the state will change from liquid through plastic and semi-solid, accompanied by gradually reducing volume, until the solid state is reached. The water contents at which the transitions between states occur differ from soil to soil. In the ground, most fine-grained soils exist in the plastic state. Plasticity is due to the presence of a significant content of clay mineral particles (or organic material) in the soil. The void space between such particles is generally very small in size with the result that water is held at negative pressure by capillary tension, allowing the soil to be deformed or moulded. Adsorption of water due to the surface forces on clay mineral particles may contribute to plastic behaviour. Any decrease in water content results in a decrease in cation layer thickness and an increase in the net attractive forces between particles.

The upper and lower limits of the range of water content over which the soil exhibits plastic behaviour are defined as the **liquid limit** (w_L) and the **plastic limit** (w_p), respectively. Above the liquid limit, the soil flows like a liquid (slurry); below the plastic limit, the soil is brittle and crumbly. The water content range itself is defined as the **plasticity index** (I_p), i.e.:

$$I_p = w_L - w_p \quad (1.1)$$

However, the transitions between the different states are gradual, and the liquid and plastic limits must be defined arbitrarily. The natural water content (w) of a soil (adjusted to an equivalent water content of the fraction passing the 425- μm sieve) relative to the liquid and plastic limits can be represented by means of the **liquidity index** (I_L), where

$$I_L = \frac{w - w_p}{I_p} \quad (1.2)$$

The relationship between the different consistency limits is shown in Figure 1.10.

The degree of plasticity of the clay-size fraction of a soil is expressed by the ratio of the plasticity index to the percentage of clay-size particles in the soil (the **clay fraction**): this ratio is called the **activity**. ‘Normal’ soils have an activity between 0.75 and 1.25, i.e. I_p is approximately equal to the clay fraction. Activity below 0.75 is considered inactive, while soils with activity above 1.25 are considered active. Soils of high activity have a greater change in volume when the water content is changed (i.e. greater swelling when wetted and greater shrinkage when drying). Soils of high activity (e.g. containing a significant amount of montmorillonite) can therefore be particularly damaging to geotechnical works.

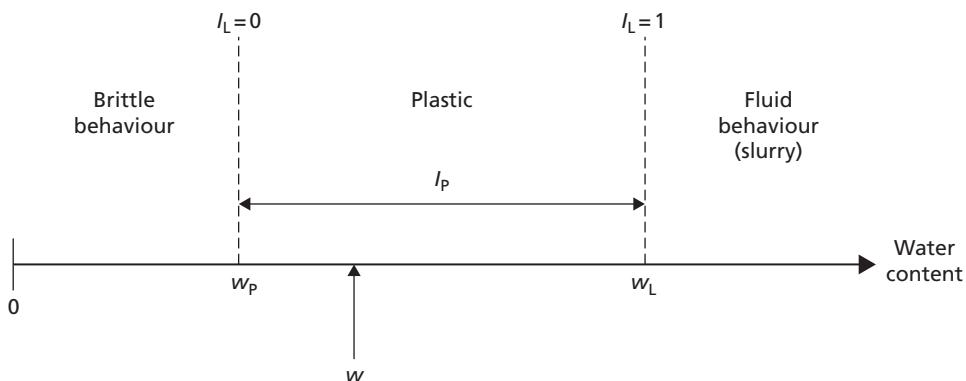


Figure 1.10 Consistency limits for fine soils.

Table 1.1 gives the activity of some common clay minerals, from which it can be seen that activity broadly correlates with the specific surface of the particles (i.e. surface area per unit mass), as this governs the amount of adsorbed water.

The transition between the semi-solid and solid states occurs at the **shrinkage limit**, defined as the water content at which the volume of the soil reaches its lowest value as it dries out.

The liquid and plastic limits are determined by means of arbitrary test procedures. In the UK, these are fully detailed in BS 1377, Part 2 (1990). In Europe CEN ISO/TS 17892–12 (2004) is the current standard, while in the United States ASTM D4318 (2010) is used. These standards all relate to the same basic tests which are described below.

The soil sample is dried sufficiently to enable it to be crumbled and broken up, using a mortar and a rubber pestle, without crushing individual particles; only material passing a 425- μm sieve is typically used in the tests. The apparatus for the liquid limit test consists of a penetrometer (or ‘fall-cone’) fitted with a 30° cone of stainless steel, 35 mm long: the cone and the sliding shaft to which it is attached have a mass of 80 g. This is shown in Figure 1.11(a). The test soil is mixed with distilled water to form a thick homogeneous paste, and stored for 24 h. Some of the paste is then placed in a cylindrical metal cup, 55 mm internal diameter by 40 mm deep, and levelled off at the rim of the cup to give a smooth surface. The cone is lowered so that it just touches the surface of the soil in the cup, the cone being locked in its support at this stage. The cone is then released for a period of 5 s, and its depth of penetration into the soil is measured. A little more of the soil paste is added to the cup and the test is repeated until a consistent value of penetration has been obtained. (The average of two values within 0.5 mm or of three values within 1.0 mm is taken.) The entire test procedure is repeated at least four times, using the same soil sample but increasing the water content each time by adding distilled water. The penetration values should cover the range of approximately 15–25 mm, the tests proceeding from the drier to the wetter state of the soil. Cone penetration is plotted against water content, and the best straight line fitting the

Table 1.1 Activity of some common clay minerals

Mineral group	Specific surface (m^2/g) ¹	Activity ²
Kaolinite	10–20	0.3–0.5
Illite	65–100	0.5–1.3
Montmorillonite	Up to 840	4–7

Notes: 1 After Mitchell and Soga (2005). 2 After Day (2001).

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plotted points is drawn. This is demonstrated in Example 1.1. The liquid limit is defined as the percentage water content (to the nearest integer) corresponding to a cone penetration of 20 mm. Determination of liquid limit may also be based on a single test (the one-point method), provided the cone penetration is between 15 and 25 mm.

An alternative method for determining the liquid limit uses the Casagrande apparatus (Figure 1.11(b)), which is popular in the United States and other parts of the world (ASTM D4318). A soil paste is placed in a pivoting flat metal cup and divided by cutting a groove. A mechanism enables the cup to be lifted to a height of 10 mm and dropped onto a hard rubber base. The two halves of the soil gradually flow together as the cup is repeatedly dropped. The water content of the soil in the cup is then determined; this is plotted against the logarithm of the number of blows, and the best straight line fitting the plotted points is drawn. For this test, the liquid limit is defined as the water content at which 25 blows are required to close the bottom of the groove. It should be noted that the Casagrande method is generally less reliable than the preferred penetrometer method, being more operator dependent and subjective.

For the determination of the plastic limit, the test soil is mixed with distilled water until it becomes sufficiently plastic to be moulded into a ball. Part of the soil sample (approximately 2.5 g) is formed into a thread, approximately 6 mm in diameter, between the first finger and thumb of each hand. The thread is then placed on a glass plate and rolled with the tips of the fingers of one hand until its diameter is reduced to approximately 3 mm: the rolling pressure must be uniform throughout the test. The thread is then remoulded between the fingers (the water content being reduced by the heat of the fingers) and the

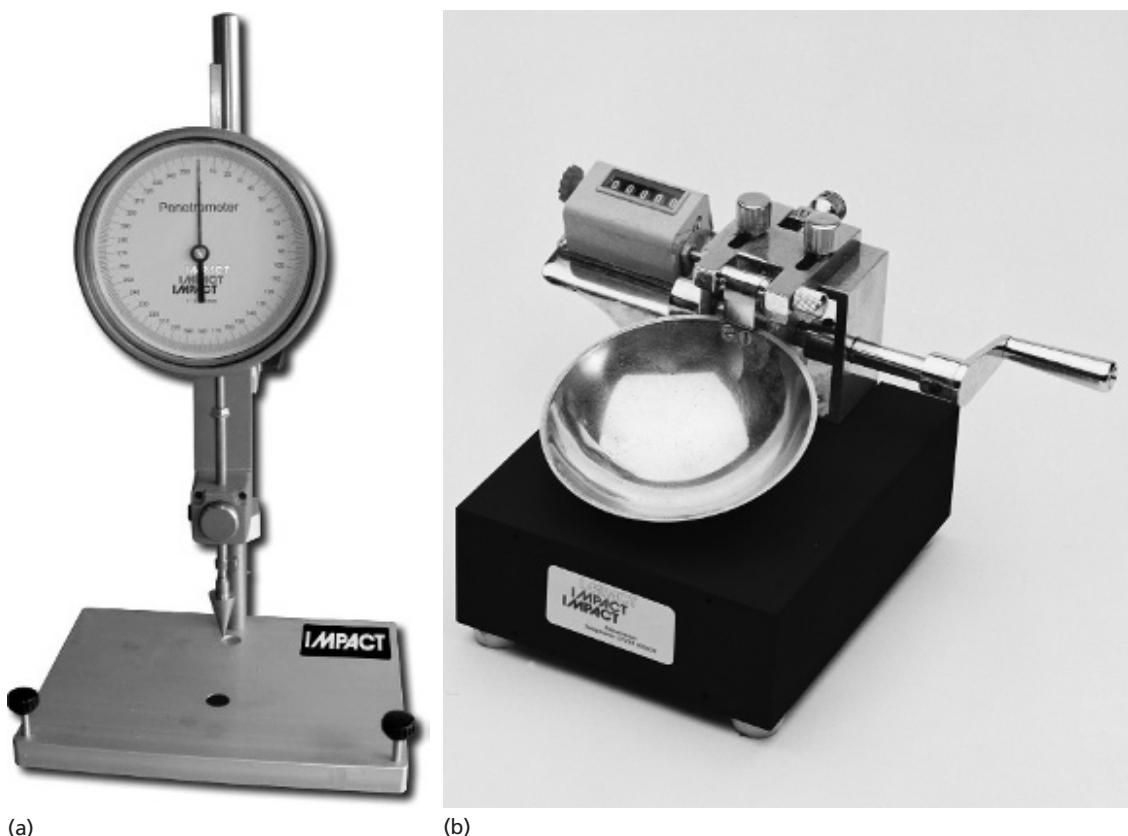


Figure 1.11 Laboratory apparatus for determining liquid limit: (a) fall-cone, (b) Casagrande apparatus (images courtesy of Impact Test Equipment Ltd).

procedure is repeated until the thread of soil shears both longitudinally and transversely when it has been rolled to a diameter of 3 mm. The procedure is repeated using three more parts of the sample, and the percentage water content of all the crumbled soil is determined as a whole. This water content (to the nearest integer) is defined as the plastic limit of the soil. The entire test is repeated using four other subsamples, and the average taken of the two values of plastic limit: the tests must be repeated if the two values differ by more than 0.5%. Due to the strongly subjective nature of this test, alternative methodologies have recently been proposed for determining w_p , though these are not incorporated within current standards. Further information can be found in Barnes (2009) and Sivakumar *et al.* (2009).

1.4 Particle size analysis

Most soils consist of a graded mixture of particles from two or more size ranges. For example, clay is a type of soil possessing cohesion and plasticity which normally consists of particles in both the clay size and silt size ranges. **Cohesion** is the term used to describe the strength of a clay sample when it is unconfined, being due to negative pressure in the water filling the void space, of very small size, between particles. This strength would be lost if the clay were immersed in a body of water. Cohesion may also be derived from cementation between soil particles. It should be appreciated that all clay-size particles are not necessarily clay mineral particles: the finest rock flour particles may be of clay size.

The particle size analysis of a soil sample involves determining the percentage by mass of particles within the different size ranges. The particle size distribution of a coarse soil can be determined by the method of **sieving**. The soil sample is passed through a series of standard test sieves having successively smaller mesh sizes. The mass of soil retained in each sieve is determined, and the cumulative percentage by mass passing each sieve is calculated. If fine particles are present in the soil, the sample should be treated with a deflocculating agent (e.g. a 4% solution of sodium hexametaphosphate) and washed through the sieves.

The particle size distribution (PSD) of a fine soil or the fine fraction of a coarse soil can be determined by the method of **sedimentation**. This method is based on Stokes' law, which governs the velocity at which spherical particles settle in a suspension: the larger the particles are the greater is the settling velocity, and vice versa. The law does not apply to particles smaller than 0.0002 mm, the settlement of which is influenced by Brownian motion. The size of a particle is given as the diameter of a sphere which would settle at the same velocity as the particle. Initially, the soil sample is pretreated with hydrogen peroxide to remove any organic material. The sample is then made up as a suspension in distilled water to which a deflocculating agent has been added to ensure that all particles settle individually, and placed in a sedimentation tube. From Stokes' law it is possible to calculate the time, t , for particles of a certain 'size', D (the equivalent settling diameter), to settle to a specified depth in the suspension. If, after the calculated time t , a sample of the suspension is drawn off with a pipette at the specified depth below the surface, the sample will contain only particles smaller than the size D at a concentration unchanged from that at the start of sedimentation. If pipette samples are taken at the specified depth at times corresponding to other chosen particle sizes, the particle size distribution can be determined from the masses of the residues. An alternative procedure to pipette sampling is the measurement of the specific gravity of the suspension by means of a special hydrometer, the specific gravity depending on the mass of soil particles in the suspension at the time of measurement. Full details of the determination of particle size distribution by these methods are given in BS 1377-2 (UK), CEN ISO/TS 17892-4 (Europe) and ASTM D6913 (US). Modern optical techniques can also be used to determine the PSD of a coarse soil. Single Particle Optical Sizing (SPOS) works by drawing a stream of dry particles through the beam of a laser diode. As each individual particle passes through the beam it casts a shadow on a light sensor which is proportional to its size (and therefore volume). The optical sizer automatically analyses the sensor output to determine the PSD by volume. Optical methods have been found to

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overestimate particle sizes compared to sieving (White, 2003), though advantages are that the results are repeatable and less operator dependent compared to sieving, and testing requires a much smaller volume of soil.

The particle size distribution of a soil is presented as a curve on a semilogarithmic plot, the ordinates being the percentage by mass of particles smaller than the size given by the abscissa. The flatter the distribution curve, the larger the range of particle sizes in the soil; the steeper the curve, the smaller the size range. A coarse soil is described as **well graded** if there is no excess of particles in any size range and if no intermediate sizes are lacking. In general, a well-graded soil is represented by a smooth, concave distribution curve. A coarse soil is described as **poorly graded** (a) if a high proportion of the particles have sizes within narrow limits (a uniform soil), or (b) if particles of both large and small sizes are present but with a relatively low proportion of particles of intermediate size (a **gap-graded** or step-graded soil). Particle size is represented on a logarithmic scale so that two soils having the same degree of uniformity are represented by curves of the same shape regardless of their positions on the particle size distribution plot. Examples of particle size distribution curves appear in Figure 1.4. The particle size corresponding to any specified percentage value can be read from the particle size distribution curve. The size such that 10% of the particles are smaller than that size is denoted by D_{10} . Other sizes, such as D_{30} and D_{60} , can be defined in a similar way. The size D_{10} is defined as the **effective size**, and can be used to estimate the permeability of the soil (see Chapter 2). The general slope and shape of the distribution curve can be described by means of the **coefficient of uniformity** (C_u) and the **coefficient of curvature** (C_z), defined as follows:

$$C_u = \frac{D_{60}}{D_{10}} \quad (1.3)$$

$$C_z = \frac{D_{30}^2}{D_{60} D_{10}} \quad (1.4)$$

The higher the value of the coefficient of uniformity, the larger the range of particle sizes in the soil. A well-graded soil has a coefficient of curvature between 1 and 3. The sizes D_{15} and D_{85} are commonly used to select appropriate material for granular drains used to drain geotechnical works (see Chapter 2).

1.5 Soil description and classification

It is essential that a standard language should exist for the description of soils. A comprehensive description should include the characteristics of both the soil material and the in-situ soil mass. Material characteristics can be determined from disturbed samples of the soil – i.e. samples having the same particle size distribution as the in-situ soil but in which the in-situ structure has not been preserved. The principal material characteristics are particle size distribution (or grading) and plasticity, from which the soil name can be deduced. Particle size distribution and plasticity properties can be determined either by standard laboratory tests (as described in Sections 1.3 and 1.4) or by simple visual and manual procedures. Secondary material characteristics are the colour of the soil and the shape, texture and composition of the particles. Mass characteristics should ideally be determined in the field, but in many cases they can be detected in undisturbed samples – i.e. samples in which the in-situ soil structure has been essentially preserved. A description of mass characteristics should include an assessment of in-situ compactive state (coarse-grained soils) or stiffness (fine-grained soils), and details of any bedding, discontinuities and weathering. The arrangement of minor geological details, referred to as the soil macro-fabric, should be carefully described, as this can influence the engineering behaviour of the in-situ soil to a considerable extent. Examples of macro-fabric features are thin layers of fine sand and silt

in clay, silt-filled fissures in clay, small lenses of clay in sand, organic inclusions, and root holes. The name of the geological formation, if definitely known, should be included in the description; in addition, the type of deposit may be stated (e.g. till, alluvium), as this can indicate, in a general way, the likely behaviour of the soil.

It is important to distinguish between soil description and soil classification. Soil description includes details of both material and mass characteristics, and therefore it is unlikely that any two soils will have identical descriptions. In soil classification, on the other hand, a soil is allocated to one of a limited number of behavioural groups on the basis of material characteristics only. Soil classification is thus independent of the in-situ condition of the soil mass. If the soil is to be employed in its undisturbed condition, for example to support a foundation, a full soil description will be adequate and the addition of the soil classification is discretionary. However, classification is particularly useful if the soil in question is to be used as a construction material when it will be remoulded – for example in an embankment. Engineers can also draw on past experience of the behaviour of soils of similar classification.

Rapid assessment procedures

Both soil description and classification require knowledge of grading and plasticity. This can be determined by the full laboratory procedure using standard tests, as described in Sections 1.3 and 1.4, in which values defining the particle size distribution and the liquid and plastic limits are obtained for the soil in question. Alternatively, grading and plasticity can be assessed using a rapid procedure which involves personal judgements based on the appearance and feel of the soil. The rapid procedure can be used in the field and in other situations where the use of the laboratory procedure is not possible or not justified. In the rapid procedure, the following indicators should be used.

Particles of 0.06 mm, the lower size limit for coarse soils, are just visible to the naked eye, and feel harsh but not gritty when rubbed between the fingers; finer material feels smooth to the touch. The size boundary between sand and gravel is 2 mm, and this represents the largest size of particles which will hold together by capillary attraction when moist. A purely visual judgement must be made as to whether the sample is well graded or poorly graded, this being more difficult for sands than for gravels.

If a predominantly coarse soil contains a significant proportion of fine material, it is important to know whether the fines are essentially plastic or non-plastic (i.e. whether they are predominantly clay or silt, respectively). This can be judged by the extent to which the soil exhibits cohesion and plasticity. A small quantity of the soil, with the largest particles removed, should be moulded together in the hands, adding water if necessary. Cohesion is indicated if the soil, at an appropriate water content, can be moulded into a relatively firm mass. Plasticity is indicated if the soil can be deformed without cracking or crumbling, i.e. without losing cohesion. If cohesion and plasticity are pronounced, then the fines are plastic. If cohesion and plasticity are absent or only weakly indicated, then the fines are essentially non-plastic.

The plasticity of fine soils can be assessed by means of the toughness and dilatancy tests, described below. An assessment of dry strength may also be useful. Any coarse particles, if present, are first removed, and then a small sample of the soil is moulded in the hand to a consistency judged to be just above the plastic limit (i.e. just enough water to mould); water is added or the soil is allowed to dry as necessary. The procedures are then as follows.

Toughness test

A small piece of soil is rolled out into a thread on a flat surface or on the palm of the hand, moulded together, and rolled out again until it has dried sufficiently to break into lumps at a diameter of around 3 mm. In this condition, inorganic clays of high liquid limit are fairly stiff and tough; those of low liquid limit are softer and crumble more easily. Inorganic silts produce a weak and often soft thread, which may be difficult to form and which readily breaks and crumbles.

Dilatancy test

A pat of soil, with sufficient water added to make it soft but not sticky, is placed in the open (horizontal) palm of the hand. The side of the hand is then struck against the other hand several times. Dilatancy is indicated by the appearance of a shiny film of water on the surface of the pat; if the pat is then squeezed or pressed with the fingers, the surface becomes dull as the pat stiffens and eventually crumbles. These reactions are pronounced only for predominantly silt-size material and for very fine sands. Plastic clays give no reaction.

Dry strength test

A pat of soil about 6 mm thick is allowed to dry completely, either naturally or in an oven. The strength of the dry soil is then assessed by breaking and crumbling between the fingers. Inorganic clays have relatively high dry strength; the greater the strength, the higher the liquid limit. Inorganic silts of low liquid limit have little or no dry strength, crumbling easily between the fingers.

Soil description details

A detailed guide to soil description as used in the UK is given in BS 5930 (1999), and the subsequent discussion is based on this standard. In Europe the standard is EN ISO 14688–1 (2002), while in the United States ASTM D2487 (2011) is used. The basic soil types are boulders, cobbles, gravel, sand, silt and clay, defined in terms of the particle size ranges shown in Figure 1.2; added to these are organic clay, silt or sand, and peat. These names are always written in capital letters in a soil description. Mixtures of the basic soil types are referred to as composite types.

A soil is of basic type sand or gravel (these being termed coarse soils) if, after the removal of any cobbles or boulders, over 65% of the material is of sand and gravel sizes. A soil is of basic type silt or clay (termed fine soils) if, after the removal of any cobbles or boulders, over 35% of the material is of silt and clay sizes. However, these percentages should be considered as approximate guidelines, not forming a rigid boundary. Sand and gravel may each be subdivided into coarse, medium and fine fractions as defined in Figure 1.2. The state of sand and gravel can be described as well graded, poorly graded, uniform or gap graded, as defined in Section 1.4. In the case of gravels, particle shape (angular, sub-angular, sub-rounded, rounded, flat, elongated) and surface texture (rough, smooth, polished) can be described if necessary. Particle composition can also be stated. Gravel particles are usually rock fragments (e.g. sandstone, schist). Sand particles usually consist of individual mineral grains (e.g. quartz, feldspar). Fine soils should be described as either silt or clay: terms such as silty clay should not be used.

Organic soils contain a significant proportion of dispersed vegetable matter, which usually produces a distinctive odour and often a dark brown, dark grey or bluish grey colour. Peats consist predominantly of plant remains, usually dark brown or black in colour and with a distinctive odour. If the plant remains are recognisable and retain some strength, the peat is described as fibrous. If the plant remains are recognisable but their strength has been lost, they are pseudo-fibrous. If recognisable plant remains are absent, the peat is described as amorphous. Organic content is measured by burning a sample of soil at a controlled temperature to determine the reduction in mass which corresponds to the organic content. Alternatively, the soil may be treated with hydrogen peroxide (H_2O_2), which also removes the organic content, resulting in a loss of mass.

Composite types of coarse soil are named in Table 1.2, the predominant component being written in capital letters. Fine soils containing 35–65% coarse material are described as sandy and/or gravelly SILT (or CLAY). Deposits containing over 50% of boulders and cobbles are referred to as very coarse, and normally can be described only in excavations and exposures. Mixes of very coarse material with finer soils can be described by combining the descriptions of the two components – e.g. COBBLES with some FINER MATERIAL (sand); gravelly SAND with occasional BOULDERS.

The state of compaction or stiffness of the in-situ soil can be assessed by means of the tests or indications detailed in Table 1.3.

Table 1.2 Composite types of coarse soil

Slightly sandy GRAVEL	Up to 5% sand
Sandy GRAVEL	5–20% sand
Very sandy GRAVEL	Over 20% sand
SAND and GRAVEL	About equal proportions
Very gravelly SAND	Over 20% gravel
Gravelly SAND	5–20% gravel
Slightly gravelly SAND	Up to 5% gravel
Slightly silty SAND (and/or GRAVEL)	Up to 5% silt
Silty SAND (and/or GRAVEL)	5–20% silt
Very silty SAND (and/or GRAVEL)	Over 20% silt
Slightly clayey SAND (and/or GRAVEL)	Up to 5% clay
Clayey SAND (and/or GRAVEL)	5–20% clay
Very clayey SAND (and/or GRAVEL)	Over 20% clay

Note: Terms such as 'Slightly clayey gravelly SAND' (having less than 5% clay and gravel) and 'Silty sandy GRAVEL' (having 5–20% silt and sand) can be used, based on the above proportions of secondary constituents.

Table 1.3 Compactive state and stiffness of soils

Soil group	Term (relative density – Section 1.6)	Field test or indication
Coarse soils	Very loose (0–20%)	Assessed on basis of N -value determined by means of Standard Penetration Test (SPT) – see Chapter 7
	Loose (20–40%)	For definition of relative density, see Equation (1.23)
	Medium dense (40–60%)	Visual examination: pick removes soil in lumps which can be abraded
	Dense (60–80%)	Easily moulded or crushed by the fingers
	Very dense (80–100%)	Can be moulded or crushed by strong finger pressure
	Slightly cemented	Finger can easily be pushed in up to 25 mm
Fine soils	Uncompact	Finger can be pushed in up to 10 mm
	Compact	Thumb can make impression easily
	Very soft	Thumb can make slight indentation
	Soft	Thumb nail can make indentation
	Firm	Thumb nail can make surface scratch
	Stiff	Fibres already pressed together
	Very stiff	Very compressible and open structure
Organic soils	Hard	Can be moulded in the hand and smears fingers
	Firm	
	Spongy	
	Plastic	

Development of a mechanical model for soil

Discontinuities such as fissures and shear planes, including their spacings, should be indicated. Bedding features, including their thickness, should be detailed. Alternating layers of varying soil types or with bands or lenses of other materials are described as **interstratified**. Layers of different soil types are described as **interbedded** or **inter-laminated**, their thickness being stated. Bedding surfaces that separate easily are referred to as **partings**. If partings incorporate other material, this should be described.

Some examples of soil description are as follows:

- Dense, reddish-brown, sub-angular, well-graded SAND
- Firm, grey, laminated CLAY with occasional silt partings 0.5–2.0 mm (Alluvium)
- Dense, brown, well graded, very silty SAND and GRAVEL with some COBBLES (Till)
- Stiff, brown, closely fissured CLAY (London Clay)
- Spongy, dark brown, fibrous PEAT (Recent Deposits)

Soil classification systems

General classification systems, in which soils are placed into behavioural groups on the basis of grading and plasticity, have been used for many years. The feature of these systems is that each soil group is denoted by a letter symbol representing main and qualifying terms. The terms and letters used in the UK are detailed in Table 1.4. The boundary between coarse and fine soils is generally taken to be 35% fines (i.e. particles smaller than 0.06 mm). The liquid and plastic limits are used to classify fine soils, employing the plasticity chart shown in Figure 1.12. The axes of the plasticity chart are the plasticity index and liquid limit; therefore, the plasticity characteristics of a particular soil can be represented by a point on the chart. Classification letters are allotted to the soil according to the zone within which the point lies. The chart is divided into five ranges of liquid limit. The four ranges I, H, V and E can be combined as an upper range (U) if closer designation is not required, or if the rapid assessment procedure has been used to assess plasticity. The diagonal line on the chart, known as the **A-line**, should not be regarded as a rigid boundary between clay and silt for purposes of soil description, as opposed to classification. The A-line may be mathematically represented by

$$I_p = 0.73(w_L - 20) \quad (1.5)$$

The letter denoting the dominant size fraction is placed first in the group symbol. If a soil has a significant content of organic matter, the suffix O is added as the last letter of the group symbol. A group symbol may consist of two or more letters, for example:

- SW – well-graded SAND
- SCL – very clayey SAND (clay of low plasticity)
- CIS – sandy CLAY of intermediate plasticity
- MHSO – organic sandy SILT of high plasticity.

The name of the soil group should always be given, as above, in addition to the symbol, the extent of subdivision depending on the particular situation. If the rapid procedure has been used to assess grading and plasticity, the group symbol should be enclosed in brackets to indicate the lower degree of accuracy associated with this procedure.

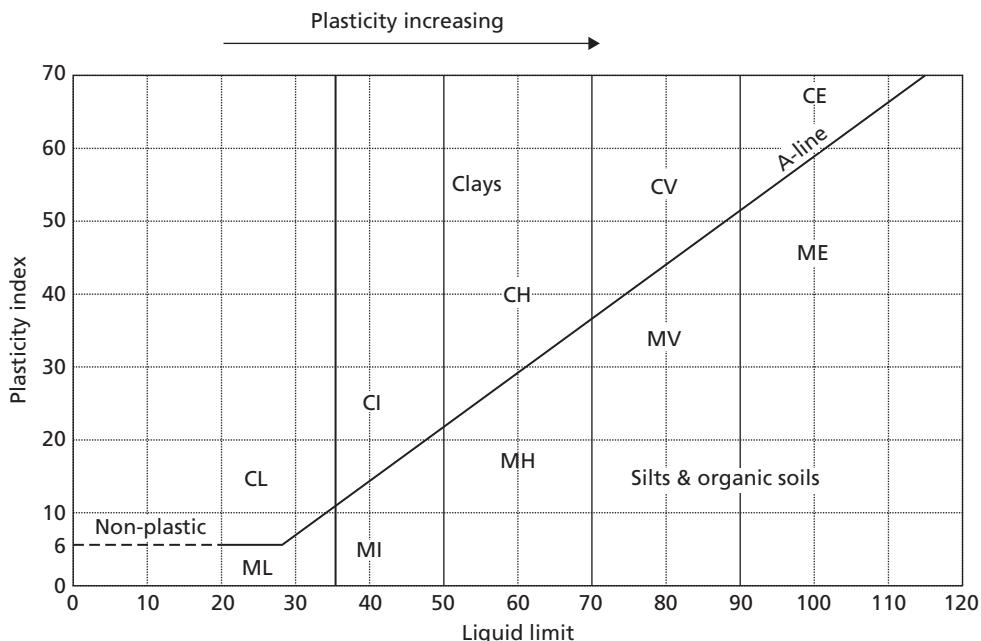


Figure 1.12 Plasticity chart: British system (BS 1377-2: 1990).

The term FINE SOIL or FINES (F) is used when it is not required, or not possible, to differentiate between SILT (M) and CLAY (C). SILT (M) plots below the A-line and CLAY (C) above the A-line on the plasticity chart, i.e. silts exhibit plastic properties over a lower range of water content than clays having the same liquid limit. SILT or CLAY is qualified as gravelly if more than 50% of the coarse fraction is of gravel size, and as sandy if more than 50% of the coarse fraction is of sand size. The alternative term M-SOIL is introduced to describe material which, regardless of its particle size distribution, plots below the A-line on the plasticity chart: the use of this term avoids confusion with soils of predominantly silt size (but with a significant proportion of clay-size particles), which plot above the A-line. Fine soils containing significant amounts of organic matter usually have high to extremely high liquid limits, and plot below the A-line as organic silt. Peats usually have very high or extremely high liquid limits.

Any cobbles or boulders (particles retained on a 63-mm sieve) are removed from the soil before the classification tests are carried out, but their percentages in the total sample should be determined or estimated. Mixtures of soil and cobbles or boulders can be indicated by using the letters Cb (COBBLES) or B (BOULDERS) joined by a + sign to the group symbol for the soil, the dominant component being given first – for example:

GW + Cb – well-graded GRAVEL with COBBLES
 B + CL – BOULDERS with CLAY of low plasticity.

A similar classification system, known as the Unified Soil Classification System (USCS), was developed in the United States (described in ASTM D2487), but with less detailed subdivisions. As the USCS method is popular in other parts of the world, alternative versions of Figure 1.12 and Table 1.4 are provided on the Companion Website.

Table 1.4 Descriptive terms for soil classification (BS 5930)

Main terms		Qualifying terms	
GRAVEL	G	Well graded	W
SAND	S	Poorly graded	P
		Uniform	Pu
		Gap graded	Pg
FINE SOIL, FINES	F	Of low plasticity ($w_L < 35$)	L
SILT (M-SOIL)	M	Of intermediate plasticity ($w_L 35-50$)	I
CLAY	C	Of high plasticity ($w_L 50-70$)	H
		Of very high plasticity ($w_L 70-90$)	V
		Of extremely high plasticity ($w_L > 90$)	E
		Of upper plasticity range ($w_L > 35$)	U
PEAT	Pt	Organic (may be a suffix to any group)	O

Example 1.1

The results of particle size analyses of four soils A, B, C and D are shown in Table 1.5. The results of limit tests on soil D are as follows:

TABLE A*Liquid limit:*

Cone penetration (mm)	15.5	18.0	19.4	22.2	24.9
Water content (%)	39.3	40.8	42.1	44.6	45.6

Plastic limit:

Water content (%)	23.9	24.3
-------------------	------	------

The fine fraction of soil C has a liquid limit of $I_L = 26$ and a plasticity index of $I_p = 9$.

- Determine the coefficients of uniformity and curvature for soils A, B and C.
- Allot group symbols, with main and qualifying terms to each soil.

Table 1.5 Example 1.1

Sieve	Particle size*	Percentage smaller			
		Soil A	Soil B	Soil C	Soil D
63 mm		100		100	
20 mm		64		76	
6.3 mm		39	100	65	
2 mm		24	98	59	
600 mm		12	90	54	
212 mm		5	9	47	100
63 mm		0	3	34	95
	0.020 mm			23	69
	0.006 mm			14	46
	0.002 mm			7	31

Note: * From sedimentation test.

Solution

The particle size distribution curves are plotted in Figure 1.13. For soils A, B and C, the sizes D_{10} , D_{30} and D_{60} are read from the curves and the values of C_u and C_z are calculated:

TABLE B

Soil	D_{10}	D_{30}	D_{60}	C_u	C_z
A	0.47	3.5	16	34	1.6
B	0.23	0.30	0.41	1.8	0.95
C	0.003	0.042	2.4	800	0.25

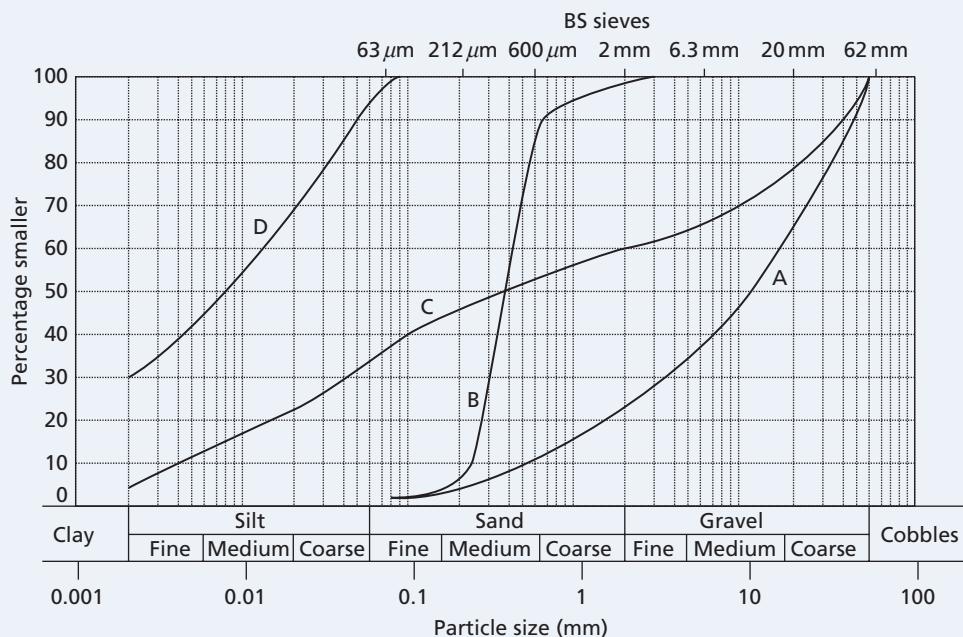


Figure 1.13 Particle size distribution curves (Example 1.1).

For soil D the liquid limit is obtained from Figure 1.14, in which fall-cone penetration is plotted against water content. The percentage water content, to the nearest integer, corresponding to a penetration of 20 mm is the liquid limit, and is 42%. The plastic limit is the average of the two percentage water contents, again to the nearest integer, i.e. 24%. The plasticity index is the difference between the liquid and plastic limits, i.e. 18%.

Soil A consists of 100% coarse material (76% gravel size; 24% sand size) and is classified as GW: well-graded, very sandy GRAVEL.

Soil B consists of 97% coarse material (95% sand size; 2% gravel size) and 3% fines. It is classified as SPu: uniform, slightly silty, medium SAND.

Soil C comprises 66% coarse material (41% gravel size; 25% sand size) and 34% fines ($w_L=26$, $I_p=9$, plotting in the CL zone on the plasticity chart). The classification is GCL: very clayey GRAVEL (clay of low plasticity). This is a till, a glacial deposit having a large range of particle sizes.

Soil D contains 95% fine material: the liquid limit is 42 and the plasticity index is 18, plotting just above the A-line in the CI zone on the plasticity chart. The classification is thus CI: CLAY of intermediate plasticity.

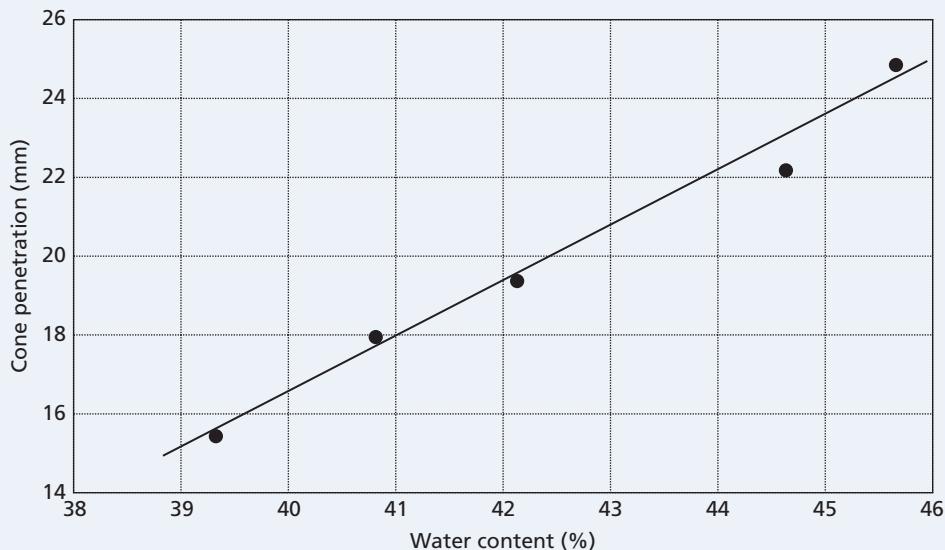


Figure 1.14 Determination of liquid limit (Example 1.1).

1.6 Phase relationships

It has been demonstrated in Sections 1.1–1.5 that the constituent particles of soil, their mineralogy and microstructure determine the classification of a soil into a certain behavioural type. At the scale of most engineering processes and constructions, however, it is necessary to describe the soil as a continuum. Soils can be of either two-phase or three-phase composition. In a completely dry soil there are two phases, namely the solid soil particles and pore air. A fully saturated soil is also two-phase, being composed of solid soil particles and pore water. A partially saturated soil is three-phase, being composed of solid soil particles, pore water and pore air. The components of a soil can be represented by a phase diagram as shown in Figure 1.15(a), from which the following relationships are defined.

The water content (w), or moisture content (m), is the ratio of the mass of water to the mass of solids in the soil, i.e.

$$w = \frac{M_w}{M_s} \quad (1.6)$$

The water content is determined by weighing a sample of the soil and then drying the sample in an oven at a temperature of 105–110°C and re-weighing. Drying should continue until the differences between successive weighings at four-hourly intervals are not greater than 0.1% of the original mass of the sample. A drying period of 24 h is normally adequate for most soils.

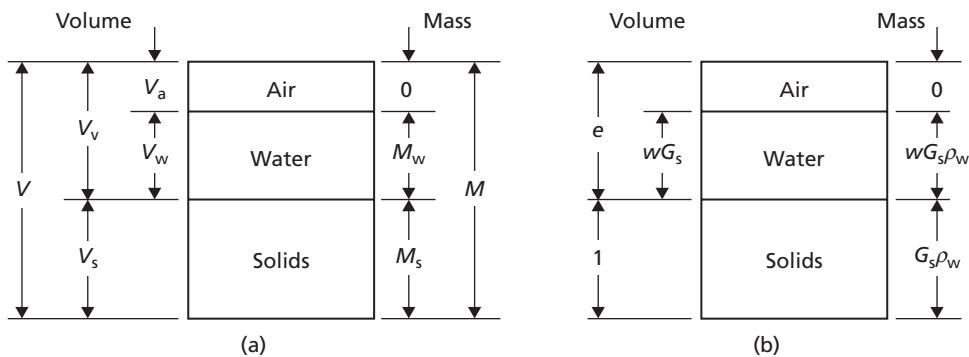


Figure 1.15 Phase diagrams.

The degree of saturation or **saturation ratio** (S_t) is the ratio of the volume of water to the total volume of void space, i.e.

$$S_r = \frac{V_w}{V_y} \quad (1.7)$$

The saturation ratio can range between the limits of zero for a completely dry soil and one (or 100%) for a fully saturated soil.

The **void ratio** (e) is the ratio of the volume of voids to the volume of solids, i.e.

$$e = \frac{V_v}{V_s} \quad (1.8)$$

The **porosity** (n) is the ratio of the volume of voids to the total volume of the soil, i.e.

$$n = \frac{V_v}{V} \quad (1.9)$$

As $V = V_v + V_s$, void ratio and porosity are interrelated as follows:

$$e = \frac{n}{1-n} \quad (1.10)$$

$$n = \frac{e}{1+e} \quad (1.11)$$

The **specific volume (v)** is the total volume of soil which contains a unit volume of solids, i.e.

$$v = \frac{V_v}{V} = 1 + e \quad (1.12)$$

The **air content** or air voids (A) is the ratio of the volume of air to the total volume of the soil, i.e.

$$A = \frac{V_a}{V} \quad (1.13)$$

The **bulk density** or mass density (ρ) of a soil is the ratio of the total mass to the total volume, i.e.

$$\rho = \frac{M}{V} \quad (1.14)$$

Development of a mechanical model for soil

Convenient units for density are kg/m³ or Mg/m³. The density of water (1000 kg/m³ or 1.00 Mg/m³) is denoted by ρ_w .

The **specific gravity** of the soil particles (G_s) is given by

$$G_s = \frac{M_s}{V_s \rho_w} = \frac{\rho_s}{\rho_w} \quad (1.15)$$

where ρ_s is the **particle density**.

From the definition of void ratio, if the volume of solids is 1 unit then the volume of voids is e units. The mass of solids is then $G_s \rho_w$ and, from the definition of water content, the mass of water is $w G_s \rho_w$. The volume of water is thus $w G_s$. These volumes and masses are represented in Figure 1.15(b). From this figure, the following relationships can then be obtained.

The degree of saturation (definition in Equation 1.7) is

$$S_r = \frac{V_w}{V_v} = \frac{w G_s}{e} \quad (1.16)$$

The air content is the proportion of the total volume occupied by air, i.e.

$$A = \frac{V_a}{V} = \frac{e - w G_s}{1 + e} \quad (1.17)$$

or, from Equations 1.11 and 1.16,

$$A = n(1 - S_r) \quad (1.18)$$

From Equation 1.14, the bulk density of a soil is:

$$\rho = \frac{M}{V} = \frac{G_s(1+w)\rho_s}{1+e} \quad (1.19)$$

or, from Equation 1.16,

$$\rho = \frac{G_s + S_r e}{1 + e} \rho_w \quad (1.20)$$

Equation 1.20 holds true for any soil. Two special cases that commonly occur, however, are when the soil is fully saturated with either water or air. For a fully saturated soil $S_r=1$, giving:

$$\rho_{\text{sat}} = \frac{G_s + e}{1 + e} \rho_w \quad (1.21)$$

For a completely dry soil ($S_r=0$):

$$\rho_d = \frac{G_s}{1 + e} \rho_w \quad (1.22)$$

The **unit weight** or weight density (γ) of a soil is the ratio of the total weight (Mg) to the total volume, i.e.

$$\gamma = \frac{Mg}{V} = \rho g$$

Multiplying Equations 1.19 and 1.20 by g then gives

$$\gamma = \frac{G_s (1+w)}{1+e} \gamma_w \quad (1.19a)$$

$$\gamma = \frac{G_s + S_r e}{1+e} \gamma_w \quad (1.20a)$$

where γ_w is the unit weight of water. Convenient units are kN/m³, the unit weight of water being 9.81 kN/m³ (or 10.0 kN/m³ in the case of sea water).

In the case of sands and gravels the **relative density** (I_D) is used to express the relationship between the in-situ void ratio (e), or the void ratio of a sample, and the limiting values e_{\max} and e_{\min} representing the loosest and densest possible soil packing states respectively. The relative density is defined as

$$I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \quad (1.23)$$

Thus, the relative density of a soil in its densest possible state ($e = e_{\min}$) is 1 (or 100%) and in its loosest possible state ($e = e_{\max}$) is 0.

The maximum density is determined by compacting a sample underwater in a mould, using a circular steel tamper attached to a vibrating hammer: a 1-l mould is used for sands and a 2.3-l mould for gravels. The soil from the mould is then dried in an oven, enabling the dry density to be determined. The minimum dry density can be determined by one of the following procedures. In the case of sands, a 1-l measuring cylinder is partially filled with a dry sample of mass 1000 g and the top of the cylinder closed with a rubber stopper. The minimum density is achieved by shaking and inverting the cylinder several times, the resulting volume being read from the graduations on the cylinder. In the case of gravels, and sandy gravels, a sample is poured from a height of about 0.5 m into a 2.3-l mould and the resulting dry density determined. Full details of the above tests are given in BS 1377, Part 4 (1990). Void ratio can be calculated from a value of dry density using Equation 1.22. However, the density index can be calculated directly from the maximum, minimum and in-situ values of dry density, avoiding the need to know the value of G_s (see Problem 1.5).

Example 1.2

In its natural condition, a soil sample has a mass of 2290 g and a volume of 1.15×10^{-3} m³. After being completely dried in an oven, the mass of the sample is 2035 g. The value of G_s for the soil is 2.68. Determine the bulk density, unit weight, water content, void ratio, porosity, degree of saturation and air content.

Solution

$$\text{Bulk density, } \rho = \frac{M}{V} = \frac{2.290}{1.15 \times 10^{-3}} = 1990 \text{ kg/m}^3 (1.99 \text{ Mg/m}^3)$$

$$\begin{aligned} \text{Unit weight, } \gamma &= \frac{Mg}{V} = 1990 \times 9.8 = 19500 \text{ N/m}^3 \\ &= 19.5 \text{ kN/m}^3 \end{aligned}$$

$$\text{Water content, } w = \frac{M_w}{M_s} = \frac{2290 - 2035}{2035} = 0.125 \text{ or } 12.5\%$$

From Equation 1.19,

$$\begin{aligned}\text{Void ratio, } e &= G_s (1+w) \frac{\rho_w}{\rho} - 1 \\ &= \left(2.68 \times 1.125 \times \frac{1000}{1990} \right) - 1 \\ &= 1.52 - 1 \\ &= 0.52\end{aligned}$$

$$\text{Porosity, } n = \frac{e}{1+e} = \frac{0.52}{1.52} = 0.34 \text{ or } 34\%$$

$$\text{Degree of saturation, } S_r = \frac{wG_s}{e} = \frac{0.125 \times 2.68}{0.52} = 0.645 \text{ or } 64.5\%$$

$$\begin{aligned}\text{Air content, } A &= n(1-S_r) = 0.34 \times 0.355 \\ &= 0.121 \text{ or } 12.1\%\end{aligned}$$

1.7 Soil compaction

Compaction is the process of increasing the density of a soil by packing the particles closer together with a reduction in the volume of air; there is no significant change in the volume of water in the soil. In the construction of fills and embankments, loose soil is typically placed in layers ranging between 75 and 450 mm in thickness, each layer being compacted to a specified standard by means of rollers, vibrators or rammers. In general, the higher the degree of compaction, the higher will be the shear strength and the lower will be the compressibility of the soil (see Chapters 4 and 5). An **engineered fill** is one in which the soil has been selected, placed and compacted to an appropriate specification with the object of achieving a particular engineering performance, generally based on past experience. The aim is to ensure that the resulting fill possesses properties that are adequate for the function of the fill. This is in contrast to non-engineered fills, which have been placed without regard to a subsequent engineering function.

The degree of compaction of a soil is measured in terms of dry density, i.e. the mass of solids only per unit volume of soil. If the bulk density of the soil is ρ and the water content w , then from Equations 1.19 and 1.22 it is apparent that the dry density is given by

$$\rho_d = \frac{\rho}{1+w} \quad (1.24)$$

The dry density of a given soil after compaction depends on the water content and the energy supplied by the compaction equipment (referred to as the **compactive effort**).

Laboratory compaction

The compaction characteristics of a soil can be assessed by means of standard laboratory tests. The soil is compacted in a cylindrical mould using a standard compactive effort. In the **Proctor test**, the volume

of the mould is 1-1 and the soil (with all particles larger than 20 mm removed) is compacted by a rammer consisting of a 2.5-kg mass falling freely through 300 mm: the soil is compacted in three equal layers, each layer receiving 27 blows with the rammer. In the **modified AASHTO test**, the mould is the same as is used in the above test but the rammer consists of a 4.5-kg mass falling 450 mm; the soil (with all particles larger than 20 mm removed) is compacted in five layers, each layer receiving 27 blows with the rammer. If the sample contains a limited proportion of particles up to 37.5 mm in size, a 2.3-1 mould should be used, each layer receiving 62 blows with either the 2.5- or 4.5-kg rammer. In the **vibrating hammer test**, the soil (with all particles larger than 37.5 mm removed) is compacted in three layers in a 2.3-1 mould, using a circular tamper fitted in the vibrating hammer, each layer being compacted for a period of 60 s. These tests are detailed in BS1377-4 (UK), EC7-2 (Europe) and, in the US, ASTM D698, D1557 and D7382.

After compaction using one of the three standard methods, the bulk density and water content of the soil are determined and the dry density is calculated. For a given soil the process is repeated at least five times, the water content of the sample being increased each time. Dry density is plotted against water content, and a curve of the form shown in Figure 1.16 is obtained. This curve shows that for a particular method of compaction (i.e. a given compactive effort) there is a particular value of water content, known as the **optimum water content** (w_{opt}), at which a maximum value of dry density is obtained. At low values of water content, most soils tend to be stiff and are difficult to compact. As the water content is increased the soil becomes more workable, facilitating compaction and resulting in higher dry densities. At high water contents, however, the dry density decreases with increasing water content, an increasing proportion of the soil volume being occupied by water.

If all the air in a soil could be expelled by compaction, the soil would be in a state of full saturation and the dry density would be the maximum possible value for the given water content. However, this degree of compaction is unattainable in practice. The maximum possible value of dry density is referred to as the '**zero air voids' dry density** ($\rho_{\text{d}0}$) or the saturation dry density, and can be calculated from the expression:

$$\rho_{\text{d}0} = \frac{G_s}{1 + w G_s} \rho_w \quad (1.25)$$

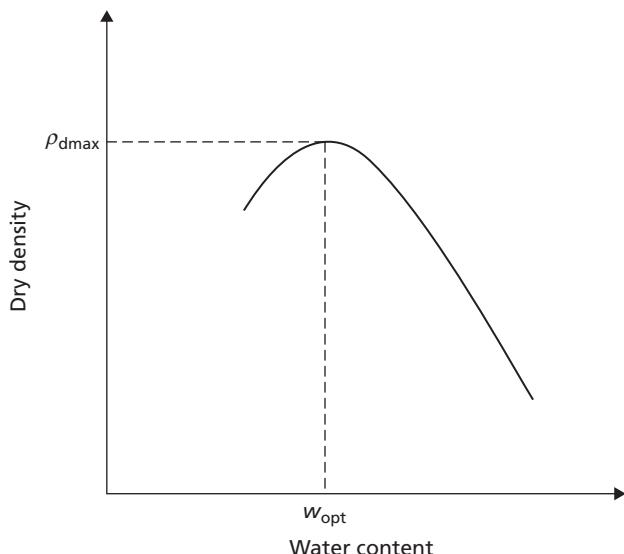


Figure 1.16 Dry density–water content relationship.

Development of a mechanical model for soil

In general, the dry density after compaction at water content w to an air content A can be calculated from the following expression, derived from Equations 1.17 and 1.22:

$$\rho_d = \frac{G_s(1-A)}{1+wG_s} \rho_w \quad (1.26)$$

The calculated relationship between zero air voids dry density ($A=0$) and water content (for $G_s=2.65$) is shown in Figure 1.17; the curve is referred to as the zero air voids line or the **saturation line**. The experimental dry density–water content curve for a particular compactive effort must lie completely to the left of the saturation line. The curves relating dry density at air contents of 5% and 10% with water content are also shown in Figure 1.17, the values of dry density being calculated from Equation 1.26. These curves enable the air content at any point on the experimental dry density–water content curve to be determined by inspection.

For a particular soil, different dry density–water content curves are obtained for different compactive efforts. Curves representing the results of tests using the 2.5- and 4.5-kg rammers are shown in Figure 1.17. The curve for the 4.5-kg test is situated above and to the left of the curve for the 2.5-kg test. Thus, a higher compactive effort results in a higher value of maximum dry density and a lower value of optimum water content; however, the values of air content at maximum dry density are approximately equal.

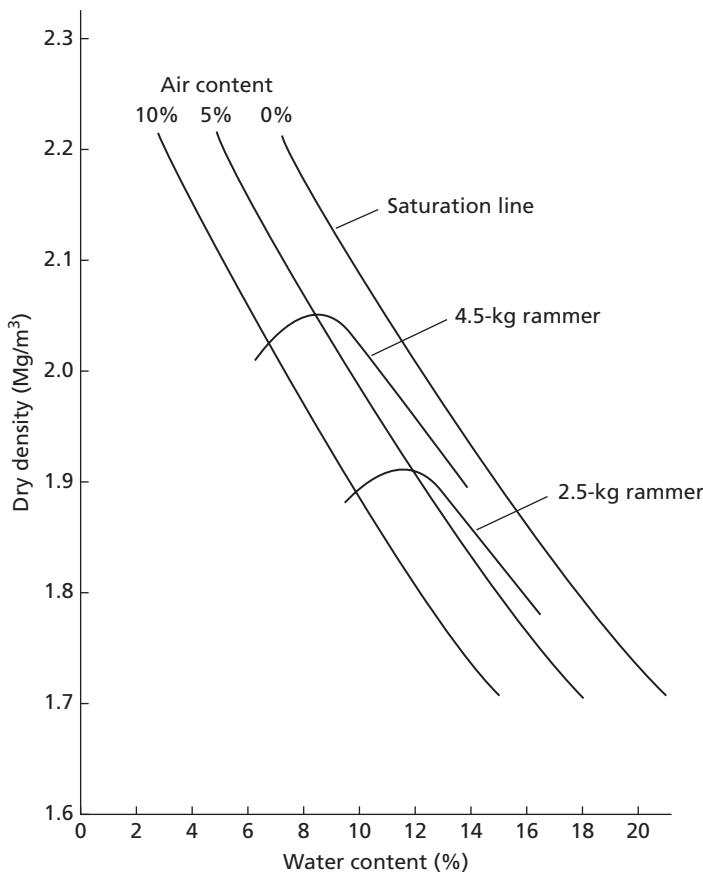


Figure 1.17 Dry density–water content curves for different compactive efforts.

The dry density–water content curves for a range of soil types using the same compactive effort (the BS 2.5-kg rammer) are shown in Figure 1.18. In general, coarse soils can be compacted to higher dry densities than fine soils.

Field compaction

The results of laboratory compaction tests are not directly applicable to field compaction because the compactive efforts in the laboratory tests are different, and are applied in a different way, from those produced by field equipment. Further, the laboratory tests are carried out only on material smaller than either 20 or 37.5 mm. However, the maximum dry densities obtained in the laboratory using the 2.5- and 4.5-kg rammers cover the range of dry density normally produced by field compaction equipment.

A minimum number of passes must be made with the chosen compaction equipment to produce the required value of dry density. This number, which depends on the type and mass of the equipment and on the thickness of the soil layer, is usually within the range 3–12. Above a certain number of passes, no significant increase in dry density is obtained. In general, the thicker the soil layer, the heavier the equipment required to produce an adequate degree of compaction.

There are two approaches to the achievement of a satisfactory standard of compaction in the field, known as **method compaction** and **end-product compaction**. In method compaction, the type and mass

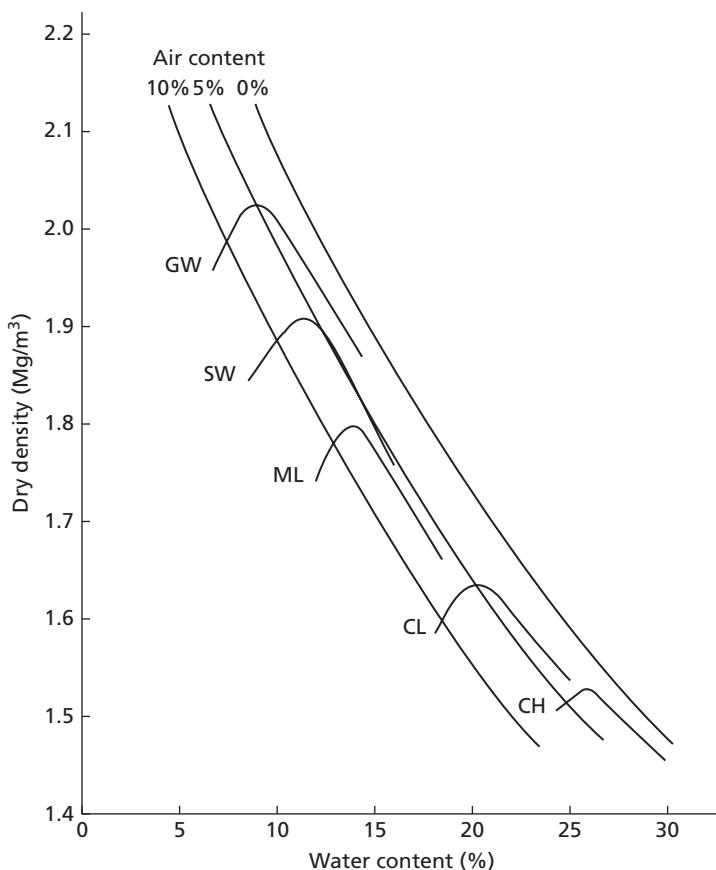


Figure 1.18 Dry density–water content curves for a range of soil types.

of equipment, the layer depth and the number of passes are specified. In the UK, these details are given, for the class of material in question, in the Specification for Highway Works (Highways Agency, 2008). In end-product compaction, the required dry density is specified: the dry density of the compacted fill must be equal to or greater than a stated percentage of the maximum dry density obtained in one of the standard laboratory compaction tests.

Field density tests can be carried out, if considered necessary, to verify the standard of compaction in earthworks, dry density or air content being calculated from measured values of bulk density and water content. A number of methods of measuring bulk density in the field are detailed in BS 1377, Part 4 (1990).

The types of compaction equipment commonly used in the field are described below, and their performance over a range of standard soils is compared in Figure 1.19. Images of all types of plant for identification purposes may be found through the Companion Website.

Smooth-wheeled rollers

These consist of hollow steel drums, the mass of which can be increased by water or sand ballast. They are suitable for most types of soil except uniform sands and silty sands, provided a mixing or kneading action is not required. A smooth surface is produced on the compacted layer, encouraging the run-off of any rainfall but resulting in relatively poor bonding between successive layers; the fill as a whole will therefore tend to be laminated. Smooth-wheeled rollers, and the other types of roller described below, can be either towed or self-propelled.

Pneumatic-tyred rollers

This equipment is suitable for a wide range of coarse and fine soils, but not for uniformly graded material. Wheels are mounted close together on two axles, the rear set overlapping the lines of the front set to ensure complete coverage of the soil surface. The tyres are relatively wide with a flat tread so that the soil is not displaced laterally. This type of roller is also available with a special axle which allows the wheels to wobble, thus preventing the bridging over of low spots. Pneumatic-tyred rollers impart a kneading action to the soil. The finished surface is relatively smooth, resulting in a low degree of bonding between layers. If good bonding is essential, the compacted surface must be scarified between layers. Increased compactive effort can be obtained by increasing the tyre inflation pressure or, less effectively, by adding additional weight (kentledge) to the body of the roller.

Sheepsfoot rollers

A sheepsfoot roller consists of hollow steel drums with numerous tapered or club-shaped feet projecting from their surfaces. The mass of the drums can be increased by ballasting. The arrangement of the feet can vary, but they are usually from 200 to 250 mm in length with an end area of 40–65 cm². The feet thus impart a relatively high pressure over a small area. Initially, when the layer of soil is loose, the drums are in contact with the soil surface. Subsequently, as the projecting feet compact below the surface and the soil becomes sufficiently dense to support the high contact pressure, the drums rise above the soil. Sheepsfoot rollers are most suitable for fine soils, both plastic and non-plastic, especially at water contents dry of optimum. They are also suitable for coarse soils with more than 20% of fines. The action of the feet causes significant mixing of the soil, improving its degree of homogeneity, and will break up lumps of stiff material, making the roller particularly suitable for re-compacting excavated clays which tend to be placed in the form of large lumps or peds. Due to the penetration of the feet, excellent bonding is produced between successive soil layers – an important requirement for water-retaining earthworks. Tamping rollers are similar to sheepsfoot rollers but the feet have a larger end area, usually over 100 cm², and the total area of the feet exceeds 15% of the surface area of the drums.

Grid rollers

These rollers have a surface consisting of a network of steel bars forming a grid with square holes. Kentledge can be added to the body of the roller. Grid rollers provide high contact pressure but little kneading action, and are suitable for most coarse soils.

Vibratory rollers

These are smooth-wheeled rollers fitted with a power-driven vibration mechanism. They are used for most soil types, and are more efficient if the water content of the soil is slightly wet of optimum. They are particularly effective for coarse soils with little or no fines. The mass of the roller and the frequency of vibration must be matched to the soil type and layer thickness. The lower the speed of the roller, the fewer the number of passes required.

Vibrating plates

This equipment, which is suitable for most soil types, consists of a steel plate with upturned edges, or a curved plate, on which a vibrator is mounted. The unit, under manual guidance, propels itself slowly over the surface of the soil.

Power rammers

Manually controlled power rammers, generally petrol-driven, are used for the compaction of small areas where access is difficult or where the use of larger equipment would not be justified. They are also used extensively for the compaction of backfill in trenches. They do not operate effectively on uniformly graded soils.

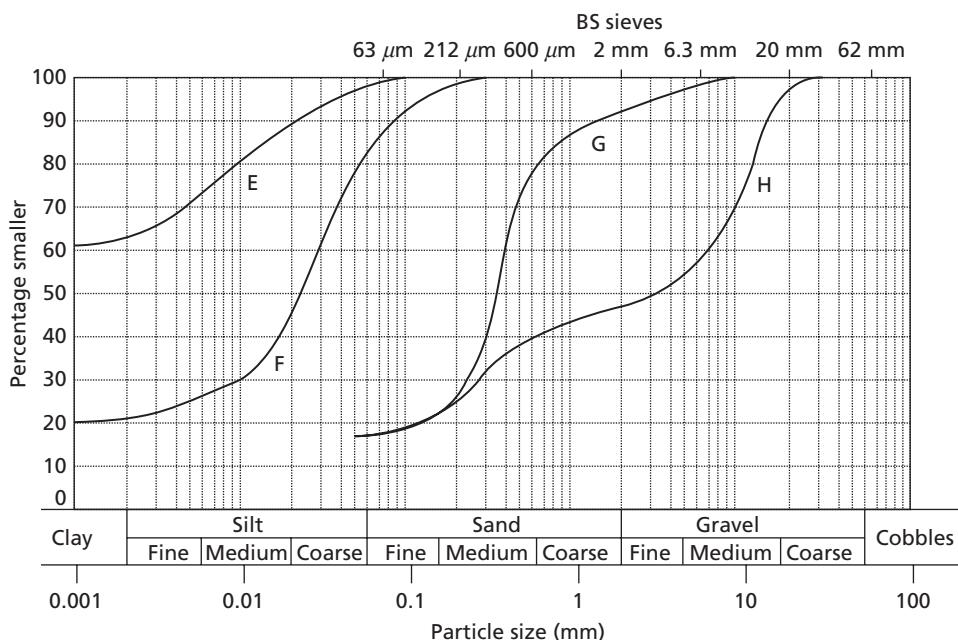


Figure 1.19(a) Performance envelopes of various compaction methods for standard soil types (replotted after Croney and Croney, 1997): PSD curves of soils.

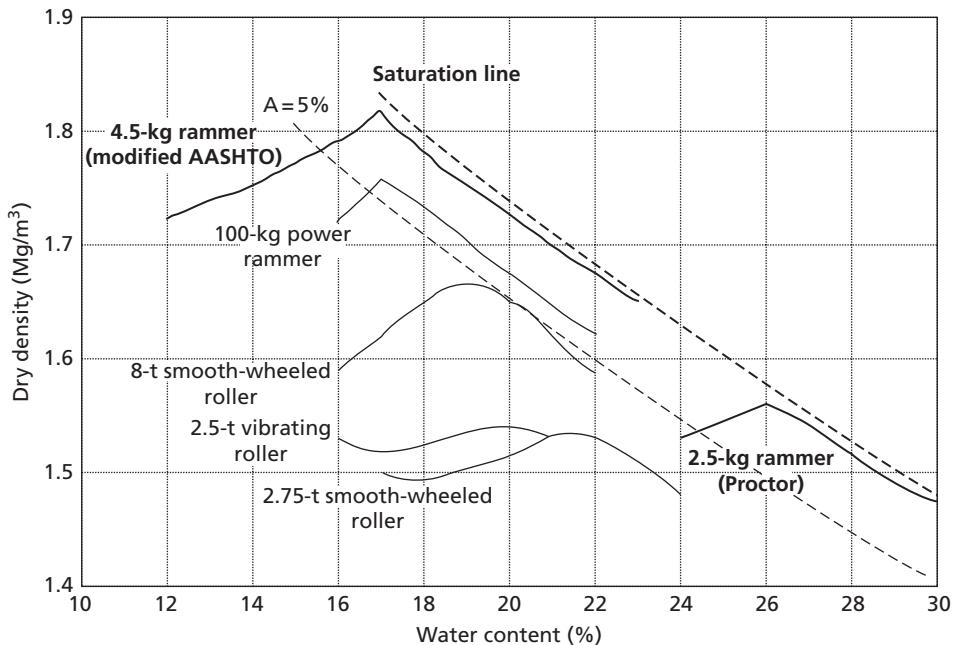


Figure 1.19(b) Performance envelopes of various compaction methods for standard soil types (replotted after Croney and Croney, 1997): Soil E.

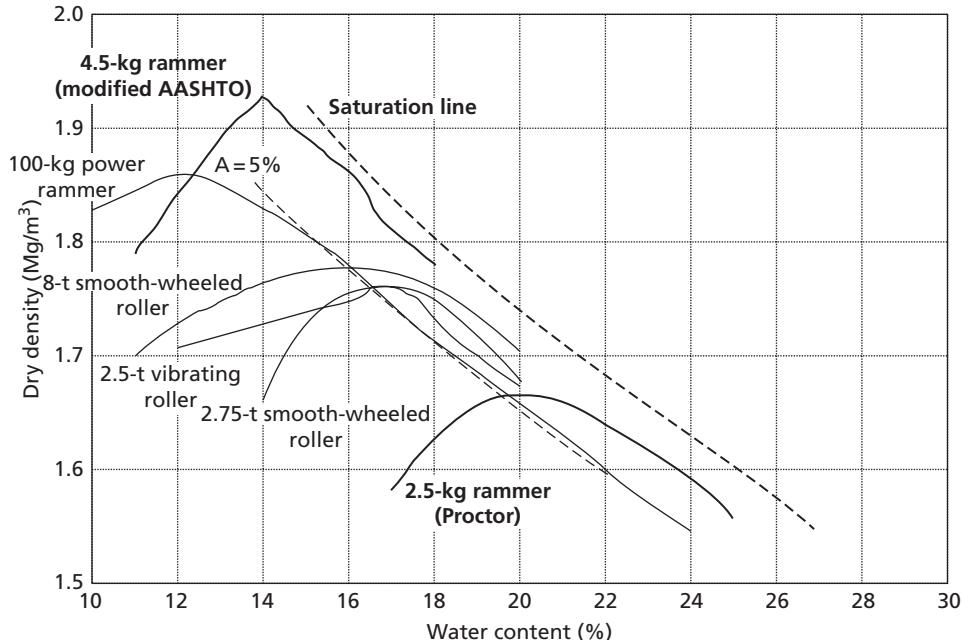


Figure 1.19(c) Performance envelopes of various compaction methods for standard soil types (replotted after Croney and Croney, 1997): Soil F.

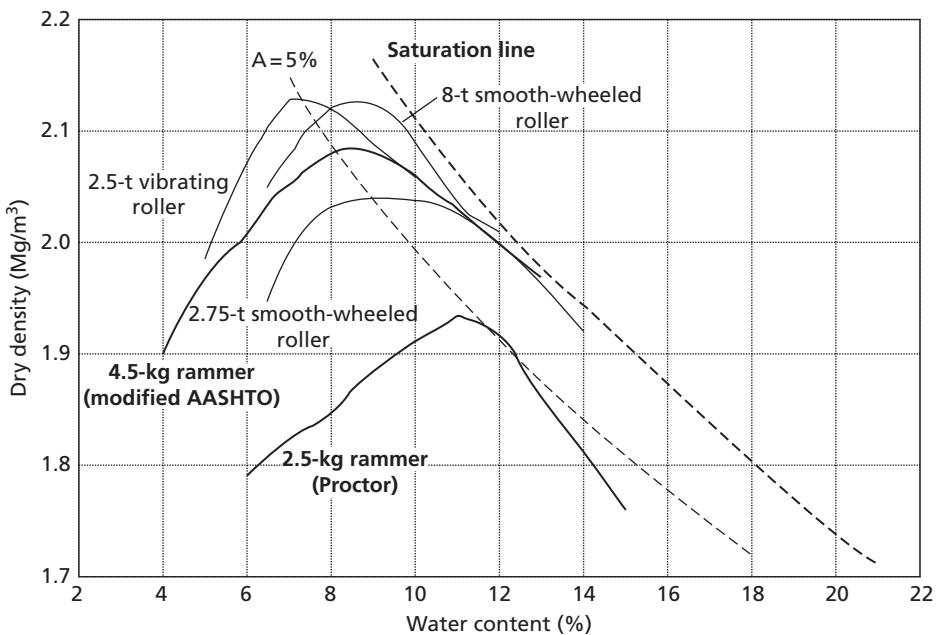


Figure 1.19(d) Performance envelopes of various compaction methods for standard soil types (replotted after Croney and Croney, 1997): Soil G.

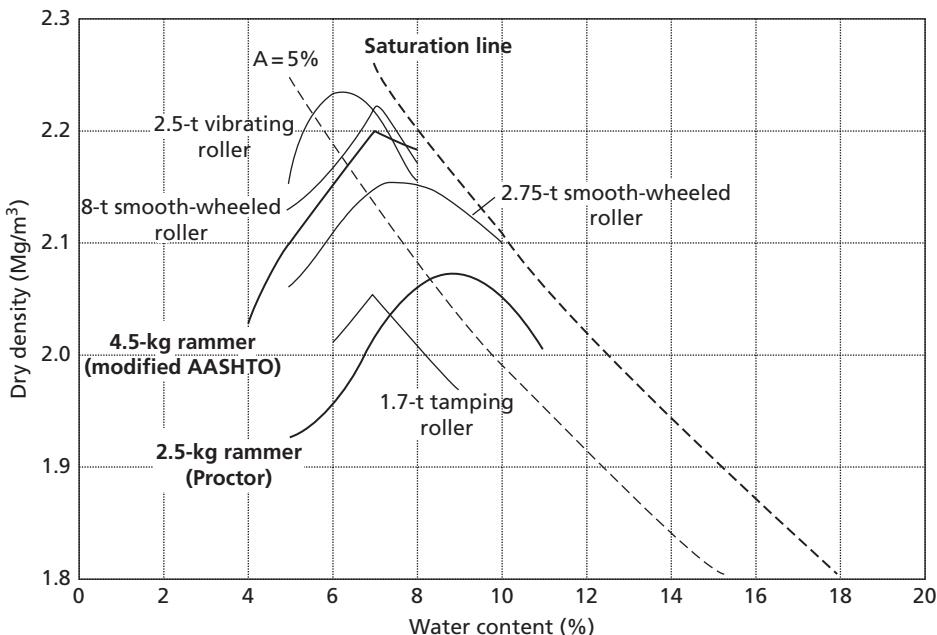


Figure 1.19(e) Performance envelopes of various compaction methods for standard soil types (replotted after Croney and Croney, 1997): Soil H.

The changed mechanical properties of a compacted soil should be determined using appropriate laboratory tests on samples taken from trial compactions in the field or from soil compacted in the laboratory (e.g. following the standard Proctor test) (see Chapters 4 and 5); alternatively in-situ tests may be conducted on the finished earthworks (Chapter 7).

Moisture condition test

As an alternative to standard compaction tests, the moisture condition test is widely used in the UK. This test, originally developed by the Transport and Road Research Laboratory (now TRL), enables a rapid assessment to be made of the suitability of soils for use as fill materials (Parsons and Boden, 1979). The test does not involve the determination of water content, a cause of delay in obtaining the results of compaction tests due to the necessity of drying the soil. In principle, the test consists of determining the effort required to compact a soil sample (normally 1.5 kg) close to its maximum density. The soil is compacted in a cylindrical mould, having an internal diameter of 10 mm, centred on the base plate of the apparatus. Compaction is imparted by a rammer having a diameter of 97 mm and a mass of 7 kg falling freely from a height of 250 mm. The fall of the rammer is controlled by an adjustable release mechanism and two vertical guide rods. The penetration of the rammer into the mould is measured by means of a scale on the side of the rammer. A fibre disc is placed on top of the soil to prevent extrusion between the rammer and the inside of the mould. Full details are given in BS 1377, Part 4 (1990).

The penetration is measured at various stages of compaction. For a given number of rammer blows (n), the penetration is subtracted from the penetration at four times that number of blows ($4n$). The change in penetration between n and $4n$ blows is plotted against the logarithm (to base 10) of the lesser number of blows (n). A change in penetration of 5 mm is arbitrarily chosen to represent the condition beyond which no significant increase in density occurs. The **moisture condition value** (MCV) is defined as ten times the logarithm of the number of blows corresponding to a change in penetration of 5 mm on the above plot. An example of such a plot is shown in Figure 1.20. For a range of soil types, it has been shown that the relationship between water content and MCV is linear over a substantial range of water content. Details of the soil types for which the test is applicable can be found in Oliphant and Winter (1997).

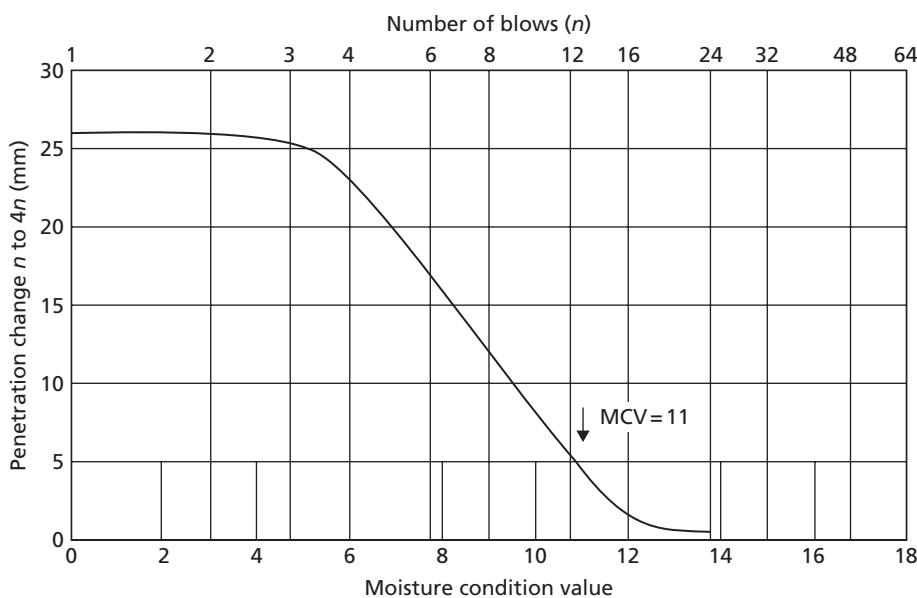


Figure 1.20 Moisture condition test.

Summary

- 1 Soil is a particulate material formed of weathered rock. The particles may be single grains in a wide range of sizes (from boulders to silt), or clay minerals (colloidal in size). Soil is typically formed from a mixture of such particles, and the presence of clay minerals may significantly alter the mechanical properties of the soil.
- 2 Soils may be described and classified by their particle size distribution. Fine soils consisting of mainly small particles (e.g. clays and silts) typically exhibit plastic behaviour (e.g. cohesion) which may be defined by the plasticity and liquidity indices. Coarse grained soils generally do not exhibit plastic behaviour.
- 3 At the level of the macro-fabric, all soils may be idealised as a three-phase continuum, the phases being solid particles, water and air. The relative proportions of these phases are controlled by the closeness of particle packing, described by the voids ratio (e), water content (w) and saturation ratio (S_r).
- 4 In addition to being used in their in-situ state, soils may be used as a fill material in geotechnical constructions. Compaction of such soils increases shear strength and reduces compressibility, and is necessary to achieve optimal performance of the fill. Compaction may be quantified using the Proctor or moisture condition tests.

Problems

- 1.1 The results of particle size analyses and, where appropriate, limit tests on samples of four soils are given in Table 1.6. Allot group symbols and give main and qualifying terms appropriate for each soil.
- 1.2 A soil has a bulk density of 1.91 Mg/m^3 and a water content of 9.5%. The value of G_s is 2.70. Calculate the void ratio and degree of saturation of the soil. What would be the values of density and water content if the soil were fully saturated at the same void ratio?

Table 1.6 Problem 1.1

BS sieve	Particle size	Percentage smaller			
		Soil I	Soil J	Soil K	Soil L
63 mm					
20 mm		100			
6.3 mm		94	100		
2 mm		69	98		
600 mm		32	88	100	
212 mm		13	67	95	100
63 mm		2	37	73	99
	0.020 mm		22	46	88
	0.006 mm		11	25	71
	0.002 mm		4	13	58
Liquid limit			Non-plastic	32	78
Plastic limit				24	31

Development of a mechanical model for soil

- 1.3 Calculate the dry unit weight and the saturated unit weight of a soil having a void ratio of 0.70 and a value of G_s of 2.72. Calculate also the unit weight and water content at a degree of saturation of 75%.
- 1.4 A soil specimen is 38 mm in diameter and 76 mm long, and in its natural condition weighs 168.0 g. When dried completely in an oven, the specimen weighs 130.5 g. The value of G_s is 2.73. What is the degree of saturation of the specimen?
- 1.5 The in-situ dry density of a sand is 1.72 Mg/m³. The maximum and minimum dry densities, determined by standard laboratory tests, are 1.81 and 1.54 Mg/m³, respectively. Determine the relative density of the sand.
- 1.6 Soil has been compacted in an embankment at a bulk density of 2.15 Mg/m³ and a water content of 12%. The value of G_s is 2.65. Calculate the dry density, void ratio, degree of saturation and air content. Would it be possible to compact the above soil at a water content of 13.5% to a dry density of 2.00 Mg/m³?
- 1.7 The following results were obtained from a standard compaction test on a soil:

TABLE C

Mass (g)	2010	2092	2114	2100	2055
Water content (%)	12.8	14.5	15.6	16.8	19.2

The value of G_s is 2.67. Plot the dry density–water content curve, and give the optimum water content and maximum dry density. Plot also the curves of zero, 5% and 10% air content, and give the value of air content at maximum dry density. The volume of the mould is 1000 cm³.

- 1.8 Determine the moisture condition value for the soil whose moisture condition test data are given below:

TABLE D

Number of blows	1	2	3	4	6	8	12	16	24	32	64	96	128
Penetration (mm)	15.0	25.2	33.0	38.1	44.7	49.7	57.4	61.0	64.8	66.2	68.2	68.8	69.7

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