Description, Properties and Behaviour of Soils and Rocks

Soil Classification

Any system of soil classification involves grouping the different soil types into categories that possess similar properties and, in so doing, providing the engineer with a systematic method of soil description. Casagrande (1948) advanced one of the first comprehensive engineering classifications of soil. In the Casagrande system, the coarse-grained soils are distinguished from the fine on the basis of particle size. Gravels and sands are the two principal types of coarse-grained soils and, in this classification, both are subdivided into five subgroups on the basis of grading (Table 5.1). Well-graded soils are those in which the particle size distribution extends over a wide range without excess or deficiency in any particular sizes, whereas in uniformly graded soils, the distribution extends over a very limited range of particle sizes. In poorly graded soils, the distribution contains an excess of some particle sizes and a deficiency of others. A plasticity chart is used when classifying fine-grained soils, that is, silts and clays (see Table 5.2).

Table 5.1. Symbols used in the Casagrande soil classification

Main soil type		Prefix
Coarse-grained soils	Gravel	G
	Sand	S
Fine-grained soils	Silt	M
	Clay	C
	Organic silts and clays	O
Fibrous soils	Peat	Pt
Subdivisions		**Suffix**
For coarse-grainedsoils	Well graded, with little or no fines	W
	Well graded with suitable clay binder	C
	Uniformly graded with little or no fines	U
	Poorly graded with little or no fines	P
	Poorly graded with appreciable fines or well graded with excess fines	F
For fine-grained soils	Low compressibility (plasticity)	L
	Medium compressibility (plasticity)	I
	High compressibility (plasticity)	H

Table 5.2. Unified Soil Classification. Coarse soils. More than half of the material is larger than No. 200 sieve size[†]

Field identification procedures (excluding particles larger than 76 mm and basing fractions on estimated weights)			Group symbols*	Typical names
Gravels. More than half of coarse fraction is larger than No. 7 sieve size*	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	GW	Well-graded gravels, gravel–sand mixtures, little or no fines
		Predominantly one size or a range of sizes with some intermediate sizes missing	GP	Poorly graded gravels, gravel–sand mixtures, little or no fines
	Gravels with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)	GM	Silty gravels, poorly graded gravel–sand–silt mixtures
		Plastic fines (for identification procedures, see CL below)	GC	Clayey gravels, poorly graded gravel–sand–clay mixtures
Sands. More than half of coarse fraction is smaller than No. 7 sieve size[‡]	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	SW	Well-graded sands, gravelly sands, little or no fines
		Predominantly one size or a range of sizes with some intermediate sizes missing	SP	Poorly graded sands, gravelly sands,little or no fines
	Sands with fines (appreciable amount of fines)	Non-plastic fines (for identification procedures, see ML below)	SM	Silty sands, poorly graded sand–silt mixtures
		Plastic fines (for identification procedures, see CL below)	SC	Clayey sands, poorly graded sand–clay mixtures

*Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW–GC, well-graded gravel–sand mixture with clay binder;

[†] All sieve sizes on this chart are US standard. The No. 200 sieve size is about the smallest particle visible to the naked eye;

[‡] For visual classification, the 6.3 mm size may be used as equivalent to the No. 7 sieve size.

Field identification procedure for fine-grained soils or fractions: These procedures are to be performed on the minus No. 40 sieve-size particles, approximately 0.4 mm. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (reacting to shaking): After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about 1 cm³. Add enough water if necessary to make the soil soft but not sticky. Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat, which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and, finally, it cracks and crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil. Very fine clean sands give the quickest and most distinct reaction, whereas a plastic clay has no reaction. Inorganic silts, such as typical rock flour, show a moderately quick reaction.

Dry strength (crushing characteristic): After removing particles larger than No. 40 sieve size, mould a pat of soil to the

Table 5.2.—Cont'd.

Information required for describing soils	Laboratory classification criteria	
Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information: and symbols in parenthesis For undistributed soils add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics Example: Silty sand, gravelly; about 20% hard, angular gravel particles 12.5 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	*Use grain size curve in identifying the fractions as given under field identification*	Determine percentages of gravel and sand from grain-size curve. Depending on fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5%: GW, GP, SW, SP. More than 12%: GM, GC, SM, SC. 5–12%: Borderline cases require use of dual symbols

$C_u = \dfrac{D_{60}}{D_{10}}$ Greater than 4

$C_e = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3

Not meeting all gradation requirements for GW

Atterberg limits below 'A' line, or *PI* less than 4	Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
Atterberg limits above 'A' line with *PI* greater than 7	

$C_u = \dfrac{D_{60}}{D_{10}}$ Greater than 6

$C_e = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3

Not meeting all gradation requirements for SW

Atterberg limits below 'A' line with *PI* less than 5	Above 'A' line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
Atterberg limits above 'A' line with *PI* greater than 7	

consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity. High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty, whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit): After removing particles larger than the No. 40 sieve size, a specimen of soil about 1 cm^3 in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about 3 mm in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation, the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays that occur below the A-line. Highly organic clays have a very weak and spongy feel at the plastic limit.

Table 5.2.—Cont'd. Unified Soil Classification. Fine soils. More than half of the material is smaller than No. 200 sieve size[b]

Identification procedures on fraction smaller than No. 40 sieve size			Group symbols[a]	Typical names
	Dry strength* (crushing characteristics)	Dilatancy* (reaction to shaking)	Toughness* (consistency near plastic limit)		

	Dry strength*	Dilatancy*	Toughness*	Group symbols[a]	Typical names
Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity
	Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity
Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts micaceous or diatomaceous fine sandy or silty soils, clastic silts[b]
	High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays
	Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity
Highly organic soils	Readily identified by colour, odour, spongy feel, and frequently by fibrous texture			Pt	Peat and other highly organic soils

*See footnotes to Table

Table 5.2.—Cont'd.

| Information required for describing soils	Laboratory classification criteria

Give typical name: indicate degree and character of plasticity, amount and maximum size of coarse grains, colour in wet conditions, odour if any, local or geological name, and other pertinent descriptive information and symbol in parentheses

For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions

Example:
Clayey silt, brown: slightly plastic; small percentage of fine sand, numerous vertical root holes; firm and dry in place; loess; (ML)

Use grain-size curve in identifying the fractions as given under field identification
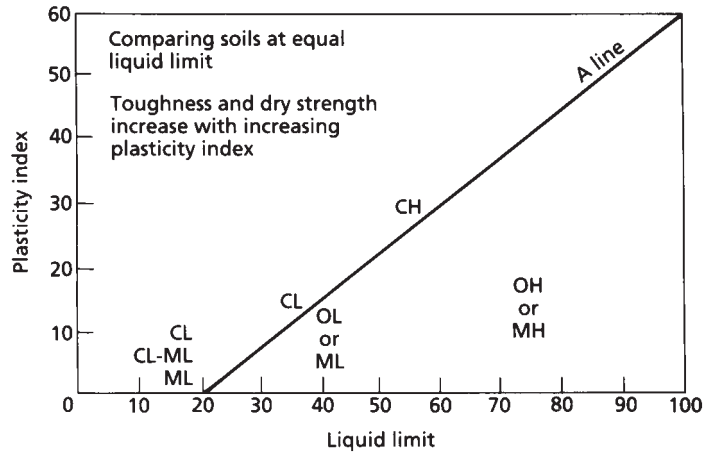
Plasticity chart for laboratory classification of fine-grained soils

On this chart, the plasticity index is plotted against liquid limit. The A line is taken as the boundary between organic and inorganic soils, the latter lying above the line. Each of the main soil types and subgroups are given a letter, a pair of which are combined in the group symbol, the former being the prefix, the latter the suffix. Subsequently, the Unified Soil Classification system was developed from the Casagrande system.

According to Anon (1999), a full description of a soil should provide data on its particle size, plasticity, particle characteristics and colour, as well as its bedding, discontinuities and strength. Anon divides soils into coarse and fine types. Coarse soils are gravels and sands, and fine soils are silts and clays.

Boulders, cobbles, gravels, sands, silts and clays are distinguished as individual groups on the basis of their particle size distribution (Table 5.3a). Gravel, sand and silt have been subdivided into coarse-, medium- and fine-grained subgroups, and fine soils have been subdivided on the basis of plasticity. Coarse soils are described as well graded or poorly graded. Two further types of poorly graded coarse soils are recognized, namely, uniformly graded and gap graded (Fig. 1.18). Silts and clays are subdivided according to their liquid limits (Table 5.3b).

Most soils consist of more than one grade size type. If boulders and cobbles are present in composite soil types, then they are removed before an attempt is made at classification, their proportions being recorded separately. Their presence should be recorded in the soil description. Very coarse deposits should be described as boulders if over half of the very coarse material is of boulder size. They may be described as cobbly boulders if cobbles are an

Table 5.3a. Particle size distribution of soils

Types of material		Sizes (mm)
Boulders		Over 200
Cobbles		60–200
Gravel	Coarse	20–60
	Medium	6–20
	Fine	2–6
Sand	Coarse	0.6–2
	Medium	0.2–0.6
	Fine	0.06–0.2
Silt	Coarse	0.02–0.06
	Medium	0.006–0.02
	Fine	0.002–0.006
Clay		Less than 0.002

Table 5.3b. Plasticity according to liquid limit

Description	Plasticity	Range of liquid limit
Lean or silty	Low plasticity	Less than 35
Intermediate	Intermediate plasticity	35–50
Fat	High plasticity	50–70
Very fat	Very high plasticity	70–90
Extra fat	Extra high plasticity	Over 90

important second constituent in the very coarse fraction. If over half of the very coarse material is of cobble size, then it is described as cobbles. Similarly, it may be described as bouldery cobbles if boulders are an important second constituent in the very coarse fraction. Mixtures of the very coarse material and soil can be described by combining the terms for the very coarse constituent and the soil constituent as shown in Table 5.4.

According to Anon (1999), further factors that should be incorporated in a soil description, to help identification of soil, include the following:

1.   Mass characteristics
    (a) Field strength or compactness and indication of moisture condition. A scale for estimating the strength of clays is given in Table 5.5, however, where assessment of strength is important, appropriate testing should be undertaken. The relative densities of sands and gravels may be determined by the standard penetration test (see Chapter 7) that, in turn, can be related to their angle of friction (Table 7.3).
    (b) The thickness of the bedding should be described as indicated in Table 2.1. Where beds are too thin to be described individually, they may be referred to as interbedded or interlaminated.
    (c) Discontinuities include joints, fissures, and shear surfaces. Their orientation, spacing, persistence, openness and surface texture (i.e. rough, smooth, polished, striated) should be described (see Chapter 2).

Table 5.4. Mixtures of very coarse materials and soil

Description	Composition
Boulders (or cobbles) with a little finer material*	Up to 5% finer material
Boulders (or cobbles) with some finer material*	5–20% finer material
Boulders (or cobbles) with much finer material*	20–50% finer material
Finer material* with many boulders (or cobbles)	50–20% boulders (or cobbles)
Finer material* with some boulders (or cobbles)	20–5% boulders (or cobbles)
Finer material* with occasional boulders (or cobbles)	Up to 5% boulders (or cobbles)

*Give the name of the finer material, e.g. Gravel with occasional boulders; cobbly boulders with some finer material (sand with some fines).

Table 5.5. Consistency of fine soils

Description	Consistency index (IC)	Approximate undrained shear strength (kPa)	Field identification
Hard	—	Over 300	Indented with difficulty by thumbnail, brittle
Very stiff	Above 1	150–300	Readily indented by thumbnail, still very tough
Stiff	0.75–1	75–150	Readily indented by thumb but penetrated only with difficulty. Cannot be moulded in the fingers
Firm	0.5–0.75	40–75	Can be penetrated several centimetres by thumb with moderate effort, and moulded in the fingers by strong pressure
Soft	Less than 0.5	20–40	Easily penetrated several centimetres by thumb, easily moulded
Very soft	—	Less than 20	Easily penetrated several centimetres by fist, exudes between fingers when squeezed in fist

$IC = \dfrac{LL - m}{LL - PL}$, where m = moisture content, PL = plastic limit and LL = liquid limit.

2. Material characteristics
   (a) The colour should relate to that of the overall impression of the soil. Soil with more than one colour can be described as mottled or multicoloured. A colour chart such as the Munsell chart should be used to help describe the colour of soils.
   (b) Particle shape, particle grading and composition. In particular instances, it may be necessary to describe the shape of soil particles (see Fig. 1.16). Grading of coarse soils has been referred to earlier.
   (c) Soil name (in capitals, e.g. SAND) is based on grading and plasticity. A coarse soil (omitting any boulders or cobbles) contains around 65% or more coarse material and is referred to as SAND or GRAVEL, according to the size fraction that predominates. Gravel and sand are further divided into coarse, medium and fine categories. Mixtures of coarse soil types can be described as shown in Table 5.6. Anon (1999) recommended that fine soil should be described as SILT or CLAY

Table 5.6. Mixed coarse soil types

Description	Composition of the coarse fraction
Slightly sandy gravel	Up to 5% sand
Sandy gravel	5–20% sand
Very sandy gravel	Over 20% sand
Gravel/sand	About equal proportions of gravel and sand
Very gravelly sand	Over 20% gravel
Gravelly sand	5–20% gravel
Slightly gravelly sand	Up to 5% gravel

depending on its plastic properties (although it cautioned against the use of the A-line on the plasticity chart as a reliable method of distinguishing between silts, which are supposed to plot below, and clays that plot above). It also was recommended that the terms should be mutually exclusive. In other words, terms such as silty clay were regarded as redundant. Field identification of fine soils can be made according to dilatancy, dry strength and toughness tests (see Table 5.2). The terms outlined in Table 5.7 can be used to describe common soils that include a mixture of soil types.

3. Geological formation, age and type of deposit.
4. Classification (optional).

When small amounts of organic matter occur throughout a soil, they can have a notable effect on plasticity and therefore the engineering properties. Increase in the quantities of organic matter can increase these effects. Nonetheless, soils in which the organic contents may be up to 30%, by weight, behave primarily as mineral soils.

Table 5.7. Description common types of soils

Description	Main soil type	Approximate proportion of secondary constituent	
		Coarse soil	Coarse or fine soil
Slightly clayey or silty and/or sandy or gravelly	Sand	—	>5%
Clayey or silty and/or sandy or gravelly	or	—	5–20%
Very clayey or silty and/or sandy or gravelly	Gravel	—	>20%
Very sandy or gravelly	Silt	>65%	
Sandy and/or gravelly	or	35– 65%	—
Slightly sandy and/or gravelly	Clay	<35%	

Table 5.8. Some values of gravels, sands and silts

	Gravels	Sands	Silts
Specific gravity	2.5–2.8	2.6–2.7	2.64–2.66
Bulk density (Mg m^{-3})	1.45–2.3	1.4–2.15	1.82–2.15
Dry density (Mg m^{-3})	1.4–2.1	1.35–1.9	1.45–1.95
Porosity (%)	20–50	23–35	—
Void ratio	—	—	0.35–0.85
Liquid limit (%)	—	—	24–35
Plastic limit (%)	—	—	14–25
Coefficient of consolidation (m^2 yr^{-1})	—	—	12.2
Cohesion (kPa)	—	—	75
Angle of friction (deg)	35–45	32–42	32–36

Peat is an accumulation of plant remains that has undergone some degree of decomposition. Inorganic soil material may occur as secondary constituents in peat, and should be described, for example, as slightly clayey or very sandy.

Coarse Soils

The microstructure of sand or gravel refers to its particle arrangement that, in turn, involves its packing. If grains approximate to spheres, then the closest type of systematic packing is rhombohedral packing, whereas the most open type is cubic packing, the porosities approximating to 26 and 48%, respectively. Put another way, the void ratio of a well-sorted and perfectly cohesionless aggregate of equidimensional grains can range between values of about 0.35 and 1.00. If the void ratio is more than unity, the microstructure will be collapsible or metastable. Some values of the physical properties of sands and gravels are given in Table 5.8.

Grain size and sorting have a significant influence on the engineering behaviour of coarse soils. Generally, the larger the particles, the higher the strength, and deposits consisting of a mixture of different-sized particles usually are stronger than those that are uniformly graded. For example, the amount of gravel in a sand–gravel mixture has a significant effect on shear strength, which increases considerably as the gravel content is increased up to 50 or 60%. Beyond this point, the material becomes less well graded, and the density does not increase. The density of a soil is governed by the manner in which its solid particles are packed. For instance, coarse soils may be densely or loosely packed. Densely packed sands are almost incompressible, whereas loosely packed deposits, located above the water table, are relatively compressible but otherwise stable. In other words, the behaviour of such sediments depends, to a large extent, on their relative density. Indeed, a maximum and minimum density can be distinguished. The smaller the range of particle sizes present and the more angular the particles, the smaller the minimum density. Conversely, if a wide range of particle sizes is present, the void space is

reduced accordingly, hence the maximum density is higher. A useful way to characterize the density of a coarse-grained soil is by its relative density, D_r, which is defined as:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

(5.1)

where e is the naturally occurring void ratio, e_{max} is the maximum void ratio and e_{min} is the minimum void ratio. If the relative density of sand varies erratically, this can give rise to differential settlement. Generally, settlement in sands is relatively rapid. However, when the stresses are large enough to produce appreciable grain fracturing, there is a significant time lag.

Fundamentally, there are two basic mechanisms that contribute towards the deformation of coarse soil, namely, distortion of the particles and the relative motion between them. These two mechanisms usually are interdependent. At any instant during the deformation process, different mechanisms may be acting in different parts of the soil, and these may change as deformation continues. Interparticle sliding can occur at all stress levels, the stress required for its initiation increasing with decreasing void ratio. Crushing and fracturing of particles begins in a minor way at small stresses, becoming increasingly important when some critical stress is reached. This critical stress is smallest when the soil is loosely packed and uniformly graded, and consists of large angular particles with a low strength. Usually, fracturing only becomes important when the stress level exceeds 3.5 MPa.

The internal shearing resistance of a coarse soil is generated by friction when the grains in the zone of shearing are caused to slide, roll and rotate against each other. The angle of shearing resistance is influenced by the grain size distribution and grain shape. The larger the grains, the wider is the zone affected. The more angular the grains are, the greater the frictional resistance to their relative movement, since they interlock more thoroughly than do rounded ones. Therefore, they produce a larger angle of shearing resistance (Table 5.9).

Figure 5.1 shows that dense sand has a high peak strength and that when it is subjected to shear stress it expands up to the point of failure, after which a slight decrease in volume may occur. Conversely, loose sand compacts under shearing stress, and its residual strength may be similar to that of dense sand. Both curves in Figure 5.1 exhibit strains that are approximately

Table 5.9. Effect of grain shape and grading on the peak friction angle of sand

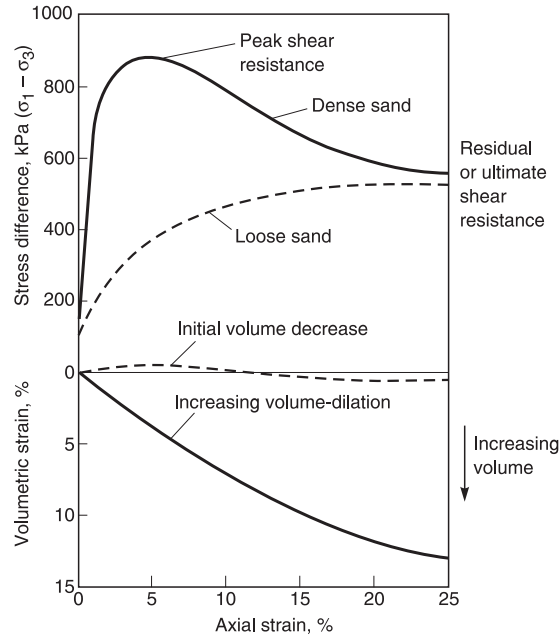| Shape and grading	Loose	Dense
Rounded, uniform	30°	37°
Rounded, well graded	34°	40°
Angular, uniform	35°	43°
Angular, well graded	39°	45°

Figure 5.1

Stress–strain curves for dense and loose sand.

proportional to stress at low stress levels, suggesting a large component of elastic distortion. If the stress is reduced, the unloading stress–strain curve indicates that not all the strain is recovered on unloading. The hysteresis loss represents the energy lost in crushing and repositioning of grains. At higher shear stresses, the strains are proportionally greater, indicating greater crushing.

The presence of water in the voids of a coarse soil usually does not produce significant changes in the value of the angle of shearing resistance. However, if stresses develop in the pore water, they may bring about changes in the effective stresses between the particles, whereupon the shear strength and the stress–strain relationships may be altered radically.

Barton et al. (1993) distinguished between normal sands and those that had undergone a notable degree of diagenetic alteration. They regarded normal sands as those that did not possess any cohesion derived from grain interlock or cementation. Any cohesion possessed by these normal sands was due to the presence of a clay matrix. If such soils do not have a clay fraction, then they are cohesionless. Normal sands grade into diagenetically altered sands. Barton et al. recognized three groups of diagenetically altered sands, namely, locked sands, overlocked sands and slightly cemented sands. Locked sands show no visible bonding and, although trace amounts of cement may be present, their effect on strength is negligible. The cohesion of locked sands is derived from grain overgrowths. Cemented sands possess

enough cement to develop cohesion but still break down into their component grains. At low stress levels, locked sands undergo high rates of dilation. Dilatancy becomes suppressed as the level of stress increases, since the asperities on the surfaces of individual grains are sheared through rather than causing dilation. They have peak frictional strengths considerably in excess of those of dense sand, with residual angles of friction varying between 30 and 35°.

Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance. The basic cause responsible for the lique-faction of saturated sands is the build-up of excess pore water pressure due to either cyclic or shock loading of the sand. As a result, the grains of sand are compacted, with a conse-quent transfer of stress to the pore water and a reduction of stress on the sand grains. If drainage cannot take place, then the decrease in volume of the grains causes an increase in pore water pressure. If the pore water pressure builds up to the point where it is the same as the overburden pressure, then the effective stress is reduced to zero and the sand loses strength with a liquefied state developing. In loose sands the pore water pressure can increase rapidly to the value of the overburden or confining pressure. If the sand undergoes more or less unlimited deformation without mobilizing any notable resistance to deformation, then it can be described as having liquefied. However, Norris et al. (1998) pointed out that a loose sand does not lose all strength during liquefaction. Loose sands at low confining pressure, and medium and dense sands undergo only limited deformation due to dilation once initial liquefaction has occurred. Such response is referred to as ranging from limited liquefaction (in the case of loose and medium dense sands at low confining pressure) to dilative behaviour (in dense sands).

Silts and Loess

The grains in a deposit of silt often are rounded with smooth outlines. This influences their degree of packing. The latter, however, is more dependent on the grain size distribution within a silt deposit, uniformly sorted deposits not being able to achieve such close packing as those in which there is a range of grain size. This, in turn, influences the porosity and void ratio values, as well as bulk and dry densities (Table 5.8).

Dilatancy is characteristic of fine sands and silts. The environment is all important for the development of dilatancy since conditions must be such that expansion can take place. What is more, it has been suggested that the soil particles must be well wetted, and it appears that certain electrolytes exercise a dispersing effect, thereby aiding dilatancy. The moisture con-tent at which a number of fine sands and silts from British formations become dilatant usually varies between 16 and 35%.

Consolidation of silt is influenced by grain size, particularly the size of the clay fraction, porosity and natural moisture content. Primary consolidation may account for over 75% of total consolidation. In addition, settlement may continue for several months after construction is completed because the rate at which water can drain from the voids under the influence of applied stress is slow.

The angle of shearing resistance decreases with increasing void ratio. It also is dependent on the plasticity index, grain interlocking and density.

Most loess is of aeolian origin. Wind-blown deposits of loess are characterized by a lack of stratification and uniform sorting, and occur as blanket deposits. Loess also is a remarkably uniform soil in terms of its dominant minerals. In other words, loess deposits have similar grain size distribution and mineral composition, as well as open texture, low degree of saturation, and bonding of grains that is not resistant to water. The fabric of loess takes the form of a loose skeleton built of grains (generally quartz) and micro-aggregates (assemblages of clay or clay and silty clay particles). The silt-sized particles are sub-angular and sub-rounded, and separate from each other, being connected by bonds and bridges, with uniformly distributed pores. The bridges are formed of clay-sized materials, be they clay minerals, fine quartz, feldspar or calcite. These clay-sized materials also occur as coatings to grains. Silica and iron oxide may be concentrated as cement at grain contacts, and amorphous overgrowths of silica occur on grains of quartz and feldspar. As silt-sized particles are not in contact, the mechanical behaviour of loess is governed by the structure and quality of the bonds.

Loess may exhibit sub-vertical columnar jointing. In addition, pipe systems may be developed in loess soils. Extensive pipe systems and sinkholes are present in some loess and have been referred to a loess karst. Pipes tend to develop by weathering and widening that takes place along the joint systems in loess. The depths to which pipes develop may be inhibited by changes in permeability associated with the occurrence of palaeosols.

Loess, as noted earlier, owes its engineering characteristics largely to the way in which it was deposited since this gives it a metastable structure, in that initially the particles are loosely packed. The porosity of the structure is enhanced by the presence of fossil root-holes. The latter are lined with carbonate cement, which helps bind the grains together. However, the chief binder is usually the clay matrix. On wetting, the clay bond in many loess soils becomes soft, which leads to the collapse of the metastable structure. The breakdown of the soil structure occurs under its own weight.

Loess deposits generally consist of 50–90% particles of silt size. In fact, sandy, silty and clayey loess can be distinguished (Fig. 5.2; Table 5.10). The range of dry density is very low to low (e.g. in Chinese loess, it may vary from 1.4 to 1.5 Mg m^{-3}). The low density is reflected
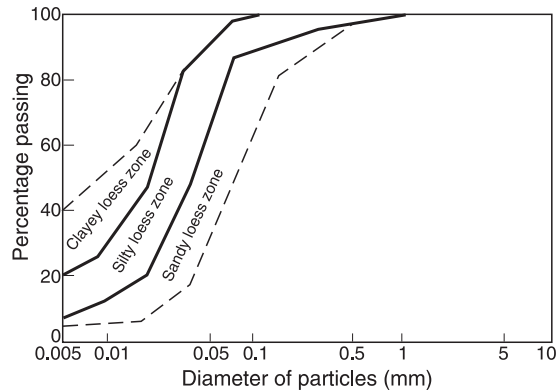
214

Figure 5.2

Particle size distribution of Missouri River Basin loess.

Table 5.10. Some geotechnical properties of loess soils

Property	Shaansi Province, China*		Lanzhow Province, China[+]	Czechos-lovakia, near Prague[†]	South Polish Uplands**
	Sandy loess	**Clayey loess**			
Natural moisture content (%)	9–13	13–20	11–10	21	3–26
Specific gravity					2.66–2.7
Bulk density (Mg m^{-3})	1.59–1.68	1.4–1.85			1.54–2.12
Dry density (Mg m^{-3})			1.4–1.5		1.46–1.73
Void ratio	0.8–0.92	0.76–1.11	1.05		
Porosity (%)				44–50	35–46
Grain-size distribution (%)					
Sand	20.5–35.2	12–15	20–24		
Silt	54.8–69.0	64–70	57–65		
Clay	8.0–15.5	17–24	16–21		
Plastic limit (%)			10–14	20	
Liquid limit (%)	26–28	30–31	27–30	36	
Plasticity index (%)	8–10	11–12	10–14	16	
Activity				1.32	
Coefficient of collapsibility	0.007–0.016	0.003–0.023		0.006–0.011	0.0002–0.06
Angle of friction					7–36°

*From Lin and Wang (1988).
[+]From Tan (1988).
[†]From Feda (1988).
**From Grabowska-Olszewska (1988).

in the void ratio and porosity. In the case of some Chinese loess, the void ratio varies from 0.81 to 0.89 and the porosity from 45–47%. Lutenegger and Hallberg (1988) observed that the bulk densities of unstable loess (such as the Peorian Loess, United States), tend to range between 1.34 and 1.55 Mg m^{-3}. If this material is wetted or consolidated (or reworked), the density increases, sometimes to as high as 1.6 Mg m^{-3} (Clevenger, 1958).

The liquid limit of loess averages about 30% (exceptionally, liquid limits as high as 45% have been recorded), and their plasticity index ranges from about 4 to 9%, but averages 6%. As far as their angle of shearing resistance is concerned, this usually varies from 30 to 34°. Loess deposits are better drained (their permeability ranges from 10^{-5} to 10^{-7} m s^{-1}) than are true silts because of the fossil root-holes. As would be expected, their permeability is appreciably higher in the vertical than in the horizontal direction.

Normally, loess possesses a high shearing resistance and can carry high loadings without significant settlement when natural moisture contents are low. For instance, moisture contents of undisturbed loess are generally around 10%, and the supporting capacity of loess at this moisture content is high. However, the density of loess is the most important factor controlling its shear strength and settlement. On wetting, large settlements and low shearing resistance are encountered when the density of loess is below 1.30 Mg m^{-3}, whereas if the density exceeds 1.45 Mg m^{-3}, settlement is small and shearing resistance fairly high.

Unlike silt, loess does not appear to be frost susceptible, this being due to its more permeable character, but it can exhibit quick conditions as with silt and it is difficult, if not impossible, to compact. Because of its porous structure, a "shrinkage" factor must be taken into account when estimating earthwork.

Several collapse criteria have been proposed that depend on the void ratios at the liquid limit, e_1, and the plastic limit, e_p, and the natural void ratio, e_o. Fookes and Best (1969) proposed a collapse index, i_c, which involved these void ratios, which is as follows:

$$i_c = \frac{e_o - e_p}{e_l - e_p} \tag{5.2}$$

Previously, Feda (1966) had proposed the following collapse index:

$$i_c = \frac{m/S_r - PL}{PI} \tag{5.3}$$

in which m is the natural moisture content, S_r is the degree of saturation, PL is the plastic limit and PI is the plasticity index. Feda also proposed that the soil must have a critical porosity of 40% or above and that an imposed load must be sufficiently high to cause structural collapse when the soil is wetted. He suggested that if the collapse index was greater than 0.85, then

this was indicative of metastable soils. However, Northmore et al. (1996) suggested that a lower critical value of collapse index, that is 0.22, was more appropriate for some loess type soils in Essex, England. The double oedometer test also can be used to assess the degree of collapsibility. The test involves loading an undisturbed specimen at natural moisture content up to a given load. At this point, the specimen is flooded and the resulting collapse strain, if any, is recorded. Then, the specimen is subjected to further loading. The total consolidation upon flooding can be described in terms of the coefficient of collapsibility, C_{col}, given as:

$$C_{col} = \Delta h/h$$
$$= \frac{\Delta e}{1+e} \tag{5.4}$$

in which Δh is the change in height of the specimen after flooding, h is the height of the specimen before flooding, Δe is the change in void ratio of the specimen upon flooding and e is the void ratio of the specimen prior to flooding. Table 5.11 provides an indication of the potential severity of collapse. This table indicates that those soils that undergo more than 1% collapse can be regarded as metastable. However, in China a figure of 1.5% is taken (Lin and Wang, 1988), and, in the United States, values exceeding 2% are regarded as indicative of soils susceptible to collapse (Lutenegger and Hallberg, 1988).

Clay Deposits

Clay deposits are composed principally of fine quartz and clay minerals. The three major clay minerals are kaolinite, illite and montmorillonite. Both kaolinite and illite have non-expansive lattices, whereas that of montmorillonite is expansive. In other words, montmorillonite is characterized by its ability to swell and by its notable cation exchange properties.

The microstructure of clay soils is governed largely by the clay minerals present and the forces acting between them. Because of the complex electrochemistry of clay minerals, the spatial arrangement of newly sedimented particles is influenced by the composition of the water in which deposition takes place. Single clay mineral platelets may associate in an edge-to-edge (EE), edge-to-face (EF), face-to-face (FF) or random type of arrangement, depending on the

Table 5.11. Collapse percentage as an indication of potential problems

Collapse (%)	Severity of problem
0–1	No problem
1–5	Moderate trouble
5–10	Trouble
10–20	Severe trouble
Above 20	Very severe trouble

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interparticle balance between the forces of attraction and repulsion, and the amount or absence of turbulence in the water in which deposition occurs. The original microstructure of a clay deposit is modified subsequently by overburden pressures due to burial, which bring about consolidation. Consolidation tends to produce a preferred orientation with the degree of reorientation of clay particles being related to both the intensity of stress and the electrochemical environment, dispersion encouraging and flocculation discouraging clay particle parallelism. These microstructures are destroyed by weathering, gradually disappearing as the degree of weathering intensifies, as Coulthard and Bell (1993) found in the Lower Lias Clay in Gloucester, England.

The principal minerals in a deposit of clay tend to influence its index properties. For example, the plasticity of clay soil is influenced by the amount of its clay fraction and the type of clay minerals present since clay minerals influence the amount of attracted water held in a soil. Burnett and Fookes (1974), for instance, demonstrated that the clay fraction of the London Clay in the London Basin increases eastwards that, in turn, leads to an increase in its plasticity. Similarly, Bell (1994b) showed that high plasticity in the Speeton Clay, East Yorkshire, was influenced by the proportion of clay fraction present. Subsequently, Marsh and Greenwood (1995) noted that as the calcite content in the Gault Clay, England, increased, the liquid limit decreased. On the other hand, it would appear that there is only a general correlation between the clay mineral composition of a deposit and its activity. In other words, kaolinitic and illitic clays usually are inactive, whereas montmorillonitic clays range from inactive to active. Usually, active clays have a relatively high water-holding capacity and a high cation exchange capacity. They also are highly thixotropic, have low permeability and have low resistance to shear. The activity of clay was defined by Skempton (1953) as:

$$\text{Activity} = \frac{\text{Plasticity index}}{\text{Percentage by mass finer than 0.002 mm}} \qquad (5.5)$$

He suggested three classes of activity, namely, active, normal and inactive, which he further subdivided into five groups as follows:

1. Inactive with activity less than 0.5,
2. Inactive with activity range 0.5–0.75,
3. Normal with activity range 0.75–1.25,
4. Active with activity range 1.25–2,
5. Active with activity greater than 2.

Particle size analyses of clay deposits indicate that they can contain appreciable fractions of grains larger than 0.002 mm. For example, Forster et al. (1994) reported that the clay fraction in the Gault Clay lay between 15 and 65% and that particles larger than 2 mm constituted less than 1%. The proportions of clay, silt and sand size material in the Claygate Beds and Bagshot Beds of south Essex, England, as recorded by Northmore et al. (1999), are given in Table 5.12.

Table 5.12. Particle size distribution in the clay deposits of the Claygate Beds and Bagshot Beds of South Essex (after Northmore et al.,1999)

	Clay (%)	Silt (%)	Sand (%)
Upper Claygate Beds	50–57 53.5	40–42 41	3–8 5.5
Middle Claygate Beds	51–54 52.8	36–46 42	3–10 5.2
Lower Claygate Beds	52–61 55.2	36–45 42.3	2–3 2.5
Bagshot Beds	53–69 61.2	28–41 35.2	1–15 3.6

The undrained shear strength is related to the amount and type of clay minerals present in a clay deposit, together with the presence of cementing agents. In particular, strength is reduced with increasing content of mixed-layer clay and montmorillonite in the clay fraction. The increasing presence of cementing agents, especially calcite, enhances the strength of the clay.

Geological age also has an influence on the engineering behaviour of a clay deposit. In particular, the porosity, moisture content and plasticity normally decrease in value with increasing depth and thereby age, whereas the strength and elastic modulus increase.

The engineering performance of clay deposits also is affected by the total moisture content and by the energy with which this moisture is held. For instance, the moisture content influences their consistency and strength, and the energy with which moisture is held influences their volume change characteristics. Indeed, one of the most notable characteristics of clays from the engineering point of view is their susceptibility to slow volume changes that can occur independently of loading due to swelling or shrinkage. Differences in the period and magnitude of precipitation and evapotransportation are the major factors influencing the swell–shrink response of active clay beneath a structure. These volume changes can give rise to ground movements that may result in damage to buildings. Low-rise buildings are particularly vulnerable to such ground movements since they generally do not have sufficient weight or strength to resist (Bell and Maud, 1995). These soils also represent a problem when they are encountered in road construction, and shrinkage settlement of embankments composed of such clays can lead to cracking and breaking up of the roads they support.

Grim (1952) distinguished two modes of swelling in clay soils, namely, intercrystalline and intracrystalline swelling. Intercrystalline swelling takes place when the uptake of moisture is restricted to the external crystal surfaces and the void spaces between the crystals. Intracrystalline swelling, on the other hand, is characteristic of the smectite family of clay
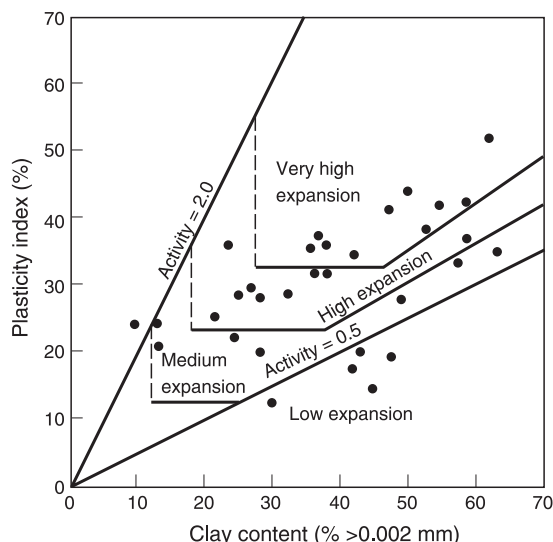
Figure 5.3

Use of the activity chart to estimate of the degree of expansiveness of some clay soils from Natal, South Africa (from Bell and Maud, 1995).

minerals, of montmorillonite in particular. The individual molecular layers that make up a crystal of montmorillonite are weakly bonded, so that on wetting moisture enters not only between the crystals but also between the unit layers that comprise the crystals. Generally, kaolinite has the smallest swelling capacity of the clay minerals, and nearly all of its swelling is of the intercrystalline type. Illite may swell by up to 15% but intermixed illite and montmorillonite may swell some 60–100%. Swelling in Ca montmorillonite is very much less than in the Na variety, it ranges from about 50–100%. Swelling in Na montmorillonite can amount to 2000% of the original volume, the clay then having formed a gel.

The maximum movement due to swelling beneath a building founded on expansive clay can be obtained from the following expression:

$$\text{Swell (\%)} = \frac{(PI - 10)}{10} \log_{10} S/p \qquad (5.6)$$

where PI is the plasticity index, S is the soil suction at the time of construction (in kPa) and p is the overburden plus foundation pressure acting on each layer of soil (in kPa). One of the most widely used soil properties to predict swell potential is the activity of clay (Fig. 5.3).

The United States Army Engineers Waterways Experimental Station (USAEWES) classification of potential swell (Table 5.13) is based on the liquid limit (LL), plasticity index (PI) and initial (in situ) suction (S_i). The latter is measured in the field by a psychrometer.

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Table 5.13. USAEWES classification of swell potential (After Snethan et al., 1977). With kind permission of the USAEWES

Liquid limit (%)	Plastic limit (%)	Initial (in situ) suction (kPa)	Potential swell (%)	Classification
Less than 50	Less than 25	Less than 145	Less than 0.5	Low
50–60	25–35	145–385	0.5–1.5	Marginal
Over 60	Over 35	Over 385	Over 1.5	High

The volume change that occurs due to evapotranspiration from a clay soil can be predicted conservatively by assuming the lower limit of the soil moisture content to be the shrinkage limit. Desiccation beyond this value cannot bring about further volume change. Transpiration from vegetative cover is a major cause of water loss from soils in semi-arid regions. Indeed, the distribution of soil suction in the soil is controlled primarily by transpiration from vegetation. The maximum soil suction that can be developed is governed by the ability of vegetation to extract moisture from the soil. The point, at which moisture is no longer available to plants is termed the permanent wilting point, and this corresponds to a pF value of about 4.2. The complete depth of active clay profiles usually does not become fully saturated during the wet season in semi-arid regions. Changes in soil suction may be expected over a depth of some 2.0 m between the wet and dry seasons. Swelling movements of over 350 mm have been reported for expansive clays in South Africa by Williams and Pidgeon (1983), and similar movements in similar soils have occurred in Texas.

The moisture characteristic (moisture content versus soil suction) of a soil provides valuable data concerning the moisture contents corresponding to the field capacity (defined in terms of soil suction, this is a pF value of about 2.0) and the permanent wilting point (pF of 4.2 and above), as well as the rate at which changes in soil suction take place with variations in moisture content. This enables an assessment to be made of the range of soil suction and moisture content that is likely to occur in the zone affected by seasonal changes in climate.

The extent to which the vegetation is able to increase the suction to the level associated with the shrinkage limit is important. In fact, the moisture content at the wilting point exceeds that of the shrinkage limit in soils with a high content of clay and is less in those possessing low clay contents. This explains why settlement resulting from the desiccating effects of trees is more notable in low to moderately expansive soils than in expansive ones. When vegetation is cleared from a site, its desiccating effect also is removed. Hence, the subsequent regain of moisture by clay soils leads to them swelling.

Desiccation cracks may extend to depths of 2 m in expansive clays and gape up to 150 mm. The suction pressure associated with the onset of cracking is approximately pF 4.6. The presence of

desiccation cracks enhances evaporation from the soil. Such cracks lead to a variable develop-ment of suction pressure, the highest suction occurring nearest the cracks. This, in turn, influ-ences the preconsolidation pressure as well as the shear strength. It has been claimed that the effect of desiccation on clay soils is similar to that of heavy overconsolidation.

Sridharan and Allam (1982), with reference to arid and semi-arid regions, found that repeated wetting and drying of clay soils can bring about aggregation of soil particles and cementation by compounds of Ca, Mg, Al and Fe. This enhances the permeability of the clays and increases their resistance to compression. Furthermore, interparticle desiccation bonding increases the shear strength, the aggregations offering higher resistance to stress. Indeed, depending on the degree of bonding, the expansiveness of an expansive clay soil may be reduced or it may even behave as a non-expansive soil.

Volume changes in clays also occur as a result of loading and unloading, which bring about consolidation and heave, respectively. When a load is applied to a clay soil, its volume is reduced, this principally being due to a reduction in the void ratio (Burland, 1990). If such a soil is saturated, then the load is carried initially by the pore water that causes a pressure, the hydrostatic excess pressure, to develop. The excess pressure of the pore water is dissipated at a rate that depends on the permeability of the soil mass, and the load is transferred even-tually to the soil structure. The change in volume during consolidation is equal to the volume of the pore water expelled and corresponds to the change in void ratio of the soil. In other words, primary consolidation is brought about by a reduction in the void ratio. In clay soils, because of their low permeability, the rate of consolidation is slow. Further consolidation may occur due to a rearrangement of the soil particles. This secondary consolidation is usually much less significant. The compressibility of a clay soil is related to its geological history, that is, to whether it is normally consolidated or overconsolidated. A normally consolidated clay is one that at no time in its geological history has been subject to vertical pressure greater than its existing overburden pressure, whereas an overconsolidated clay has.

The compressibility of a clay soil can be expressed in terms of the compression index, C_c, or the coefficient of volume compressibility, m_v. The compression index tends to be applied to normally consolidated clays. The value of C_c for fine soils ranges from 0.075 for sandy clays to more than 1.0 for highly colloidal bentonic clays. An approximation of the degree of com-pressibility is given in Table 5.14a. It can be seen that the compressibility index increases with increasing clay content and so with increasing liquid limit. The coefficient of volume compress-ibility is defined as the volume change per unit volume per unit increase in load. The value of m_v for a given soil depends on the stress range over which it is determined. Anon, (1990a) recommended that it should be calculated for a pressure increment of 100 kPa in excess of the effective overburden pressure on the soil at the depth from which the sample was taken. Some typical values of m_v are given in Table 5.14b.

Table 5.14. Range of compressibility of fine soils

(a) Compressibility index

Soil type	Range (C_c)	Degree of compressibility
Soft clay	Over 0.3	Very high
Clay	0.3–0.15	High
Silt	0.15–0.075	Medium
Sandy clay	Less than 0.075	Low

(b) Some typical values of coefficient of volume compressibility

Coefficient of volume compressibility (m^2 MN^{-1})	Degree of compressibility	Soil types
Above 1.5	Very high	Organic alluvial clays and peats
0.3–1.5	High	Normally consolidated alluvial clays
0.1–0.3	Medium	Varved and laminated clays Firm to stiff clays
0.05–0.1	Low	Very stiff or hard clays Tills
Below 0.05	Very low	Heavily overconsolidated tills

When an excavation is made in clay with weak diagenetic bonds, elastic rebound causes immediate dissipation of some stored strain energy in the soil. However, part of the strain energy is retained due to the restriction on lateral straining in the plane parallel to the ground surface. The lateral effective stress either remains constant or decreases as a result of plastic deformation of the clay as time passes. This plastic deformation can result in significant time-dependent vertical heaving. However, creep of weakly bonded soils is not a common cause of heaving in excavations.

Overconsolidated clay is considerably stronger at a given pressure than normally consolidated clay, and it tends to dilate during shear, whereas normally consolidated clay consolidates (Burland, 1990). Hence, when overconsolidated clay is sheared under undrained conditions, negative pore water pressures are induced, the effective strength is increased, and the undrained strength is much higher than the drained strength, (the exact opposite to normally consolidated clay). When the negative pore water pressure gradually dissipates, the strength falls as much as 60 or 80% to the drained strength.

In 1964, Skempton observed that when clay is strained, it develops an increasing resistance (strength), but that under a given effective pressure, the resistance offered is limited, the maximum value corresponding to the peak strength. If testing is continued beyond the peak

strength, then as displacement increases, the resistance decreases, again to a limiting value that is termed the residual strength. Skempton noted that in moving from peak to residual strength, cohesion falls to almost, or actually, zero and the angle of shearing resistance is reduced to a few degrees (it may be as much as 10° in some clays). He explained the drop in strength that occurred in overconsolidated clay as due to its expansion on passing peak strength and associated increasing water content on the one hand. On the other, he maintained that platey clay minerals became orientated in the direction of shear and thereby offered less resistance. Failure occurs once the stress on clay exceeds its peak strength and, as failure progresses, the strength of the clay along the shear surface is reduced to the residual value.

It was suggested that under a given effective pressure, the residual strength of clay is the same whether it is normal, or overconsolidated (Fig. 5.4). In other words, the residual shear strength of clay is independent of its post-depositional history, unlike the peak undrained shear strength that is controlled by the history of consolidation as well as diagenesis. Furthermore, the value of residual shear strength, ϕ_r', decreases as the amount of clay fraction increases in a deposit. In this context, not only is the proportion of detrital minerals important but so is that of the diagenetic minerals. The latter influence the degree of induration of a deposit of clay, and the value of ϕ_r' can fall significantly as the ratio of clay minerals to detrital and diagenetic minerals increases.

The shear strength of undisturbed clay is frequently found to be greater than that obtained when it is remoulded and tested under the same conditions and at the same water content levels. The ratio of the undisturbed to the remoulded strength at the same moisture content is termed the sensitivity of clay. Skempton and Northey (1952) proposed six grades of
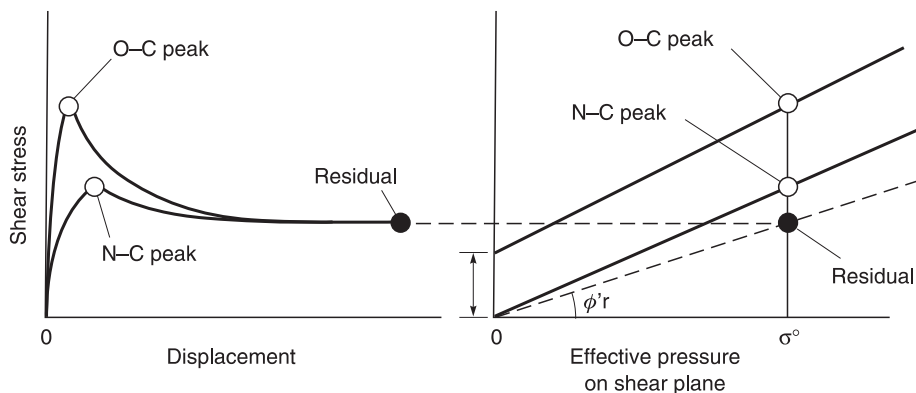

Figure 5.4

Peak strength and residual strength of normally consolidated (N–C) and overconsolidated (O–C) clay soils (after Skempton, 1964). With kind permission of the Institution of Civil Engineers.

sensitivity, namely, insensitive clays (under 1), low sensitive clays (1–2), medium sensitive clays (2–4), sensitive clays (4–8), extra-sensitive clays (8–16) and quick clay (over 16).

Clays with high sensitivity values have little or no strength after being disturbed. Indeed, if they suffer disturbance, this may cause an initially fairly strong material to behave as a viscous fluid. Any subsequent gain in strength due to thixotropic hardening does not exceed a small fraction of its original value. Sensitive clays generally possess high moisture contents, frequently with liquidity indices well in excess of unity. A sharp increase in moisture content may cause a great increase in sensitivity, sometimes with disastrous results. Heavily overconsolidated clays are insensitive.

Fissures play an extremely important role in the failure mechanism of overconsolidated clays. For example, the strength along fissures in clay is only slightly higher than the residual strength of the intact clay. Hence, it can be concluded that the upper limit of the strength of fissured clay is represented by its intact strength, whereas the lower limit corresponds to the strength along the fissures. The operational strength, which is somewhere between the two, however, is often significantly higher than the fissure strength. In addition to allowing clay to soften, fissures allow concentrations of shear stress that locally exceed the peak strength of clay, thereby giving rise to progressive failure. Under stress, the fissures in clay seem to propagate and coalesce in a complex manner. The ingress of water into fissures means that the pore water pressure in the clay concerned increases, which, in turn, means that its strength is reduced. Fissures in normally consolidated clays have no significant practical consequences.

The greatest variation in the engineering properties of clays can be attributed to the degree of weathering that they have undergone (Fig. 5.5). For instance, consolidation of a clay deposit gives rise to an anisotropic texture due to the rotation of the platey minerals. Secondly, diagenesis bonds particles together either by the development of cement, the intergrowth of adjacent grains or the action of van der Waals charges that are operative at very small grain separations. Weathering reverses these processes, altering the anisotropic structure and destroying or weakening interparticle bonding (Coulthard and Bell, 1993). Ultimately, weathering, through the destruction of interparticle bonds, leads to a clay deposit reverting to a normally consolidated, sensibly remoulded condition. Higher moisture contents are found in more weathered clay. This progressive degrading and softening also is accompanied by reductions in strength and deformation modulus with a general increase in plasticity. The reduction in strength has been illustrated by Cripps and Taylor (1981), who quoted the strength parameters for brown (weathered) and blue (unweathered) London Clay, as given in Table 5.15. These values indicate that the undrained shear strength, τ_u, is reduced by approximately half and that the effective cohesion, c', can suffer significant reduction on weathering. The effective angle of shearing resistance also is reduced and, at $\phi = 20°$, the value corresponds to a fully softened condition.
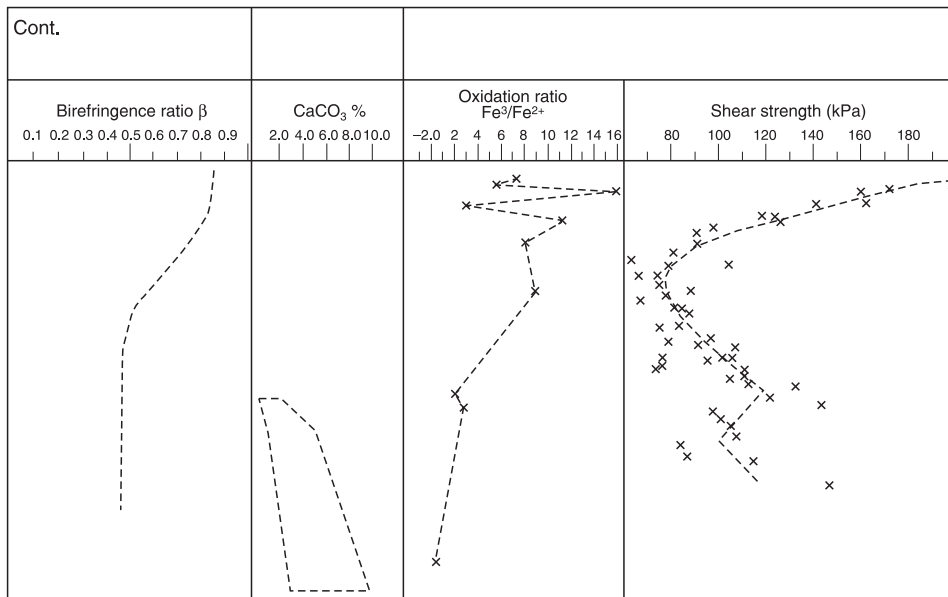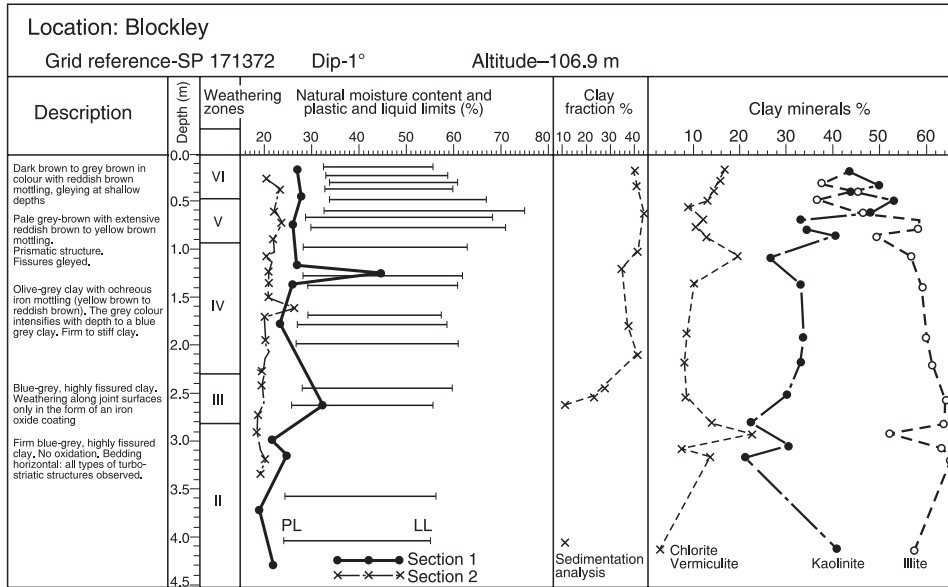
Figure 5.5

Geotechnical profile of the Lower Lias Clay at Blockley, Gloucestershire, England. A measure of the average orientation of clay particles seen in a thin section beneath the petrological microscope is afforded by the birefringence ratio (i.e. the ratio between the minimum and maximum light transmitted under crossed polars). This ratio varies between 0 for perfect parallel orientation to 1 for perfect random orientation. With increasing weathering, the birefringence ratio increases, indicating that the fabric of the clay becomes more disordered as the ground surface is approached. Weathering of pyrite (Fe_2S) produces sulphate and sulphuric acid. The latter reacts with calcium carbonate. The oxidation of iron compounds tends to increase as the surface is approached, which leads to increasing shear strength. *PL* = plastic limit; *LL* = liquid limit (after Coulthard and Bell, 1993).

Table 5.15. Strength of weathered (brown) and unweathered (blue) London Clay (after Cripps and Taylor, 1981)

Parameter	Brown	Blue
τ_u (kPa)	100–175	120–250
c' (kPa)	0–31	35–252
φ°	20–23	25–29

Tropical Soils

In humid tropical regions, weathering of rock is more intense and extends to greater depths than in other parts of the world. Residual soils develop in place as a consequence of weathering, primarily chemical weathering (Rahardjo et al., 2004). Consequently, climate (temperature and rainfall), parent rock, water movement (drainage and topography), age and vegetation cover are responsible for the development of the soil profile.

Ferruginous and aluminous clay soils are frequent products of weathering in tropical latitudes. They are characterized by the presence of iron and aluminium oxides and hydroxides. These compounds, especially those of iron, are responsible for the red, brown and yellow colours of the soils. The soils may be fine grained, or they may contain nodules or concretions. Concretions occur in the matrix where there are higher concentrations of oxides in the soil. More extensive accumulations of oxides give rise to laterite.

Laterite is a residual ferruginous clay-like deposit that generally occurs below a hardened ferruginous crust or hardpan (Charman, 1988). The ratios of silica (SiO_2) to sesquioxides (Fe_2O_3, Al_2O_3) in laterites usually are less than 1.33, those ratios between 1.33 and 2.0 are indicative of lateritic soils, and those greater than 2.0 are indicative of non-lateritic types. During drier periods, the water table is lowered. The small amount of iron that has been mobilized in the ferrous state by the groundwater is then oxidized, forming hematite, or goethite if hydrated. The movement of the water table leads to the gradual accumulation of iron oxides at a given horizon in the soil profile. A cemented layer of laterite is formed that may be a continuous or honeycombed mass, or nodules may be formed, as in laterite gravel. Concretionary layers often are developed near the surface in lowland areas because of the high water table.

Laterite hardens on exposure to air. Hardening may be due to a change in the hydration of iron and aluminium oxides.

Laterite commonly contains all size fractions from clay to gravel and sometimes even larger material (Fig. 5.6). Usually, at or near the surface, the liquid limit of laterite does not exceed 60% and the plasticity index is less than 30%. Consequently, laterite is of low to medium
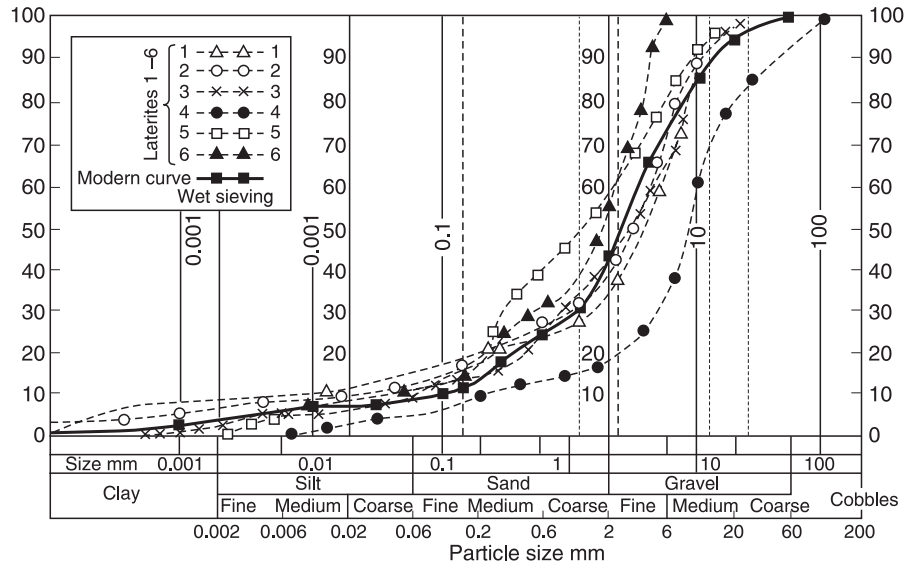
Figure 5.6

Grading curves of laterite (after Madu, 1977). With kind permission of Elsevier.

plasticity. The activity of laterite may vary between 0.5 and 1.75. Some values of common properties of laterite are given in Table 5.16.

Lateritic soils, particularly where they are mature, furnish a good bearing stratum (Blight, 1990). The hardened crust has a low compressibility and, therefore, settlement is likely to be negligible. In such instances, however, the strength of the soil may decrease with increasing depth.

Red earths or latosols are residual ferruginous soils in which oxidation readily occurs. Most such soils appear to have been derived from the first cycle of weathering of the parent material.

Table 5.16. Some common properties of laterites (after Madu, 1977)

Moisture content (%)	10–49
Liquid limit (%)	33–90
Plastic limit (%)	13–31
Clay fraction	15–45
Dry unit weight (kN m^{-3})	15.2–17.3
Cohesion (kPa)	466–782
Angle of friction (°)	28–35
Unconfined compressive strength (kPa)	220–825
Compression index	0.0186
Coefficient of consolidation (m^2 a^{-1})	262
Young's modulus (kPa)	5.63×10^4

They differ from laterite in that they behave as a clay soil and do not possess strong concretions. They, however, do grade into laterite.

Black clays typically are developed on poorly drained plains in regions with well-defined wet and dry seasons, where the annual rainfall is not less than 1250 mm. Generally, the clay fraction in these soils exceeds 50%, silty material varying between 20 and 40% and sand forming the remainder. The organic content usually is less than 2%. The liquid limits of black clays may range between 50 and 100%, with plasticity indices of between 25 and 70%. The shrinkage limit frequently is around 10–12%. Montmorillonite commonly is present in the clay fraction and is the chief factor determining the behaviour of these clays. For instance, they undergo appreciable volume changes on wetting and drying due to the montmorillonite content. These volume changes, however, tend to be confined to an upper critical zone of the soil, which frequently is less than 1.5 m thick. Below this, the moisture content remains more or less the same, for instance, around 25%.

Dispersive Soils

Dispersion occurs in soils when the repulsive forces between clay particles exceed the attractive forces, thus bringing about deflocculation so that, in the presence of relatively pure water, the particles repel each other to form colloidal suspensions. In non-dispersive soil, there is a definite threshold velocity below which flowing water causes no erosion. The individual particles cling to each other and are removed by water flowing with a certain erosive energy. By contrast, there is no threshold velocity for dispersive soil, the colloidal clay particles go into suspension even in quiet water and, therefore, these soils are highly susceptible to erosion and piping. Dispersive soils contain a moderate to high content of clay material but there are no significant differences in the clay fractions of dispersive and non-dispersive soils, except that soils with less than 10% clay particles may not have enough colloids to support dispersive piping. Dispersive soils contain a higher content of dissolved sodium (up to 12%) in their pore water than do ordinary soils. The clay particles in soils with high salt contents exist as aggregates and coatings around silt and sand particles, and the soil is flocculated. Dispersive soils generally occur in semi-arid regions where the annual rainfall is less than 860 mm (Bell and Walker, 2000).

For a given eroding fluid, the boundary between the flocculated and deflocculated states depends on the value of the sodium adsorption ratio. The sodium adsorption ratio, SAR, is used to quantify the role of sodium where free salts are present in the pore water and is defined as:

$$SAR = \frac{Na}{\sqrt{0.5\,(Ca + Mg)}} \tag{5.7}$$

with units expressed in milli-equivalents per litre of the saturated extract. There is a relationship between the electrolyte concentration of the pore water and the exchangeable ions in the adsorbed layers of clay particles. This relationship is dependent on pH value and also may be influenced by the type of clay minerals present. Hence, it is not necessarily constant. Gerber and Harmse (1987) considered a SAR value greater than 10 as indicative of dispersive soils, between 6 and 10 as intermediate and less than 6 as nondispersive. However, Aitchison and Wood (1965) regarded soils in which the SAR exceeded 2 as dispersive.

Dispersive erosion depends on the mineralogy and chemistry of soil on the one hand, and the dissolved salts in the pore and eroding water on the other. The presence of exchangeable sodium is the main chemical factor contributing towards dispersive clay behaviour. This is expressed in terms of the exchangeable sodium percentage, ESP:

$$ESP = \frac{\text{exchangeable sodium}}{\text{cation exchange capacity}} \times 100 \qquad (5.8)$$

where the units are given in meq/100 g of dry soil. Above a threshold value of ESP of 10, soils have their free salts leached by seepage of relatively pure water and are prone to dispersion. Soils with ESP values above 15% are highly dispersive, according to Gerber and Harmse (1987). On the other hand, those soils with low cation exchange values (15 meq/100 g of clay) are non-dispersive at ESP values of 6% or below. Soils with high cation exchange capacity values and a plasticity index greater than 35% swell to such an extent that dispersion is not significant. High ESP values and piping potential may exist in soils in which the clay fraction is composed largely of smectitic and other 2:1 clays. Some illitic soils are highly dispersive. High values of ESP and high dispersibility are generally not common in clays composed largely of kaolinite.

Another property that has been claimed to govern the susceptibility of clayey soils to dispersion is the total content of dissolved salts, TDS, in the pore water. In other words, the lower the content of dissolved salts in the pore water, the greater the susceptibility of sodium-saturated clays to dispersion. Sherard et al. (1976) regarded the total dissolved salts for this specific purpose as the total content of calcium, magnesium, sodium and potassium in milli-equivalents per litre. They designed a chart in which sodium content was expressed as a percentage of TDS and was plotted against TDS to determine the dispersivity of soils (Fig. 5.7a). However, Craft and Acciardi (1984) showed that this chart had poor overall agreement with the results of physical tests. Furthermore, Bell and Maud (1994) showed that the use of the dispersivity chart to distinguish dispersive soils had not proved reliable in Natal, South Africa. There, the determination of dispersive potential frequently involves the use of a chart designed by Gerber and Harmse (1987) that plots ESP against cation exchange capacity, CEC (Fig. 5.7b).
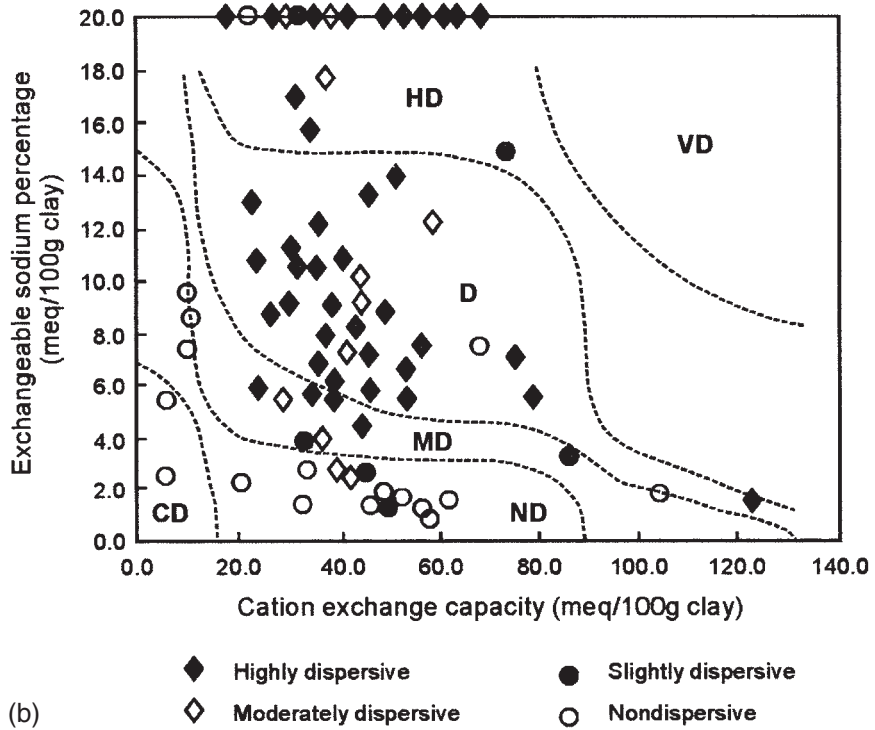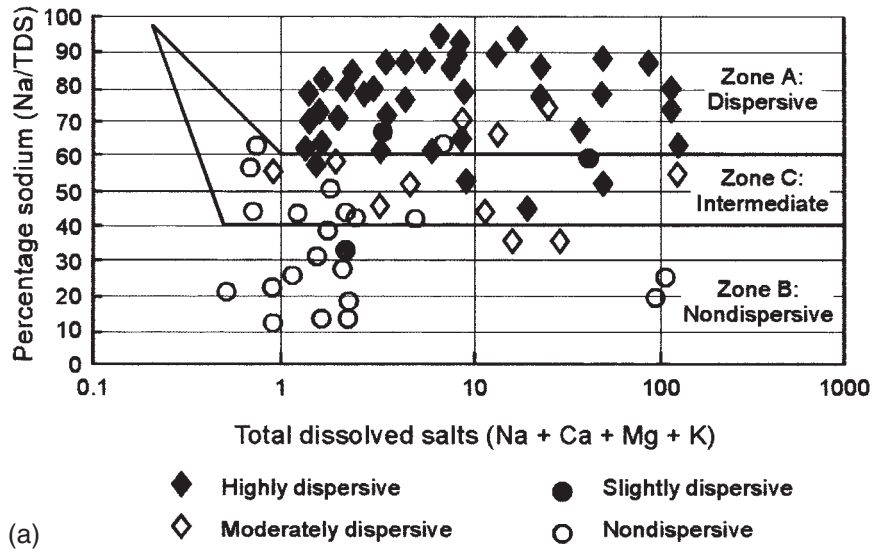
(a)

(b)

Figure 5.7

(a) Potential dispersivity chart of Sherard et al., 1976, with some examples of soils from Natal, South Africa (after Bell and Walker, 2000). (b) Chart for classification of soils to determine their dispersivity, developed by Gerber and von Harmse (1987), with some examples of soils from Natal, South Africa (after Bell and Walker, 2000). The dispersivities plotted in (a) and (b) were determined by a rating system developed by a Bell and Walker.

Damage due to internal erosion of dispersive soil leads to the formation of pipes and internal cavities within slopes. Piping is initiated by dispersion of clay particles along desiccation cracks, fissures and root-holes. Piping has led to the failure of earth dams built with dispersive soil (Fig. 5.8). Indications of piping take the form of small leakages of muddy-coloured water after initial filling of the reservoir. In addition, severe erosion damage forming deep gullies occurs on embankments after rainfall. Fortunately, when dispersive soils are treated with lime, they are transformed to a non-dispersive state if the lime is mixed thoroughly into the soil.

Soils of Arid Regions

Most arid deposits consist of the products of physical weathering of bedrock formations. Weathering activity tends to be dominated by the physical breakdown of rock masses into poorly sorted assemblages of fragments ranging in size down to silts. Many of the deposits within alluvial plains and covering hillsides are poorly consolidated. As such, they may undergo large settlements, especially if subjected to vibration due to earthquakes or cyclic loading. Some gravels may consist of relatively weak, low-durability materials. Many arid areas are dominated by the presence of large masses of sand. Depending on the rate of

Figure 5.8

Failure of a small dam constructed of dispersive soil, near Ramsgate, Natal, South Africa.

supply of sand, the wind speed, direction, frequency and constancy and the nature of the ground surface, sand may be transported and/or deposited in mobile or static dunes. For the most part, aeolian sands are poorly (uniformly) graded. In the absence of downward leaching, surface deposits become contaminated with precipitated salts, particularly sulphates and chlorides. Alluvial plain deposits often contain gypsum particles and cement, and also fragments of weak weathered rock and clay.

In arid regions, sabkha conditions commonly develop in low-lying coastal zones and inland plains with shallow water tables. These are extensive saline flats that are underlain by sand, silt or clay and often are encrusted with salt. Highly developed sabkhas tend to retain a greater proportion of soil moisture than moderately developed sabkhas. Within coastal sabkhas, the dominant minerals are calcite ($CaCO_3$), dolomite [$(Ca,Mg)CO_3$] and gypsum ($CaSO_4.nH_2O$), with lesser amounts of anhydrite ($CaSO_4$), magnesite ($MgCO_3$), halite ($NaCl$) and carnalite ($KCl.MgCl_2.6H_2O$), together with various other sulphates and chlorides (James and Little, 1994). Highly saline groundwater may contain up to 23% sodium chloride and occur close to ground level. In fact, the sodium chloride content of groundwater can be high enough to represent a corrosion hazard.

Minerals that are precipitated from groundwater in arid deposits also have high solution rates, so that flowing groundwater may lead to the development of solution features. Problems such as increased permeability, reduced density and settlement are liable to be associated with engineering works or natural processes that result in a decrease in the salt concentration of groundwater. Changes in the state of hydration of minerals, such as swelling clays and calcium sulphate, also cause significant volume changes in soils. In particular, low-density sands that are cemented with soluble salts such as sodium chloride are vulnerable to salt removal by dissolution by freshwater, leading to settlement. Hence, rainstorms and burst water mains present a hazard, as does watering of grassed areas and flower beds. The latter should be controlled, and major structures should be protected by drainage measures to reduce the risks associated with rainstorms or burst water pipes. In the case of inland sabkhas, the minerals precipitated within the soil are much more variable than those of coastal sabkhas since they depend on the composition of local groundwater.

Sabkha soils frequently are characterized by low strength. Furthermore, some surface clays that are normally consolidated or lightly overconsolidated may be sensitive to highly sensitive. The low strength is attributable to the concentrated salt solutions in sabkha brines; the severe climatic conditions under which sabkha deposits are formed (e.g. large variations in temperature and excessive wetting–drying cycles) that can give rise to instability in sabkha soils; and the ready solubility of some of the minerals that act as cements in these soils. As a consequence, the bearing capacity of sabkha soils and their compressibility frequently do not meet routine design requirements.

A number of silty deposits formed under arid conditions are liable to undergo considerable volume reduction or collapse when wetted. Such metastability arises due to the loss of strength of interparticle bonds, resulting from increases in water content. Thus, infiltration of surface water, including that applied by irrigation, leakage from pipes and rise of the water table may cause large settlements to occur.

A common feature of arid regions is the cementation of sediments by the precipitation of mineral matter from the groundwater. The species of salt held in solution, and also those precipitated, depends on the source of the water, as well as the prevailing temperature and humidity conditions. The process may lead to the development of various crusts or cretes in which unconsolidated deposits are cemented. The most commonly precipitated material is calcium carbonate (Netterburg, 1994). As the carbonate content increases in these soils, it first occurs as scattered concentrations of flaky habit, then as hard concretions. Once it exceeds 60%, the concentration becomes continuous. These deposits are referred to as calcrete (Fig. 5.9). The calcium carbonate in calcrete profiles decreases from top to base, as generally does the hardness. The development of calcrete is inhibited beyond a certain aridity since the low precipitation is unable to dissolve and drain calcium carbonate towards the water table. Consequently, in very arid climates, gypcrete may take the place of calcrete.

Figure 5.9

Calcrete in the north of Namib-Nauluft Park, Namibia.

Tills and Other Glacially Associated Deposits

Till usually is regarded as being synonymous with boulder clay. It is deposited directly by ice, whereas stratified drift is deposited in melt waters associated with glaciers. The character of till deposits varies appreciably and depends on the lithology of the material from which it was derived, on the position in which it was transported in the glacier, and on the mode of deposition. The underlying bedrock material usually constitutes up to about 80% of basal or lodgement tills, depending on its resistance to abrasion and plucking.

Deposits of till consist of a variable assortment of rock debris ranging from fine rock flour to boulders (Hughes et al., 1998). The shape of the rock fragments found in till varies but is conditioned largely by their initial shape at the moment of incorporation into the ice. Angular boulders are common, their irregular sharp edges resulting from crushing. Tills may consist essentially of sand and gravel with very little binder, alternatively they may have an excess of clay. Lenses and pockets of sand, gravel and highly plastic clay frequently are encountered in some tills. Argillaceous rocks, such as shales and mudstones, are abraded more easily and so produce fine-grained tills that are richer in clay minerals and, therefore, more plastic than other tills. Mineral composition also influences the natural moisture content, which is slightly higher in tills containing appreciable quantities of clay minerals.

Lodgement till is plastered on to the ground beneath a moving glacier in small increments as the basal ice melts. Because of the overlying weight of ice, such deposits are overconsolidated. Due to abrasion and grinding, the proportion of silt and clay size material is relatively high in lodgement till (e.g. the clay fraction varies from 15 to 40%). Lodgement till is commonly stiff, dense and relatively incompressible (Sladen and Wrigley, 1983). Hence, it is practically impermeable. Fissures frequently are present in lodgement till, especially if it is clay matrix dominated.

Ablation till accumulates on the surface of the ice when englacial debris melts out, and as the glacier decays, the ablation till is lowered slowly to the ground. Therefore, it is normally consolidated. Ablation tills have a high proportion of far-travelled material and may not contain any of the local bedrock. Because it has not been subjected to much abrasion, ablation till is characterized by abundant large fragments that are angular and not striated, the proportion of sand and gravel is high and clay is present only in small amounts (usually less than 10%). Because the texture is loose, ablation till can have an extremely low in situ density. Since ablation till consists of the load carried at the time of ablation, it usually forms a thinner deposit than lodgement till.

The particle size distribution and fabric (stone orientation, layering, fissuring and jointing) are among the most significant features as far as the engineering behaviour of a till is concerned. McGown and Derbyshire (1977) therefore used the percentage of fines to distinguish

granular, well-graded and matrix-dominated tills, the boundaries being placed at 15 and 45%, respectively.

Tills frequently are gap graded, the gap generally occurring in the sand fraction (Fig. 5.10). Large, often very local, variations can occur in the grading of till that reflect local variations in the formation processes, particularly the comminution processes. The range in the proportions of coarse and fine fractions in tills dictates the degree to which the properties of the fine fraction influence the properties of the composite soil. The variation in the engineering properties of the fine soil fraction is greater than that of the coarse fraction, and this often tends to dominate the engineering behaviour of the till.

The specific gravity of till deposits often is remarkably uniform, varying from 2.77 to 2.78. These values suggest the presence of fresh minerals in the fine fraction, that is, rock flour rather than clay minerals. Rock flour behaves more like granular material than cohesive and has a low plasticity. The consistency limits of tills are dependent on moisture content, grain size distribution and the properties of the fine-grained fraction (Bell, 2002). Generally, however, the plasticity index is small and the liquid limit of tills decreases with increasing grain size. The variations in some simple index properties with depth of the Upper Boulder Clay of Teesside, England, are given in Figure 5.11.
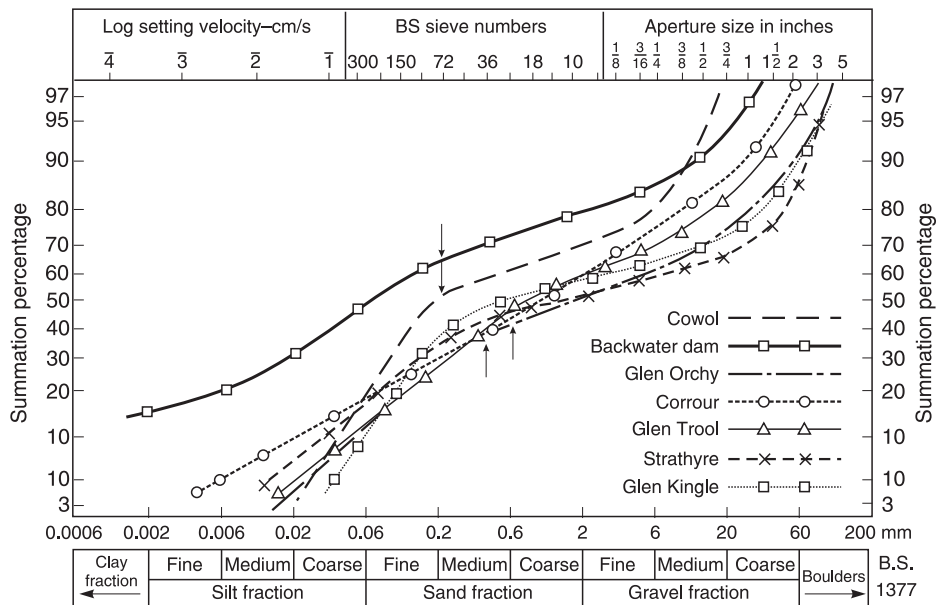

Figure 5.10

Typical gradings of some Scottish tills (after McGown, 1971). With kind permission of the Geological Society.
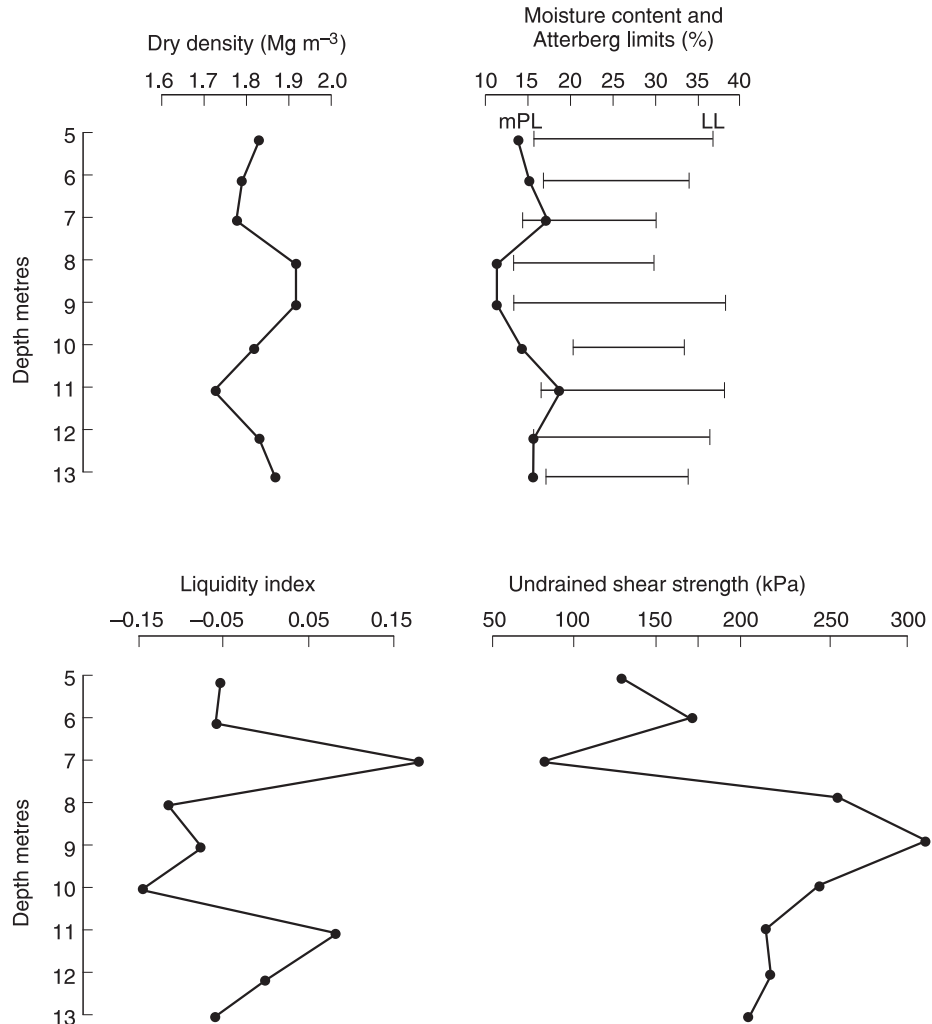
Figure 5.11

Variation in some simple index properties with depth of the Upper Boulder Clay of Teesside, England (after Bell, 2002).

The compressibility and consolidation of tills are determined principally by the clay content, as is the shear strength (Table 5.17). For example, the value of compressibility index tends to increase linearly with increasing clay content, whereas for tills of very low clay content (less than 2%), this index remains about constant ($C_c = 0.01$). The shear strength of till can range from 150 kPa to over 1.5 MPa.

Fissures in till tend to be variable in character, spacing, orientation and areal extent, although they can have a preferred orientation. Opening up and softening along these fissures gives

Table 5.17. Strength of tills from Holderness (after Bell, 2002)

	Unconfined compressive strength (kPa)		Sensitivity	Direct shear				Triaxial			
	Intact	Remoulded		c	$\phi°$	c_r	$\phi_r°$	c_u	$\phi_u°$	c'	$\phi°$
1. Hessle Till (Dimlington, Hornsea)											
Max	138	116	1.31 (L)	30	25	3	23	98	8	80	24
Min	96	74	1.10 (L)	16	16	0	13	22	5	10	13
Mean	106	96	1.19 (L)	20	24	1	20	35	7	26	25
2. Withernsea Till (Dimlington)											
Max	172	148	1.18 (L)	38	30	2	27	62	19	42	34
Min	140	122	1.15 (L)	21	20	0	18	17	5	17	16
Mean	160	136	1.16 (L)	26	24	1	21	30	9	23	25
3. Skipsea Till (Dimlington)											
Max	194	168	1.15 (L)	45	38	5	35	50	21	25	36
Min	182	154	1.08 (L)	25	20	0	19	17	10	22	24
Mean	186	164	1.13 (L)	27	26	1	25	29	12	28	30
4. Basement Till (Dimlington)											
Max	212	168	1.27 (L)	47	34	2	30	59	17	42	36
Min	163	140	1.19 (L)	23	20	0	18	22	6	19	20
Mean	186	156	1.21 (L)	29	24	1	23	38	9	34	29

Note: c = cohesion in kPa, r = residual, u = undrained; ϕ = angle of friction; L = low sensitivity.

rise to a rapid reduction of undrained shear strength along the fissures. In fact, the undrained shear strength along fissures in till may be as little as one-sixth that of the intact soil. The nature of the various fissure coatings (sand, silt or clay-size material) is of critical importance in determining the shear strength behaviour of fissured tills. Deformation and permeability also are controlled by the nature of the fissure surfaces and coatings.

Eyles and Sladen (1981) recognized four zones of weathering within the soil profile of lodgement till in the coastal area of Northumberland, England (Table 5.18a). As the degree of weathering of the till increases, so does the clay fraction and moisture content. This, in turn, leads to changes in the liquid and plastic limits and in the shear strength (Table 5.18b and Fig. 5.12).

Deposits of stratified drift often are subdivided into two categories, namely, those that develop in contact with ice, namely, ice contact deposits, and those that accumulate beyond the limits of ice, forming in streams, lakes or seas, that is, proglacial deposits.

Table 5.18a. A weathering scheme for Northumberland lodgement tills (after Eyles and Sladen, 1981). With kind permission of Elsevier

Weathering zone	Zone	Description	Maximum depth (m)
Highly weathered	IV	Oxidized till and surficial material Strong oxidation colours High rotten boulder content Leaching of most primary carbonate Prismatic gleyed jointing Pedological profile usually leached brown earth	3
Moderately weathered	III	Oxidized till Increased clay content Low rotten boulder content Little leaching of primary carbonate Usually dark brown or dark red brown Base commonly defined by fluvioglacial sediments	8
Slightly weathered	II	Selective oxidation along fissure surfaces where present, otherwise as Zone I	10
Unweathered	I	Unweathered till No post-depositionally rotted boulders No oxidation No leaching of primary carbonate Usually dark grey	

Table 5.18b. Typical geotechnical properties for Northumberland lodgement tills (after Eyles and Sladen, 1981). With kind permission of Elsevier

| Property	Weathered zones I	III & IV
Bulk density (Mg m^{-3})	2.15–2.30	1.90–2.20
Natural moisture content (%)	10–15	12–25
Liquid limit (%)	25–40	35–60
Plastic limit (%)	12–20	15–25
Plasticity index	0–20	15–40
Liquidity index	−0.20 to −0.05	III −0.15 to +0.05 IV −0 to +30
Grading of fine (<2 mm) fraction		
% clay	20–35	30–50
% silt	30–40	30–50
% sand	30–50	10–25
Average activity	0.64	0.68
c' (kPa)	0–15	0–25
ϕ' (degrees)	32–37	27–35
ϕ_r' (degrees)	30–32	15–32
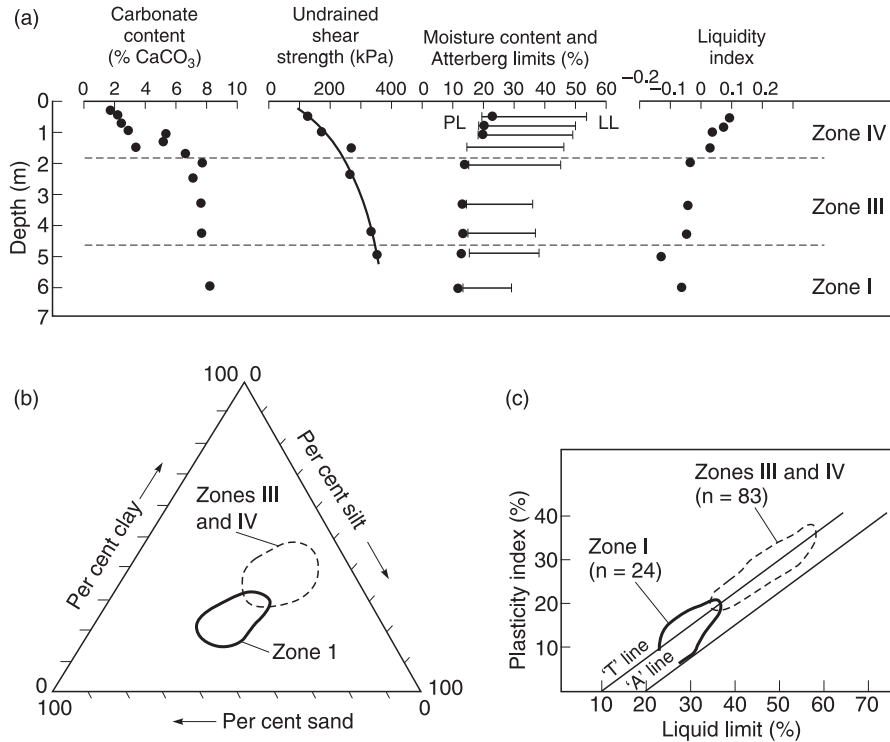
Figure 5.12

(a) Lodgement till from Northumberland, England, showing the variation with depth of carbonate content, undrained shear strength, moisture content, Atterberg limits and liquidity index. (b) Particle size distribution for weathered and unweathered tills shown in (a). (c) Plasticity chart for weathered and unweathered tills shown in (a). n = number of determinations (after Eyles and Sladen, 1981). With kind permission of Elsevier.

The range of particle size found in outwash fans varies from coarse sands to boulders. When they are first deposited, their porosity may be anything from 25 to 50%, and they tend to be very permeable. The finer silt–clay fraction tends to be transported further downstream. Other ice contact deposits, namely, kames, kame terraces and eskers, usually consist of sands and gravels.

The most familiar proglacial deposits are varved clays. The thickness of the individual varve is frequently less than 2 mm, although much thicker layers have been noted in some deposits. Generally, the coarser layer is of silt size and the finer of clay size. Varved clays tend to be normally consolidated or lightly overconsolidated, although it usually is difficult to make the distinction. In many cases, the precompression may have been due to ice loading. The range of liquid limit for varved clays tends to vary between 30 and 80%, whereas that of plastic limit often varies between 15 and 30% (Table 5.19). These limits allow the material to

Table 5.19. Some properties of varved and laminated clays

	Varved clays, Elk Valley, British Columbia*	Laminated clays, Teesside, England**
Moisture content (%)	35	25–35 (30)
Plastic limit (%)	22	18–31 (26)
Liquid limit (%)	34	29–78 (56)
Plasticity index (%)	15.5	19–49 (33)
Liquidity index	0.36	⁻0.12–0.35 (0.15)
Activity	0.36	0.47–0.65 (0.54)
Linear shrinkage		9–14 (11)
Compression index	0.405–0.587 (0.496)	0.55
Undrained shear strength (kPa)		20–102 (62)

Note: Range with average value in brackets.
*After George, 1986.
**After Bell and Coulthard, 1997.

be classified as inorganic silty clay of medium to high plasticity or compressibility. In some varved clay, the natural moisture content is near the liquid limit. Consequently, these clays are soft and frequently have sensitivities around 4 (Bell and Coulthard, 1997). Their activity tends to range between active and normal, and some may be expansive. The average strength of some varved clays, for example, from Ontario, is about 40 kPa, with a range of 24–49 kPa. The effective stress parameters of apparent cohesion and angle of shearing resistance range from 0.7 to 19.5 kPa, and 22–25°, respectively.

The material of which quick clays are composed is predominantly smaller than 0.002 mm (Geertsema and Torrance, 2005). Many deposits, however, seem to be very poor in clay minerals, containing a high proportion of ground-down, fine quartz. The fabric of these soils contains aggregations. Granular particles, whether aggregations or primary minerals, are rarely in direct contact, being linked generally by bridges of fine particles. Clay minerals usually are non-oriented, and clay coatings on primary minerals tend to be uncommon, as are cemented junctions. Networks of platelets occur in some soils. Quick clays generally exhibit little plasticity, their plasticity index usually varying from 8 to 12%. Their liquidity index normally exceeds 1, and their liquid limits are often less than 40%. Quick clays usually are inactive, their activity frequently being less than 0.5. The most extraordinary property possessed by quick clays is their very high sensitivity. In other words, a large proportion of their undisturbed strength is permanently lost following shear (Fig. 5.13). The small fraction of the original strength regained after remoulding may be attributable to the development of some different form of interparticle bonding. The reason why only a small fraction of the original strength is recovered is because the rate at which it develops is so slow.
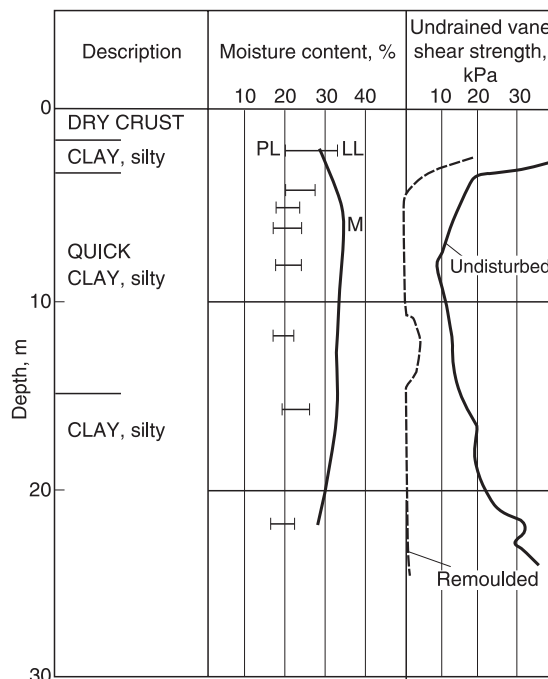
Figure 5.13

Moisture content, consistency indices, undrained shear strength and sensitivity of quick clay from near Trondheim, Norway (from Bell and De Bruyn, 1998).

Frost Action in Soil

Frost action in a soil is influenced by the initial temperature of the soil, as well as the air temperature; the intensity and duration of the freeze period; the depth of frost penetration; the depth of the water table; and the type of ground cover. If frost penetrates down to the capillary fringe in fine soils, especially silts, then, under certain conditions, lenses of ice may be developed. The formation of such ice lenses may, in turn, cause frost heave and frost boil that may lead to the break-up of roads, the failure of slopes, etc. Shrinkage, which is attributable to thermal contraction and desiccation, gives rise to polygonal cracking in the ground. Water that accumulates in the cracks is frozen and consequently helps increase their size. This action may lead to the development of lenses of ice.

Classification of Frozen Soil

Ice may occur in frozen soil as small disseminated crystals whose total mass exceeds that of the mineral grains. It also may occur as large tabular masses that range up to several metres

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thick, or as ice wedges. The latter may be several metres wide and may extend to 10 m or so in depth. As a consequence, frozen soils need to be described and classified for engineering purposes. A method of classifying frozen soils involves the identification of the soil type and the character of the ice (Andersland, 1987). First, the character of the actual soil is classified according to the Unified Soil Classification system (Table 5.2). Second, the soil characteristics consequent on freezing are added to the description. Frozen-soil characteristics are divided into two basic groups based on whether or not segregated ice can be seen with the naked eye (Table 5.20). Third, the ice present in the frozen soil is classified; this refers to inclusions of ice that exceed 25 mm in thickness.

The amount of segregated ice in a frozen mass of soil depends largely on the intensity and rate of freezing. When freezing takes place quickly, no layers of ice are visible, whereas slow

Table 5.20. Description and classification of frozen soils (from Andersland, 1987)

I. Description of soil phase (independent of frozen state)	Classify soil phase by the Unified Soil Classification system				
	Major group		Subgroup		
	Description	Designation	Description		Designation
	Segregated ice not visible by eye	N	Poorly bonded or friable		Nf
			Well bonded	No excess ice	Nbn
				Excess ice	Nbe
II. Description of frozen soil			Individual ice crystals or inclusions		Vx
	Segregated ice visible by eye (ice 25 mm or less thick)	V	Ice coatings on particles		Ve
			Random or irregularly oriented ice formations		Vr
			Stratified or distinctly oriented ice formations		Vs
III. Description of substantial ice strata	Ice greater than 25 mm thick	ICE	Ice with soil inclusions		ICE + soil type
			Ice without soil inclusions		ICE

freezing produces visible layers of ice of various thicknesses. Ice segregation in soil also takes place under cyclic freezing and thawing conditions.

Mechanical Properties of Frozen Soil

The presence of masses of ice in a soil means that as far as engineering is concerned, the properties of both have to be taken into account. Ice has no long-term strength, that is, it flows under very small loads. If a constant load is applied to a specimen of ice, instantaneous elastic deformation occurs. This is followed by creep, which eventually develops a steady state. Instantaneous elastic recovery takes place on removal of the load, followed by recovery of the transient creep.

The relative density influences the behaviour of frozen coarse soils, especially their shearing resistance, in a manner similar to that when they are unfrozen. The cohesive effects of the ice matrix are superimposed on the latter behaviour, and the initial deformation of frozen sand is dominated by the ice matrix. Sand in which all the water is more or less frozen exhibits a brittle type of failure at low strains, for example, at around 2% strain. The water content of coarse soils is converted almost wholly into ice at a very few degrees below freezing point. Hence, frozen coarse soils exhibit a reasonably high compressive strength only a few degrees below freezing, and there is justification for using this parameter as a design index of their perform-ance in the field, provided that a suitable factor of safety is incorporated. The order of increase in compressive strength with decreasing temperature is shown in Figure 5.14.

On the other hand, frozen clay, in addition to often containing a lower content of ice than sand, has layers of unfrozen water (of molecular proportions) around the clay particles. These molecular layers of water contribute towards a plastic type of failure. In fact, in fine sediments, the intimate bond between the water and clay particles results in a significant proportion of soil moisture remaining unfrozen at temperatures as low as $-25°C$. The more the clay material in the soil, the greater is the quantity of unfrozen moisture. As far as the unconfined compressive strength of frozen clays is concerned, there is a dramatic increase in strength with decreasing temperature. In fact, it appears to increase exponentially with the relative proportion of frozen moisture. Using silty clay as an example, the amount of moisture frozen at $-18°C$ may be only 1.25 times that frozen at $-5°C$, but the increase in compressive strength may be more than fourfold.

Because frozen ground is more or less impermeable, this increases the problems due to thaw by impeding the removal of surface water. What is more, when thaw occurs, the amount of water liberated may greatly exceed that originally present in the melted out layer of the soil (see below). As the soil thaws downwards, the upper layers become saturated, and since
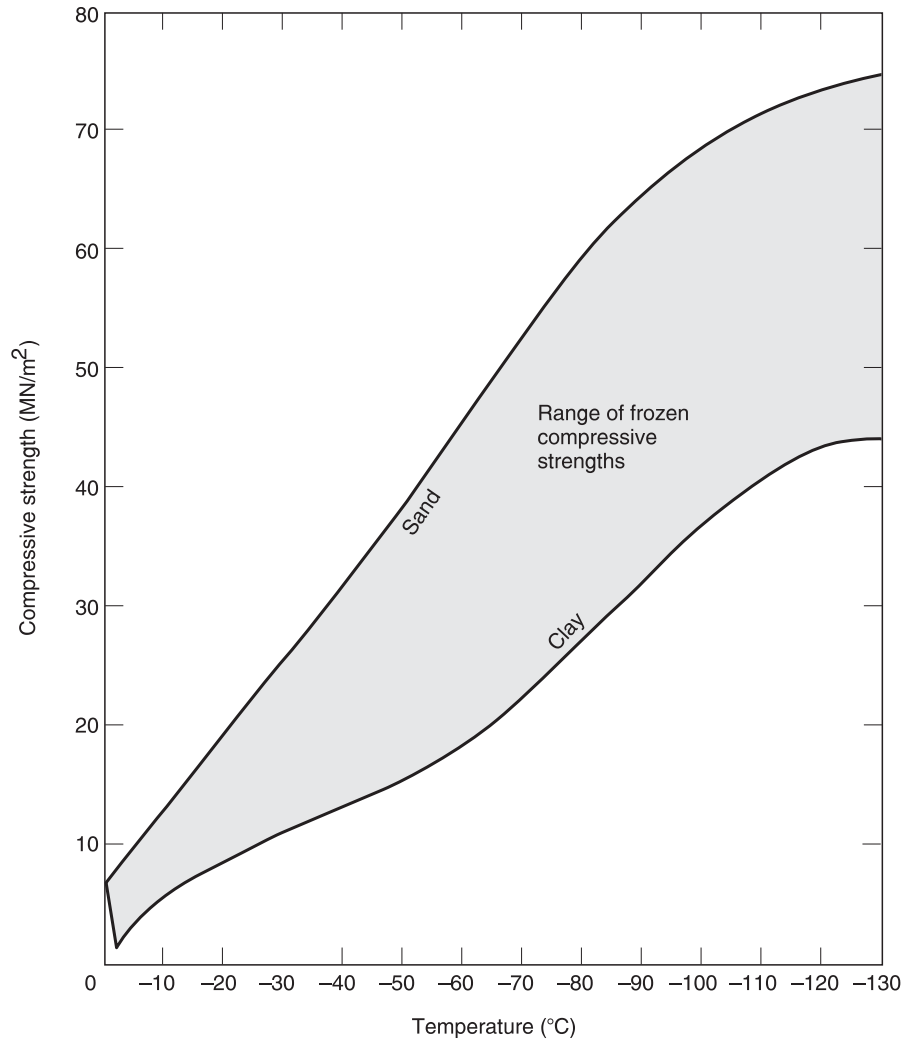
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Figure 5.14

Increase in compressive strength with decreasing temperature.

water cannot drain through the frozen soil beneath, they may suffer a complete loss of strength. Indeed, under some circumstances excess water may act as a transporting agent, thereby giving rise to soil flows.

Thaw settlement is associated with the thawing of frozen ground. As ice melts, settlement occurs, water being squeezed from the ground by overburden pressure or by any applied loads. Excess pore water pressures develop when the rate of ice melting is greater than the

discharge capacity of the soil. Since excess pore pressures can lead to the failure of slopes and foundations, both the rate and amount of thaw settlement should be determined. Pore water pressures also should be monitored.

Further consolidation, due to drainage, may occur on thawing. If the soil was previously in a relatively dense state, then the amount of consolidation is small. This situation only occurs in coarse frozen soils containing very little segregated ice. On the other hand, some degree of segregation of ice is always present in fine frozen soils. For example, lenses and veins of ice may be formed when silts have access to capillary water. Under such conditions, the moisture content of the frozen silts significantly exceeds the moisture content present in their unfrozen state. As a result, when such ice-rich soils thaw under drained conditions, they undergo large settlements under their own weight.

Frost Heave

The following factors are necessary for the occurrence of frost heave, namely, capillary saturation at the beginning and during the freezing of soil, plentiful supply of subsoil water and soil possessing fairly high capillarity together with moderate permeability. Grain size is another important factor influencing frost heave. For example, gravels, sands and clays are not particularly susceptible to heave, whereas silts definitely are. The reason for this is that silty soils are associated with high capillary rises, but at the same time their voids are large enough to allow moisture to move quickly enough for them to become saturated rapidly. If ice lenses are present in clean gravels or sands, then they simply represent small pockets of moisture that have been frozen. Casagrande (1932) suggested that the particle size critical to the development of frost heave was 0.02 mm. If the quantity of such particles in a soil is less than 1%, then no heave is to be expected, but considerable heaving may take place if the amount is over 3% in non-uniform soils and over 10% in very uniform soils.

As heave amounting to 30% of the thickness of the frozen layer have frequently been recorded, moisture other than that initially present in the frozen layer must be drawn from below, since water increases in volume by only 9% when frozen. In fact, when a soil freezes, there is an upward transfer of heat from the groundwater towards the area in which freezing is occurring. The thermal energy, in turn, initiates an upward migration of moisture within the soil. The moisture in the soil can be translocated upwards either in the vapour or liquid phase or by a combination of both. Moisture diffusion by the vapour phase occurs more readily in soils with larger void spaces than in fine soils. If a soil is saturated, migration in the vapour phase cannot take place.

Organic Soils: Peat

Peat is an accumulation of partially decomposed and disintegrated plant remains that have been fossilized under conditions of incomplete aeration and high water content (Hobbs, 1986). Physico-chemical and biochemical processes cause this organic material to remain in a state of preservation over a long period of time.

Macroscopically, peaty material can be divided into three basic groups, namely, amorphous granular, coarse fibrous and fine fibrous peat (Landva and Pheeney, 1980). The amorphous granular peat has a high colloidal fraction, holding most of its water in an adsorbed rather than free state. In the other two types, the peat is composed of fibres, these usually being woody. In the coarse variety, a mesh of second-order size exists within the interstices of the first-order network, and in fine fibrous peat, the interstices are very small and contain colloidal matter.

The ash percentage of peat consists of the mineral residue remaining after its ignition, which is expressed as a fraction of the total dry weight. Ash contents may be as low as 2% in some highly organic peat, or it may be as high as 50%. The mineral material is usually quartz sand and silt. In many deposits, the mineral content increases with depth. The mineral content influences the engineering properties of peat.

The void ratio peat ranges between 9, for dense amorphous granular peat, and 25, for fibrous types with a high content of sphagnum (Table 5.21). It usually tends to decrease with depth within a peat deposit. Such high void ratios give rise to phenomenally high water contents. The latter is the most distinctive characteristic of peat. Indeed, most of the peculiarities in the physical characteristics of peat are attributable to the amount of moisture present. This varies according to the type of peat; it may be as low as 500% in some amorphous granular varieties, although values exceeding 3000% have been recorded from coarse fibrous varieties.

The volumetric shrinkage of peat increases up to a maximum and then remains constant, the volume being reduced almost to the point of complete dehydration. The amount of shrinkage that can occur generally ranges between 10 and 75% of the original volume of the peat, and it can involve reductions in void ratio from over 12 down to about 2.

Amorphous granular peat has a higher bulk density than the fibrous types. For instance, in the former, it can range up to 1.2 Mg m^{-3}, whereas in woody fibrous peat, it may be half this figure. However, the dry density is a more important engineering property of peat, influencing its behaviour under load. Dry densities of drained peat fall within the range of 65–120 kg m^{-3}. The dry density is influenced by the mineral content, and higher values than those quoted can be obtained

Table 5.21. Some properties of bog peat from Pant Dedwydd, North Wales (after Nichol and Farmer, 1998). With kind permission of Elsevier

Depth (m)	Moisture content (%)	pH	Organic content (%)	Bulk unit weight (kN m^{-3})	Dry unit weight (kN m^{-3})	Specific gravity	Initial void ratio (e_o)	Coefficient of volume change m_v (m^2 MN^{-1})		Compression index, C_c	
								a	b	a	b
1.5	894	4.0	86.2	10.4	1.05	1.51	13.38	11.34	2.17	7.02	10.76
2.0	561	4.4	67.6	10.2	1.55	1.67	9.77	8.91	1.46	4.13	5.42
2.5	620	3.8	69.0	11.6	1.61	1.65	9.24	11.12	2.23	4.91	7.87
3.0	795	3.8	75.7	11.2	1.26	1.59	11.61	11.74	2.11	6.38	9.17
3.5	971	4.3	61.5	10.1	0.94	1.73	17.40	11.77	1.94	9.33	12.31
4.0	662	4.1	81.6	10.2	1.34	1.54	10.49	10.26	1.60	5.08	6.34
4.5	583	4.1	63.8	10.3	1.51	1.70	10.26	11.51	1.66	5.58	6.44
5.5	943	4.4	79.9	10.6	1.02	1.56	14.29	10.80	1.57	7.17	8.28
6.5	965	4.3	75.6	9.9	0.92	1.59	16.28	8.76	2.32	6.14	13.82

Note: Load ranges, σ_v: (a) 12.5–25 kN m^{-2}; (b) 100–200 kN m^{-2}.