Engineering geology of Quaternary soils: I. Processes and properties

M. G. Culshaw¹, J. C. Cripps², F. G. Bell³ & C. F. Moon⁴

INTRODUCTION

Soils and rocks of Quaternary age are probably the commonest geological material occurring at, or close to, the Earth's surface. Whilst Quaternary rocks may be similar to those formed earlier, the soils have characteristics that distinguish them from older soils; for example, they are being laid down at the present time and are subject particularly to modification by active geological/geomorphological processes.

Because of their surface occurrence, Quaternary soils are of particular interest to engineering geologists and geotechnical engineers. Not only do many engineering structures have to be founded on, or in, these soils, but some of the soils have characteristics, such as low strength and high compressibility, which make them more difficult to use as a foundation material. Yet, despite their importance to civil engineering and construction, Quaternary deposits frequently do not receive the same attention in geology and civil engineering courses as older materials. In addition, geological maps in many countries do not distinguish Quaternary deposits sufficiently. For example, alluvium frequently is represented with little attempt to identify lateral variations in lithology or to portray changes with depth, even though it forms the founding material for part, or all, of many of the world’s cities.

As Quaternary deposits are young, their geological, and hence engineering, character is controlled largely by the processes that brought about their formation and they have been less altered by processes that act over long periods of time, such as diagenesis. Consequently, climate has a significant influence on the nature of the material formed and its engineering characteristics. In cold climatic conditions processes of freeze and thaw have an effect on the way that materials behave, while in tropical climates rapid weathering with leaching has a major influence.

This paper discusses different Quaternary materials in terms of the climatic environment under which they were formed. These environments are principally, the glacial, periglacial, temperate, arid and tropical climatic zones.

Some deposits, for example those laid down under fluviatile or shallow marine conditions may be similar, regardless of the climatic environment of deposition. Such materials are considered under the section on temperate soils, even though they may be found extensively elsewhere (for example, in the tropics). Consequently, the soils considered under the other climatic zones are those that have characteristics peculiar to the climatic conditions under which they formed.

GLACIAL DEPOSITS

TILL

Till is regarded as being synonymous with boulder clay and is deposited directly by ice. The character of a till deposit depends on the lithology of the material from which it was derived, the position in which it was transported in the glacier, and the mode of deposition. The underlying
bedrock usually constitutes up to about 80% of basal tills, depending on its resistance to abrasion and plucking. The uppermost tills of a sequence contain a high proportion of far-travelled material and may not contain any of the local bedrock. Till sheets can comprise one or more layers of different material, not all of which are likely to be found at any one locality. Shrinking and reconstituting of an ice sheet can complicate the sequence.

Deposits of till consist of a variable assortment of rock debris ranging from fine rock flour to boulders. At one extreme, tills may consist mainly 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, slickensided clay frequently are encountered in some tills. Most tills contain a significant amount of quartz in their silt-clay fractions.

Distinction has been made between tills derived from rock debris which was carried along at the base of a glacier and those deposits which were transported within and on the ice. The former is referred to as lodgement till whereas the latter is known as ablation till. More elaborate classifications of till have been proposed, for example, by Elson (1961) and by McGown and Derbyshire (1977). Lodgement till is thought to be plastered onto the ground beneath the moving glacier in small increments as the basal ice melts. Because of the overlying weight of ice such deposits are overconsolidated. Consequently, lodgement till is commonly stiff, dense and relatively incompressible and, hence, practically impermeable. Lodgement till contains fewer, smaller stones (they generally possess a preferred orientation in the path of ice movement) than ablation till; these stones are rounded and striated. Due to abrasion and grinding the proportion of silt and clay size material is relatively high in lodgement till (for example, the clay fraction varies from 15-40%).

Ablation till accumulates on the surface of the ice when englacial debris melts out, and as the glacier decays the ablation till is slowly lowered to the ground. Therefore, it is normally consolidated. Because it has not been subjected to much abrasion, such till is characterized by abundant large stones 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%). Clast orientation in ablation till varies from almost random to broadly parallel to the ice flow direction. It may have an extremely low in situ density. Since ablation till consists of the sediment load carried at the time of ablation it usually forms a thinner deposit than lodgement till.

Tills are often fissile, tending to split into irregular lenticular flakes, ranging from less than a millimetre thick in clay tills, up to several tens of millimetres in sandy tills. McGown et al. (1977) noted a definite preferred orientation of fissures in some Scottish tills and that their intensity increased as the surface of the till was approached. This was attributed to greater stresses caused by ice movement and to the effects of weathering. Fissures frequently are present in lodgement till, especially if it is clay matrix dominated. Subhorizontal fissures have been developed as a result of incremental loading and periodic unloading whilst subvertical fissures owe their formation to the overriding effects of ice and stress relief. Joints commonly extend obliquely through massive, clay tills; some may be caused by shearing and others by desiccation after deposition. Some joints contain thin layers of compact sand.

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) 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. Large, often very local, variations can occur in the gradings of till which reflect local variations in the formation processes, particularly that of comminution. The clast size consists principally of rock fragments and composite grains, and presumably was formed by frost action and crushing by ice. Single grains predominate in the matrix. 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 consistency limits of tills are dependent upon water content, grain size distribution and the properties of the fine grained fraction. However, their plasticity indices generally are small, with the liquid limits decreasing with increasing grain size.

In heavily overconsolidated unweathered lodgement tills the natural moisture is
generally rather low and slightly below that of the plastic limit. Hence, the liquidity indices of such tills typically lie within the range -0.1 to -0.35. When the plasticity indices of tills are plotted against their liquid limits on the plasticity chart they tend to fall above the A-line (Fig. 1). They also tend to fall along the T-line of Boulton (1976), indicating the unsorted nature of the tills, their somewhat different locations reflecting the composition and particle size distribution of their matrix material (Bell 1991, Bell and Forster 1991).

The compressibility and consolidation characteristics of tills are determined principally by the clay content. For example, the value of compressibility index tends to increase linearly with increasing clay content whilst for tills of very low clay content, less than 2%, this index remains about constant ($C_c = 0.01$). Dense, heavily overconsolidated till is relatively incompressible and when loaded undergoes little settlement, most of which is elastic.

In a survey of dense till, Radhakrishna and Klym (1974) found that the undrained shear strength, as obtained by pressuremeter and plate loading tests, averaged around 1.6 MPa, while the values from triaxial tests ranged between 0.75 and 1.3 MPa. The average values of the initial modulus of deformation were around 215 MPa which was approximately twice the laboratory value. These differences between field and laboratory results were attributed to stress relief of material on sampling and sampling disturbance. Much lower values of shear strength were found by Bell (1991) and Bell and Forster (1991) for tills in Norfolk and

Table 1 A weathering scheme for lodgement tills from Northumberland (after Eyles and Sladen 1981)

<table>
<thead>
<tr>
<th>WEATHERING STATE</th>
<th>ZONE</th>
<th>DESCRIPTION</th>
<th>MAXIMUM DEPTH (m)</th>
</tr>
</thead>
</table>
| Highly weathered | IV   | Oxidized till and surficial material  
Highly weathered till  
Strong oxidation colours  
High rotten boulder content  
Leached 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 fluvio-glacial sediments | 8 |
| Slightly weathered | II  | Selective oxidation along fissure surfaces where present, otherwise as Zone I | 10 |
| Unweathered | I   | Unweathered till  
No post-depositionally rotten boulders  
No oxidation  
No leaching of primary carbonate  
Usually dark grey |
Fig. 2 a) Northumberland lodgement tills: carbonate content, undrained strength, moisture content, Atterberg indices and liquidity index versus depth at a single representative site (Sandy Bay).

b) Particle size distribution envelopes for weathered and unweathered Northumberland lodgement tills. (After Eyles & Sladen 1981).

Table 2 Typical geotechnical properties of lodgement tills from Northumberland (after Eyles and Sladen 1981)

<table>
<thead>
<tr>
<th>PROPERTY</th>
<th>WEATHERING ZONES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
</tr>
<tr>
<td>Bulk density Mg/m³</td>
<td>2.15-2.3</td>
</tr>
<tr>
<td>Natural moisture content (%)</td>
<td>10-15</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>25-40</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>12-20</td>
</tr>
<tr>
<td>Plasticity index</td>
<td>0-20</td>
</tr>
<tr>
<td>Liquidity index</td>
<td>-0.20 to -0.05</td>
</tr>
<tr>
<td>Grading of fine (&lt;2mm) fraction</td>
<td></td>
</tr>
<tr>
<td>% clay</td>
<td>20-35</td>
</tr>
<tr>
<td>% silt</td>
<td>30-40</td>
</tr>
<tr>
<td>% sand</td>
<td>30-50</td>
</tr>
<tr>
<td>Average activity</td>
<td>0.64</td>
</tr>
<tr>
<td>(c'(kPa))</td>
<td>0-15</td>
</tr>
<tr>
<td>(\phi'(\text{degrees}))</td>
<td>32-37</td>
</tr>
<tr>
<td>(\phi_r'\text{(degrees)})</td>
<td>30-32</td>
</tr>
</tbody>
</table>
The presence of fissures influences the shear strength of tills. For example, McGown et al. (1977) showed that the opening of fissures sympathetically orientated to cut slopes, and softening of till along fissures as a result of weathering, gave rise to a rapid reduction of undrained shear strength along the fissures. They found that the undrained shear strength of fissures in till may be as little as one-sixth that of the intact soil. They emphasized that the distinction between the nature of the various fissure coatings (sand, silt or clay-size material) is of critical importance in determining the shear strength behaviour of the fissured soil mass. Deformation and permeability also are controlled by the nature of the fissure surface and coatings.

Eyles and Sladen (1981) recognized four zones of weathering within the soil profile of lodgement till in the coastal area of Northumberland (Table 1). As the degree of weathering of the till increases, so does the plasticity of the clay fraction and moisture content. This, in turn, leads to changes in the liquid and plastic limits and the shear strength (Table 2 and Fig. 2).

FLUVIO-GLACIAL DEPOSITS; STRATIFIED DRIFT

Stratified deposits of drift are often subdivided into two categories, namely, those which accumulate beyond the limits of the ice, forming in streams, lakes or seas, and those deposits which develop in contact with the ice. The former type are referred to as pro-glacial deposits whilst the latter are termed ice-contact deposits.

Most melt-water streams which deposit outwash fans do not originate at the snout of a glacier but from within, or upon, the ice. Many of the streams which flow through a glacier have steep gradients and, therefore, are efficient transporting agents. However, when they emerge at the snout, they do so on to a shallower incline and deposition results. Outwash deposits are typically cross-bedded and range in size from boulders to coarse sand. When first deposited the porosity of these sediments varies from 25 to 50%. Consequently, they are very permeable and can resist erosion by local run-off. The finer silt-clay fraction is transported further downstream. Also in this direction an increasing amount of stream alluvium is contributed by tributaries so that eventually the fluvio-glacial deposits cannot be distinguished. Most outwash masses are terraced.

Valley trains are outwash deposits which are confined within long narrow valleys. Since deposition occurs more rapidly at the centres of valleys than at the sides, the deposits are thickest there. If outwash quickly accumulates in a valley it may eventually dam tributary streams so that small lakes form along the sides of the main valley. Such deposition also may bury small watersheds and divert pro-glacial streams.

Five different types of stratified drift deposited in glacial lakes have been recognized, namely, terminal moraines, deltas, bottom deposits, ice-rafted erratics and beach deposits. Terminal moraines that formed in glacial lakes differ from those which arose on land in that lacustrine deposits are inter-stratified with drift. Glacial lake deltas are usually composed of sands and gravels which are typically cross-bedded. By contrast, those sediments which accumulated on the floors of glacial lakes are fine grained, consisting of silts and clays. These fine grained sediments are sometimes composed of alternating laminae of finer and coarser grain size. Each couplet has been termed a varve and sediments so stratified are consequently described as varved (see below). Large boulders which occur on the floors of glacial lakes were transported on rafts of ice and deposited when the ice melted. Usually the larger the glacial lake, the larger were the beach deposits which developed about it. If changes in lake level took place, then these may be represented by a terraced series of beach deposits.

Deposition which takes place at the contact of a body of ice with the ground is frequently sporadic and irregular. Locally the sediments possess a wide range of grain size, shape and sorting. Most are granular and variations in their engineering properties reflect differences in particle size distribution and shape. Deposits often display abrupt changes in lithology and consequently in relative density (or density index). They are characteristically deformed since they sag, slump or collapse as the ice supporting them melts.

Kame terraces are deposited by melt-water streams which flow along the contact between the ice and the valley side. The drift is derived principally from the glacier, although some is supplied by tributary streams. They occur in pairs, one each side of the valley. The surfaces of these terraces are often pitted with kettle holes.

Kames are mounds of stratified drift which originate as small deltas or fans built against the snout of a glacier where a tunnel in the ice, along which melt-water travels, emerges. Other small ridge-like kames accumulate in crevasses in stagnant or
Table 3 Some properties of varved clays from the Elk Valley, British Columbia (after George 1986)

<table>
<thead>
<tr>
<th>Varved clay description (typical)</th>
<th>Bulk</th>
</tr>
</thead>
<tbody>
<tr>
<td>-yellowish brown;</td>
<td>17.85</td>
</tr>
<tr>
<td>-typical thicknesses</td>
<td>CL</td>
</tr>
<tr>
<td>&quot;clay&quot; 33-37 mm</td>
<td>35.4</td>
</tr>
<tr>
<td>&quot;silt&quot; 2-8 mm</td>
<td>34.3</td>
</tr>
<tr>
<td>Usually fine silty to sandy laminae and lenses (&lt;0.5 mm thick) in clay layers, 2-14 mm</td>
<td>22.4</td>
</tr>
<tr>
<td>Soil properties</td>
<td>1.09</td>
</tr>
<tr>
<td>Unit weight kN/m³</td>
<td>(0.496)</td>
</tr>
<tr>
<td>USCS Classification</td>
<td>0.405-0.587</td>
</tr>
<tr>
<td>Natural moisture content (%)</td>
<td>0.496</td>
</tr>
<tr>
<td>Liquid limit (%)</td>
<td>290-375</td>
</tr>
<tr>
<td>Plastic limit (%)</td>
<td>17.09</td>
</tr>
<tr>
<td>Liquidity index</td>
<td>0.36</td>
</tr>
<tr>
<td>% Clay (&lt;0.002 mm)</td>
<td>-</td>
</tr>
<tr>
<td>Activity</td>
<td>-</td>
</tr>
<tr>
<td>Compression index</td>
<td>-</td>
</tr>
<tr>
<td>Effective preconsolidation pressure (kPa)</td>
<td>-</td>
</tr>
</tbody>
</table>

near-stagnant ice.

Eskers are long, narrow, sinuous, ridge-like masses of stratified drift which are unrelated to surface topography. They represent sediments deposited by streams which flowed within channels in a glacier. Eskers may reach up to 50 m in height, whilst they range up to 200 m wide. Their sides are often steep. Eskers are composed principally of sands and gravels, although silts and boulders are found within them. These deposits are generally cross-bedded.

The most familiar pro-glacial deposits are varved clays. These deposits accumulated in pro-glacial lakes and generally are characterized by alternating laminae of finer and coarser grain size, each couplet being termed a varve. The thickness of the individual varve is frequently less than 2 mm, although much thicker layers have been noted. Generally, the coarser layer is of silt size and the finer of clay size.

Usually very finely comminuted quartz, feldspar and mica form the major part of varved clays rather than clay minerals. For example, the clay mineral content may be as low as 10%, although instances where it has been as high as 70% have been recorded. For example, Bell and Coulthard (1991) showed that the bulk mineralogical composition of the Tees Laminated Clay was dominated by illite and kaolinite with lesser amounts of chlorite. Similarly, George (1986) found that kaolinite and illite were the dominant minerals in the clay-size material of the varved clay of the Elk Valley, British Columbia. Montmorillonitic clay also has been found in varved clays.

Taylor et al. (1976) showed that in Devensian clays from Gale Common in Yorkshire, the clay minerals were well-orientated around silt grains such that at boundaries between silty partings and matrix the clay minerals tended to show a high degree of orientation parallel to the laminae. A similar situation occurs in the Tees Laminated Clay (Bell and Coulthard 1991) and in the varved clay of the Elk Valley, British Columbia (George 1986). According to Wu (1958) the fabric of the glacial lake clays found around the shores of the Great Lakes in North America varies from well-orientated to almost random. He suggested that the latter must involve a flocculent or honeycomb structure.

The range of liquid limits for varved clays tends to vary between 30 and 80% whilst that of plastic limit often varies between 15 and 30%. These limits, obtained from varved clays in Ontario (Metcalf and Townsend 1961) allow the material to be classified as inorganic silty clay of medium to high plasticity. In some varved clays in Ontario the natural moisture content would appear to be near the liquid limit. They are consequently soft and have sensitivities generally of the order of four. Since triaxial and unconfined compression tests tended to give very low strains at failure, around 3%, Metcalf and Townsend (1961) presumed that this indicated a structural effect in the varved clays. The average strength reported was about 40 kPa, with a range of 24 to 49 kPa. The effective stress parameters of apparent cohesion and angle of shearing resistance ranged from 5 to 19 kPa and 22 to 25° respectively.
Table 4 Engineering properties of sensitive soils (after Gillott 1979)

<table>
<thead>
<tr>
<th>LOCATION*</th>
<th>DEPTH (m)</th>
<th>NATURAL MOISTURE CONTENT (%)</th>
<th>PRECONSOLIDATION PRESSURE (kPa)</th>
<th>UNDRAINED STRENGTH (kPa)</th>
<th>SENSITIVITY (%)</th>
<th>LIQUID LIMIT (%)</th>
<th>PLASTIC LIMIT (%)</th>
<th>LIQUIDITY INDEX</th>
<th>ACTIVITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>O</td>
<td>13.7</td>
<td>60</td>
<td>450</td>
<td>160</td>
<td>-</td>
<td>49</td>
<td>23</td>
<td>1.4</td>
<td>0.35</td>
</tr>
<tr>
<td>Q</td>
<td>5.2</td>
<td>75</td>
<td>150</td>
<td>50</td>
<td>-</td>
<td>70</td>
<td>26</td>
<td>1.1</td>
<td>0.64</td>
</tr>
<tr>
<td>Q</td>
<td>14.3</td>
<td>81</td>
<td>150</td>
<td>50</td>
<td>-</td>
<td>65</td>
<td>28</td>
<td>1.4</td>
<td>0.45</td>
</tr>
<tr>
<td>O</td>
<td>2.6</td>
<td>65</td>
<td>60</td>
<td>20</td>
<td>100</td>
<td>55</td>
<td>22</td>
<td>1.3</td>
<td>0.73</td>
</tr>
<tr>
<td>Q</td>
<td>12.2</td>
<td>28</td>
<td>590</td>
<td>230</td>
<td>-</td>
<td>23</td>
<td>16</td>
<td>1.7</td>
<td>0.18</td>
</tr>
<tr>
<td>O</td>
<td>5.2</td>
<td>78</td>
<td>320</td>
<td>120</td>
<td>-</td>
<td>65</td>
<td>28</td>
<td>1.3</td>
<td>0.44</td>
</tr>
<tr>
<td>BC</td>
<td>20.1</td>
<td>38</td>
<td>-</td>
<td>20</td>
<td>30</td>
<td>28</td>
<td>22</td>
<td>2.7</td>
<td>0.22</td>
</tr>
<tr>
<td>BC</td>
<td>14.0</td>
<td>29</td>
<td>-</td>
<td>-</td>
<td>4</td>
<td>23</td>
<td>16</td>
<td>1.9</td>
<td>0.33</td>
</tr>
<tr>
<td>BC</td>
<td>35.4</td>
<td>37</td>
<td>-</td>
<td>60</td>
<td>5</td>
<td>28</td>
<td>23</td>
<td>2.8</td>
<td>0.17</td>
</tr>
<tr>
<td>A</td>
<td>61.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>26</td>
<td>21</td>
<td>0.8</td>
<td>-</td>
</tr>
<tr>
<td>A</td>
<td>60.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>23</td>
<td>20</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

* O = Ontario, Q = Quebec, BC = British Columbia, A = Alaska

Varved clays tend to be normally consolidated or lightly overconsolidated (Table 3), although it is usually difficult to make the distinction. In many cases the precompression may have been due to ice loading. They generally are highly compressible. However, Saxena et al. (1978) reported that the upper part of the varved clay of Hackensack Valley, New Jersey was highly overconsolidated. This they attributed to the effects of fluctuating water levels or to desiccation.

QUICK CLAYS

The material of which quick clays are composed is predominantly smaller than 0.002 mm but many deposits seem to be very poor in clay minerals, containing a high proportion of ground-down fine quartz. For instance, it has been shown that quick clay from St Jean Vienney consists of very fine quartz and plagioclase. Furthermore, in a recent review of the mineralogy, chemistry and physical properties of sensitive clays from eastern Canada, Locat et al. (1984) found that plagioclase was the dominant mineral (from 25 to 48%) followed by quartz, microcline and hornblende. Small quantities of dolomite and calcite also were present. The phyllosilicates (including the clay minerals of which illite was the most common), together with the amorphous materials never represented more than one third of the minerals present. Indeed, examination of quick clays with the scanning electron microscope has revealed that they do not possess clay-based structures, although such work has not lent unequivocal support to the view that non-clay particles govern the physical properties.

Cabrera and Smalley (1973) suggested that quick clays owe their distinctive properties to the predominance of short range interparticle bonding forces which they maintained were characteristic of deposits in which there was an abundance of glacially produced, fine non-clay minerals. In other words, they contended that ice sheets supplied abundant ground quartz in the form of rock flour for the formation of quick clays. Certainly quick clays have a restricted geographical distribution, occurring in certain parts of the northern hemisphere which were subjected to glaciation during Pleistocene times.

The open fabric which is characteristic of quick clays has been attributed to their initial deposition, during which time colloidal particles interacted to form loose aggregations by gelation and flocculation. Clay minerals exhibit strongly marked colloidal properties and other inorganic materials such as silica behave as colloids when sufficiently fine grained. Gillott (1979) suggested that the open fabric may have been retained during very early consolidation because it remained a near equilibrium arrangement. Its subsequent retention to the present day may be due to mutual interference between particles and buttressing of junctions between granules by clay and other fine constituents, precipitation of cement at particle contacts, low rates of loading, and low load increment ratio. Gillott (1979) has shown that the fabric and mineralogical composition of sensitive soils from Canada, Alaska and Norway are qualitatively similar. He pointed out that they all possess an open fabric, high moisture content and similar index properties (Table 4).
Quick clays often exhibit little plasticity, their plasticity indices often varying between 8 and 12%. Their liquidity indices normally exceed 1, and their liquid limits are often less than 40%. Quick clays are usually 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. The small fraction of the original strength gained 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 can ever be recovered is because the rate at which it develops is so slow. As an example, the Leda Clay is characterized by exceptionally high sensitivity, commonly between 20 and 50, and a high natural moisture content and void ratio, the latter being commonly about 2. It has a low permeability of around $10^{-10}$ m/s. The plastic limit is approximately 25% with a liquid limit about 60% and an undrained shear strength of 700 kPa. When subjected to sustained load, an undrained triaxial specimen of Leda Clay exhibits a steady time-dependent increase in both pore water pressure and axial strain. Continuing undrained creep may often result in a collapse of the sample after long periods of time have elapsed. The natural moisture contents of the sensitive clays examined by Locat et al. (1984) always exceeded the plastic limits and commonly exceeded the liquid limits. Hence, their liquidity indices were greater than 1 and sometimes exceeded 2. The undrained shear strength was as low as 23 kPa and never rose above 250 kPa. Strength decreased and sensitivity increased dramatically as liquidity index increased, the strength more or less disappearing on remoulding, giving sensitivity values varying from 24 to over 1000. The variation in the geotechnical properties of these soils was primarily attributed to their differences in mineralogy and texture.

Quick clays are associated with several serious engineering problems. Their bearing capacity is low, and settlement is high. Slides in quick clays sometimes have proved disastrous and they can liquefy on sudden shock.

PERIGLACIAL DEPOSITS

It appears that there still exists some confusion regarding the term "periglacial." The origin of the word is attributed to Loziński (1912) but this author omitted to define it (Embleton and King 1975). Pêwé (1969) observed: "No single definition acceptable to everyone concerned with studying the periglacial environment has been formulated...."

The problem is whether "periglacial" should define a specific climate, an environment or the manifestation of certain diagnostic features or processes, for example, permafrost. If one considers the term in its widest sense, that is, those processes which operate around a glacial mass, then, to a certain extent, climate and environment are implicit factors. Such included aspects are frozen ground (permafrost) phenomena, associated frost action processes, resultant mass movements and distal deposits. Possibly, fluvio-glacial deposits should also be covered by the heading.

Engineering geologists frequently encounter periglacial deposits in diverse circumstances. In present day temperate climates the interest will involve "fossil" sediments of the Quaternary but current periglacial conditions have presented great problems, for example, the Alyeska (Trans-Alaska) oil pipeline. An excellent treatment of periglacial engineering geology in Great Britain has been provided by Higginbottom and Pookes (1970) to which the reader is referred.

FROZEN GROUND PHENOMENA

Considered here is permafrost and the various features associated with frost action. Foremost in any consideration of frozen ground is permafrost, a term proposed by Muller (1947) to describe: "a thickness of soil or other superficial deposit or even of bedrock ..... in which a temperature below freezing has existed continually for a long time". It is suggested that the "long time" referred to be a minimum of 2-3 years. Two main categories of permafrost are currently recognized:

(i) continuous permafrost in which frozen ground has an uninterrupted lateral extent (this layer may be in excess of 1000 m thick);

(ii) discontinuous permafrost which contains small, isolated areas of unfrozen ground (Fig. 3).

Frozen ground is covered by an 'active layer' which is frozen in winter and thawed in summer. This tends to act as an insulating layer and if disrupted, leads to thawing of the permafrost zone below.

In what is generally regarded as permafrost, ice is found to be spread as a continuous mass throughout a rock or soil as frozen pore or fissure water. However, ice may occur in separate, discrete forms. Such segregations may be ice veins and wedges, lenses and pingos.

Ice veins and wedges form in
Unfrozen ground at base of active layer

Permafrost (permanently frozen ground)

Unfrozen ground within permafrost (talik)

Unfrozen ground below permafrost

Fig. 3 Distribution of permafrost.

water-filled fissures in frost susceptible materials, usually those with a high silt content. It would seem that fissures initially form as a result of thermal contraction and when infilled with frozen water produce ice veins. Further contraction causes renewed fissuring and veins gradually increase in depth and width to form an ice wedge. Fossil ice wedges are recognizable by a secondary infilling of sand or gravel known as ice wedge casts. Thermal contraction often produces a pattern and ice wedges are sometimes seen to take on a polygonal arrangement when viewed in plan.

Pingos, or ice-cored mounds, have been extensively researched by Washburn (1973, 1979). Their formation is thought to be due to water which freezes below a lake or pond; freezing of the subsurface water begins at the periphery of the water mass and extends inwards. Total freezing causes subsequent expansion and hence the raising of ground level to form a mound. Fossil pingos are recognized by marshy, circular depressions and good examples are to be found in the Cledlyn Basin, Dyfed, Wales.

Long (1991) presented evidence for many of the fossil ground ice features mentioned above, for example, ice wedge polygons and what may be pingos in the bed of the North Sea. The evidence strongly suggests that this area once existed as a vast tundra plain during the Pleistocene epoch.

Involutions (sometimes referred to as cryoturbations) may be explained by the disruption of sediment by growing ice masses. Sharp (1942) described involutions as "aimless deformation, distribution and interpenetration of beds produced by frost action." Some dispute has arisen concerning the formation of these features, whether it be due to growing ice or, as is currently thought, distortion as a result of entrapped pore water rising upwards during the freezing process. Most examples have a peculiar, concave upwards, bulbous shape which could be produced by sandy material sinking into a less dense, finer grained sediment (c.f. load casts) (Fig. 4). These have been referred to as "drops." Often, the features can be of a very large scale and may manifest themselves as gravel pipes clearly intruded from some depth below ground level (Higginbottom and Fookes 1970).

PERIGLACIAL MASS MOVEMENT: FEATURES AND DEPOSITS

Periglacial mass movement phenomena, both past and present, are of great concern to the engineering geologist. Under this heading are included "solifluction", frost creep, cambering and valley bulging; talus formation and movement may also be added.

The term "solifluction" is well entrenched in the literature but has been criticized in that its literal translation ("earth/soil flow") implies no periglacial connection. It has been suggested that gelifluction "the flow of water-soaked debris resulting from seasonal thawing of the active layer" be more apposite (Embleton and King 1975). Therefore, it is possible to divide periglacial mass movement into three broad groups:

(i) gelifluction processes;
(ii) frost creep;
(iii) cambering and valley bulges.
Gelifluction

This term concerns the down slope flow of water-saturated soil and rock (regolith), the thawed active layer. It seems that movements can involve either flowing (involving internal deformation) or sliding (manifested by the development of a shear plane or planes). However, in the majority of cases, it is likely that both processes occur sequentially or simultaneously during movement.

The transported material is commonly known as "head" in the British Isles except when found in Chalk areas of southern England in which case it is referred to as "Coombe Rock." The term "guck" (that is, glacial muck) has been coined in the USA (Eyles and Paul 1983).

In general, the deposits are unsorted with a predominance of angular fragments indicative of a short transport distance. Since movement is essentially down slope, the composition of the head reflects the rock types outcropping in the valley sides, although in some cases the material might be re-worked till.

Crude stratification may be present in some deposits with indistinct layers parallel to the slope. The origin of these remains obscure. Elongate rock fragments are often orientated with their long axes pointing down slope (Wright 1991), and this feature may help in distinguishing the deposits from other, similar diamictons but this is by no means a diagnostic feature.

The silt content of head appears to be critical in the promotion of flowage of geliflucted regolith. Clearly, an important aspect of flow propagation is grading which controls permeability. In turn, permeability governs the rate of dissipation of pore water pressure in the material; a build up of pore pressure during thawing would instigate movement. High clay contents are not conducive to flow since clay minerals adsorb excess water which is then rendered unavailable for the flow process. A low clay content is reflected in the low plasticity indices of head deposits.

An important characteristic of gelifluction is that saturated debris is able to flow on slopes with gradients as low as 2-4°. The velocity of movement, as judged by the internal features of the deposits, is probably "moderate" to "rapid" (Varnes 1958). Displacements in which sliding has been the major mechanism, exhibit shear planes and are almost always translational rather than rotational. Under certain circumstances fossil periglacial "mudflows" may be reactivated by man's interference or a change in the level of the water table (Higginbottom and Fookes 1970).

It has been stated that it should be possible to distinguish between a flow and a slide. This has been discussed by Washburn (1979) and Moon (1985). For a slide to occur it is implicit that a failure plane be present, at least in part, and the nature of such planes has been rigorously researched (Skempton 1964). Failure surfaces and associated structural disturbances are readily visible within translational movements. Normally, the failure surface is more of a zone than a single plane and occurs perhaps at 1-2 m depth. Skempton et al. (1991) have carried out an extensive study of the Carsington (Derbyshire) dam failure and movement along a shear zone has also been recognized by Spink (1991) but at a greater depth. Some of these features have been attributed to the shearing of the...
Table 5 Shear resistance mobilized on gelification shear surfaces

<table>
<thead>
<tr>
<th>SOURCE MATERIAL</th>
<th>LOCATION</th>
<th>MOBILIZED SHEAR RESISTANCE</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Angle of internal friction (°)</td>
<td></td>
</tr>
<tr>
<td>Weald Clay</td>
<td>Kent</td>
<td>16*</td>
<td>Weeks (1969)</td>
</tr>
<tr>
<td>Fuller’s Earth (mudstone)</td>
<td>Avon</td>
<td>12*</td>
<td>Cook (1973)</td>
</tr>
<tr>
<td>Lias Clay</td>
<td>Northamptonshire</td>
<td>16*</td>
<td>Chandler (1970)</td>
</tr>
<tr>
<td>Carboniferous mudrocks</td>
<td>Staffordshire</td>
<td>14.5*</td>
<td>Earley &amp; Skempton (1972)</td>
</tr>
</tbody>
</table>

* back analysis; + drained triaxial test

Active (surficial) layer whilst others are due, it is held, to a thaw-collapse mechanism. Both Skempton et al. (1991) and Spink (1991), rely upon the development of residual shear strength as opposed to peak strength as the controlling factor in the sliding failure mechanism of slopes. This is a realistic view as many investigations have shown.

It has been suggested that gelification is stimulated by insolation on frozen slopes. This would mean that slopes having a certain orientation receive more warming than others and hence suffer more mass movement in a thawing active layer. This could result in asymmetrical valley cross profiles and Long (1991) has identified such profiles on a fossil periglacial surface on the bed of the North Sea. Although asymmetry undoubtedly exists within periglaciated areas "...it is abundantly clear that the steeper slope of the asymmetric pair may face in any direction." (Embleton and King 1975). Probably, factors other than insolation have operated.

Unconsolidated, gelified deposits exposed at coastal sites are especially prone to marine erosion as are glacial tills (for example, Devensian tills of the Yorkshire coast). The inherent lack of resistance to erosion of cliffed periglacial deposits has been focused on south west England; Hallsands and Start Point are classic areas. These are considered in detail by Kalaugher and Grainger (1991).

Wright (1991) considered mudflows in south Wales valleys; an area of much slope instability. In the area under study, head deposits overlie till and, since there is no evidence of a failure plane within, or at the lower boundary of the deposit, it is suggested that these are true flows. The author utilizes clast orientation to differentiate between gelified and true glacial material.

**Shear strength of gelification surfaces**

The shear resistance mobilized on gelification surfaces has been measured by back analysing failed slopes and by carrying out various laboratory tests. Some typical values determined by back analysis and in triaxial tests are presented in Table 5 for several source rocks. Owing to the dependence of the value obtained on the pore water pressure conditions which prevailed at the time of failure, these values may not apply in all cases. Differences of lithology and in the normal stress conditions also would give rise to variation from the values given.

The values are similar to residual shear strength values determined by shear box tests on pre-cut samples and samples containing natural shear planes. As a first approximation, it probably is reasonable to assume that this would apply for other formations of similar lithology. However, residual shear strength values obtained in ring shear tests on remoulded samples tend to give angles 3 to 6° lower than those given by back analysis or reversing shear box tests.

**Frost creep**

It is likely that gelification and frost creep operate simultaneously and their effects are difficult to separate: "the resulting deposit is therefore of joint origin" (Washburn 1973). The process by
which there is a general slow, downslope movement of regolith appears to be relatively simple. Materials involved must be frost susceptible (that is, silty) and permit the growth of ice crystals. These grow at right angles to a cold surface and in doing so, push rock particles towards the surface. When the ice crystals thaw, gravity takes over and causes the grains to be displaced a short distance downslope. Frost creep has been successfully modelled in the laboratory (Embleton and King 1975) and movement rates observed in the field. An analysis of various field results yields a "global" rate of approximately 20 mm/week, although this varies widely.

**Cambering and valley bulging**

These phenomena are included under the general heading Mass Movement in that they are indirectly connected with slope movement. Cambering commonly occurs on valley sides where a bed or beds of competent rock rests upon argillaceous sediments; usually the competent rock forms a capping to the valley slopes. Thawing of the frozen, downslope argillaceous rocks may cause them to be geliflucted, thus removing support for the upper competent rocks. These will then fail in tension, bend toward the valley bottom, become detached from the main mass and may be rafted down the valley side on an active, fluid layer. Associated with the cambering process are tension cracks which appear in the competent capping rock; subsequently these become filled with rock debris and are referred to as "gulls." A detailed observation of these processes is provided by Worssam (1981).

Valley bulges are frequently associated with cambering and are manifested as distorted beds in a valley bottom at depth. Generally, the distortion involves updoming of strata and associated dislocations. Such features are extremely problematic when associated with dam foundations since disruption greatly influences foundation bedrock permeability. The construction of many Pennine dams in the UK has encountered valley bulging and extensive remedial works have been necessary in order to overcome a highly permeable bedrock, for example, deep cut-off trenches and grout curtains.

The actual cause of valley bulges has for long been open to speculation; some authors attribute deformation to a later plastic flow of a clay rock towards the valley axis due to the weight of the valley-side overburden. Others see the process as being due to the growth of ice in rocks of the saturated valley floor. Parks (1991) provided an excellent consideration of current theories and attempts to resolve them.

**DISTAL PERIGLACIAL DEPOSITS**

The deposits referred to here include those that are held to have a glacial origin but have been transported and deposited within the periglacial environment. As a result of distant transport processes, the materials involved are usually well-sorted and upon deposition, acquire geotechnical properties peculiar to themselves. Transport mechanisms in operation include fluvial and aeolian; deposits may be continental or marine.

Barton et al. (1991) described the well-known Plateau Gravels of southern England. Although the origin of these widespread sediments has been much considered and attributed to deposition on river terraces in a periglacial zone, little attention has been paid to their geotechnical properties. The authors' preliminary examination encompasses an investigation of mineralogy, granulometry and shear strength characteristics. However, the problem remains as to why these so-called cohesionless materials are able to sustain vertical slopes; cohesion via cementation can only account for this.

Loess, an aeolian deposit is a silt-sized sediment originating from deflated deserts. The formation of loess has provoked much discussion and various authors (for example, Eyles and Paul 1983) describe loess as: "the most widespread of periglacial sediments." Confining the origin of loess to a periglacial environment is restrictive in that it is currently accepted that there are two possible modes of origin, that is, warm desert and cold desert. Most authors maintain that the majority of deposits are glacially derived but may be deposited in a wide variety of geographical areas.

The geotechnical properties of loess are well documented. Deposits have high porosity and low natural moisture content; they are prone to collapse on wetting which has given rise to considerable foundation problems (Feda 1966).

Finely ground rock particles, essentially quartz, that constitute loess are often cemented by calcium carbonate and this is thought to be responsible for the ability of loess to sustain vertical slopes in cuttings which sometimes approach 9 m in height. These cuts are quite stable until affected by excess moisture; they are collapsing soils. Loess is considered in more detail below.

Marginal to the periglacial zone are found water-lain clays which may occur in pro-glacial lakes or in a marine environment. Glacial lake clays are widespread within the UK and North America, well known examples being Lake Pickering in England and Lake Agassiz in the USA. The
seds have distinctive properties in that normally they are laminated due to varving. They are described in more detail above.

High sensitivity is common in certain clays found in areas close to Pre-Cambrian shields which have suffered intense Pleistocene glaciation. The available evidence shows that these sediments have been laid down in marine, brackish or freshwater conditions; eastern Canada and Scandinavia are notable for their occurrence.

Many of these soft clays exhibit extremely high sensitivities and in some samples, remoulding can convert a brittle solid into a viscous liquid. The consequent engineering implications are obvious in this context. Some deposits are varved and it is common to find that the natural moisture content exceeds the liquid limit; plasticity indices are characteristically low.

The origin of the so-called "quick-clays" is problematic and controversial. Various theories have been advanced and these include the development of sensitivity as a result of salt-leaching, a collapsed, flocculated clay microfabric or a high percentage of inactive clay-size particles (for a review see Cabrera and Smalley 1973). The evidence suggests that various processes may operate to produce clays of high sensitivity. Quick-clays are described more fully above.

It should be emphasized that there is a wide variation in the occurrence and properties of periglacial sediments also in the processes by which they are formed. The periglacial environment hosts deposits of diverse composition which may arise in a wide variety of engineering situations with consequent problems.

**TEMPERATE DEPOSITS**

The engineering behaviour of temperate deposits is a function of composition, structure and present environment, in particular their water content. The composition and structure of these deposits are controlled by the type of parental material, the conditions of formation or deposition and their subsequent geological history. A strong influence is exerted by the type of minerals present. Deposits containing significant quantities of clay minerals tend to display significant plasticity, the properties are very sensitive to the amount and composition of pore water and the materials tend to be both weaker and more compressible than materials consisting of clastic grains. The presence of organic matter may also make a significant difference to the engineering behaviour.

Although lithology exerts a notable control over behaviour, both micro- and macro-structures are also important. The micro-structure is influenced by a number of inter-related features including the shape, surface character, size distribution and the packing arrangement of the particles. Increased strength and reduced compressibility usually correlate with greater grain size and a wider distribution of particle sizes. Most deposits contain significant macro-structures produced by lithological variation, bedding surfaces and other sedimentary structures, joints and fissures. In some instances materials are partially cemented to varying degrees by the precipitation of interstitial cements. This is a feature of many soil profiles, especially in more arid regions.

Most temperate deposits are soil-like in character. They are deposited within the valleys of rivers, in lakes, estuaries and the sea. Some residual soil profiles also form under temperate conditions, particularly where weak, and rapidly weathered, rock types crop out. The constituent materials are produced by moderate degrees of concurrently acting physical and chemical weathering (Peltier 1950).

Due to the importance of frost action, physical weathering is likely to dominate in the colder regions at high latitude and high altitude. Physical weathering processes result in the formation of variously sized clastic detritus by the fragmentation of rocks and minerals. In warmer, humid areas chemical weathering leading to the production of clays and the precipitation of interstitial cements also is likely to become a significant process. Under suitable conditions significant accumulations of organic matter may be preserved in organic soils and peats.

The type of deposit formed also depends on the conditions of deposition. In turn, these are controlled by a number of factors including the topography of the area and the climatic conditions. Apart from some aeolian sands near to beaches and the downslope movement of material in mountainous areas, the principal means by which weathering products are transported is by the action of rivers. This tends to sort the detritus according to size and, during transport, the larger fragments become reduced in size and rounder in shape. Such material may accumulate to form river flood plain or beach deposits of boulders, gravels, sands and silts (Reading 1978).

Owing to their small size, clay particles are liable to be deposited only under conditions of relatively still water in lakes, lagoons, estuaries and the sea.
However, they may also form deposits in temporary lakes formed within river valleys. Similarly, peat and organic soil deposits may also occur in this situation, as well as in poorly drained lowland areas. In upland areas the principal deposits include course grained clastic detritus that mantles the land surface and accumulates in valleys, and organic soils and peat which is preserved due to the retardation of bacterial action in cool wet climates.

Some typical values of important engineering properties of temperate deposits are presented in Table 6. The purpose of this brief review of temperate deposits is to discuss the variation in these engineering parameters with particular reference to the composition and mode of formation of the materials concerned.

BOULDERS, COBBLES AND GRAVELS

Gravels are defined as uncemented materials consisting predominantly of particles between 2 mm and 60 mm in size (Anon. 1981). Coarser grained deposits than this consist of cobbles with particles between 60 mm and 200 mm, and boulders in which the particle size is over 200 mm. These coarse grained deposits vary greatly in character and properties depending on the size and size distribution of the grains and the structure. In turn, these are influenced by the source rocks, topographic relief, climatic conditions and other factors including the conditions of erosion, transport and deposition. Compared with gravels deposited in upland areas, those formed in areas of low relief such as river flood plains and alluvial terraces tend to consist of smaller and rounder fragments. In addition, the latter are often associated with sand which occurs either as a matrix or as interbed material. The particles tend to be composed of the more resistant rocks such as quartzite, vein quartz, flint and chert. On the other hand, upland gravels and screes usually consist of larger, more angular, generally poorly sorted fragments of rocks having low resistance to breakdown. The sizes of these fragments are highly dependent on the spacing of discontinuities within the parental rockmass. Beach gravels are usually uniformly graded deposits with rounded particles. Many raised beaches consist of bouldery deposits with sand in the upper parts and successively finer material in a seaward direction.

Medium dense and denser gravels usually provide high bearing capacity and low compressibility. Settlement of these deposits is attributed to the rearrangement of particles by relative motion possibly accompanied by some distortion of the particles. Sliding may occur at all stress levels, but will decrease in significance with increasing density index. However, for major bending and fracturing of particles to occur stress levels need to be higher than approximately 3.5 MPa. This critical stress is lowest for loosely packed, uniformly graded deposits consisting of angular fragments of low strength and highest in well graded dense gravels which have a sandy matrix.

Tomlinson (1982) noted that difficulties can arise with the driving of wooden or concrete piles in gravels and consequently steel piles are usually preferred. Damp sandy gravels above the water table usually possess sufficient cohesion to stand vertically for limited periods of time. Loose, non-sandy gravels will not do so, they slump back to an angle of repose of 30 to 35°. Owing to high permeability, excavations, particularly those in river flood plains which have a high ground water table, are liable to require heavy pumping. Sandy gravels may be drained by pumping from excavations although internal erosion of the fines may cause a reduction in strength of the material.

It can be very difficult to obtain satisfactory samples of gravels for laboratory tests. Large diameter, thick wall open ended samplers provide material

<table>
<thead>
<tr>
<th></th>
<th>GRAVEL</th>
<th>SAND</th>
<th>SILT</th>
<th>CLAY</th>
<th>PEAT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density Mg/m³</td>
<td>1.45-2.30</td>
<td>1.40-2.15</td>
<td>1.82-2.15</td>
<td>1.50-2.15</td>
<td>0.60-1.20</td>
</tr>
<tr>
<td>Dry density Mg/m³</td>
<td>1.40-2.10</td>
<td>1.35-2.15</td>
<td>1.45-1.95</td>
<td>1.20-1.75</td>
<td>0.07-0.11</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.25-1.00</td>
<td>0.30-0.54</td>
<td>0.35-0.85</td>
<td>0.42-0.96</td>
<td>0.90-25.00</td>
</tr>
<tr>
<td>Liquid limit %</td>
<td>-</td>
<td>24 - 35</td>
<td>&gt;50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Plastic limit %</td>
<td>-</td>
<td>14 - 25</td>
<td>&gt;20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Shear strength kPa</td>
<td>c=200-600</td>
<td>c=100-400</td>
<td>c'w75</td>
<td>c'=20-200</td>
<td>c'&lt;20</td>
</tr>
<tr>
<td>Friction angle °</td>
<td>ϕ = 35-40</td>
<td>ϕ = 32-42</td>
<td>ϕ' = 32-36</td>
<td>ϕ'w18-25</td>
<td>ϕ' = 5</td>
</tr>
</tbody>
</table>
Table 7 Angle of shearing resistance for gravels, sands and silts (after Terzaghi and Peck 1967)

<table>
<thead>
<tr>
<th>SOIL</th>
<th>ANGLE OF SHEARING RESISTANCE °</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loose</td>
</tr>
<tr>
<td>Sandy gravel</td>
<td>35</td>
</tr>
<tr>
<td>Sand (well graded with angular grains)</td>
<td>33</td>
</tr>
<tr>
<td>Sand (uniformly graded with rounded grains)</td>
<td>27.5</td>
</tr>
<tr>
<td>Silty sand</td>
<td>27-33</td>
</tr>
<tr>
<td>Silt (inorganic)</td>
<td>27-30</td>
</tr>
</tbody>
</table>

suitable for carrying out particle size analyses, and for the description of the particle shape and surface character. If the deposit contains a matrix of cohesive material and the stones are small, then the samples may be suitable for strength and compressibility tests. Generally, bearing capacity and compressibility calculations are based on the results of in situ cone standard penetration tests rather than rely on laboratory testing (Bell et al. 1990).

River flood plain and terrace gravels provide a very valuable resource of aggregate for concrete making and other purposes. For the most part the deposits require little pre-treatment apart from grading, washing to remove clay and organic material and the crushing of over-sized fragments. Some residual and upland valley gravel deposits are also exploited but these are more likely to be coarser grained, less well sorted and contain more undesirable, weaker components.

SANDS

Most sands consist of an assemblage of one or two types of minerals with or without some rock fragments. These tend to be the more chemically stable varieties and the main constituent of most sands is quartz. The grain size of sands varies between 60 microns and 2 mm. The majority of sandy deposits are found in river flood plains and terraces where they are liable to be associated with gravels and silts. Those which occur in estuaries are usually interbedded with silts, clays and organic deposits.

Due to a wider range of particle sizes in the water lain deposits, these tend to have higher density, greater strength, and lower compressibility than aeolian sands in which the particles typically range between 0.1 and 0.15 mm in size. Many raised beaches consist of mixtures of sands and gravels, with sands becoming more dominant both upwards and seawards.

As indicated in Table 6, the strength of these deposits is of a purely frictional nature. Cornforth (1964) concluded that the shear strength of sand is primarily a function of the strain conditions imposed on the material. Because the causative external stress conditions control the sliding action between grains they also influence the amount of intergranular stress. Hence the packing is also important. In densely packed sands, due to interlocking of grains, intergranular motion mobilizes shear stress and produces significant dilation. Such dilation can result in the development of negative pore water pressures.

Interlocking of grains does not contribute to the shear strength of loose sands, which, during shear, usually experience relatively minor consolidation and then dilation. In consequence of this, the shear strength of a dense sand is much greater than that of a loose sand. Once failed, further strain causes the strength of dense sand to drop to a residual value approximately equal to the shear resistance mobilized by the sand in a loose condition. The value of this depends on the applied stress rather than the initial density of the sand. Naturally, the grain shape and surface roughness of grains also exert a control over the friction developed between grains.

Terzaghi and Peck (1967) indicated considerable variation in the angle of shearing resistance as shown in Table 7. At high values of confining stress, the crushing of grains at points of contact tends to remove asperities which contribute to the interlocking action experienced at lower stress levels. The angle of internal friction then drops.

The compression of sands under load is dependent on their density such that loosely packed varieties above the water table are liable to be much more compressible than saturated dense sands. This is because in dense sands there is less opportunity for
grain movement under the influence of an imposed load. Such motion occurs at all stress levels but for major bending or fracturing of grains to occur the stress needs to exceed a certain critical value. This critical stress is lowest for uniformly graded, loosely packed deposits consisting of angular particles of low strength. In turn, the packing is controlled by the particle size distribution and the geological loading history.

Generally, well sorted sands, such as aeolian deposits, have lower densities than poorly sorted water lain ones. Since the shearing action is distributed over a wider zone in coarser grained deposits, they tend to be stronger than fine grained ones. Holtz and Gibbs (1956) noted that in sand-gravel mixtures a considerable increase in strength accompanies an increase in the amount of gravel to 50 or 60%. The presence of relatively minor quantities of intergranular cement also exerts a control over engineering behaviour. In deposits having variable packing, differential settlement is liable to occur.

Provided that the mass drainage conditions of the soil allow the immediate equalization of pore fluid pressures and suction, the presence of pore water has only a negligible effect on the shear behaviour of medium and coarse grained sands. Consolidation occurs rapidly unless stress values are sufficiently high to cause grain fracturing. As with silts, under some circumstances saturated fine sands can be subject to liquefaction due to the presence of high seepage pressures. Due to this possibility, in order to excavate fine grained sands below the water table it may be necessary either to install well points to reduce water pressures or stabilize the material by the injection of grout. Liquefaction can also result from rapid or cyclical loading.

Samples of sands can be obtained by the use of a thin walled piston sampler. However, due to friction between the tube and the sample and the effect of changes in the in situ stress conditions, usually it is not possible to obtain samples in an undisturbed condition. In most cases bearing capacity and settlement calculations are based on the results of either laboratory tests on recompacted disturbed samples or in situ standard penetration tests (Bell et al. 1990). Clayton et al. (1982) described various sophisticated sampling methods but pointed out that since these are expensive and difficult to perform, they are used only in studies in which the structure of the sediment is particularly important, for example in relation to liquefaction potential.

Deposits of sand, particularly fluvial ones which tend to be well sorted, are extensively exploited as a source of fine aggregate for concrete and mortar making and many other purposes. Residual and upland valley deposits tend to be less well sorted and they are also liable to contain a significant proportion of over-sized fragments. Marine and estuarine sands are often associated with silty and clayey material which must be removed by washing.

Some sands display particularly high degrees of packing. To distinguish them from cemented deposits such materials were described by Dusseault and Morgenstern (1979) as locked sands. They have much higher shearing resistance and lower porosity than dense sands. Locked sands are also relatively incompressible. Diagenetic processes involving the dissolution of quartz at grain contacts and its precipitation within the pore space has given rise to a deposit in which the density index exceeds 100%, that is, they are more dense in their natural state than may be achieved by the laboratory methods for obtaining minimum porosity.

**SILTS**

Silts are deposits in which the predominant grain size is in the range 2 to 60 microns. They are formed by the mechanical breakdown of rocks. The most common constituent is quartz, although some varieties also contain a proportion of aggregated clay particles and varying proportions of rarer but chemically stable minerals. Although silt deposits may occur in residual soil horizons, by far their most usual mode of occurrence is within alluvial flood plain deposits, usually in association with beds of sand and clay. Silts also occur in similar associations in estuarine and deltaic deposits whereas, lacustrine and marine silts tend to be interbedded with clays.

Loess is an aeolian silt deposit formed under either dry arid or cold conditions. As explained below, these deposits tend to exist in a metastable condition in that they may collapse when wetted. Some deposits, for example the brickearths of southern England are similar to loess. Brickearths have gradings equivalent to coarse silts with in situ dry densities ranging between 1.48 and 1.73 Mg/m³ and porosities of 36 to 45%. Derbyshire and Mellors (1988) explained that although these materials are more compact than loess, they are marginally stable with respect to soil collapse.

The engineering properties of silts depend on their grain size, grain shape, moisture content, particle size distribution and density. Other factors that influence their properties include pore water...
chemistry and the stress conditions. The strength increases and compressibility decreases as the grain size and range of sizes of grains increase. Under certain conditions both fine sands and silts become dilatant during shearing. This causes a reduction in pore water pressure. In British silts it has been found that dilatancy occurs at water contents of between 16 and 35%. Penman (1953) showed that the magnitude of this volume change increased with the density of the material. The shear strength of the material is a function of the force required to cause dilatancy against the applied pressures. Grain interlocking, and the friction mobilized between grains, contributes to the strength. Thus the strength increases as the density of the deposit becomes greater.

Work on Rhine valley silts by Schultz and Kotzias (1961) indicated that the consolidation of these deposits is influenced by the grain size, particularly the clay size fraction, porosity and moisture content. The silts tested underwent significant secondary consolidation, amounting to 24% of total volume reduction. These authors concluded that triaxial testing provides a better indication of shear strength than direct shear tests. At low confining pressures silts generally possess both cohesive and frictional strength but, above a critical pressure, the value of which depends on the density, they behave in a purely cohesive manner. In other words, the angle of internal friction ($\phi$) was zero.

The fact that many silts are interbedded with clays renders the deposit anisotropic with horizontal permeability higher by several orders of magnitude than vertical permeability. Anisotropy is also displayed by other geotechnical parameters. It is often difficult to obtain undisturbed and undisturbed samples of silts from boreholes. However, some silts can be satisfactorily sampled using thin-walled open tubes with an inside clearance. Clayton et al. (1982) described the use of fixed piston sampling devices which are more likely to be suitable for obtaining undisturbed samples in silts with an undrained shear strength of more than 5 kPa.

In situations in which silts are interbedded with sands, it is often preferable to obtain design parameters by the use of in situ testing, for example static cone penetration or shear vane tests. Difficulties are liable to arise with the stability of excavations below the water table. High seepage pressures can result in the liquefaction of deposits. Such a problem can be dealt with by lowering the ground water pressures by installing well points, or alternatively the material can be stabilized by the injection of grout. Liquefaction of saturated silt is also a likely outcome of sudden loading or of vibratory loadings, such as may occur due to earthquakes or the operation of machinery.

CLAYS

Clays formed under temperate climatic conditions are produced by chemical weathering of silicate minerals and by the breakdown of existing clay deposits. Clays are sedimented from still or very slow moving water conditions in lacustrine, estuarine and marine situations. They also occur in residual soil profiles, where they may be concentrated by eluvial processes, as a residual product of the weathering of mudrocks, in fluvial systems as over bank deposits and in abandoned meanders. The Clay-with-flints head which blankets much of the Chalk outcrop of the south of England has been derived either as a product of Tertiary mudrocks that formerly overlay the Chalk or as an insoluble residue resulting from the weathering of chalk (Catt 1988).

The engineering behaviour of clays is dependent on their composition, grain size, water content and micro-structure. Owing to their small particle size, inter-particle forces in clays are dominated by electrical and Van der Waals forces. These are affected by charges carried by the particles and the cationic concentration of the pore water. A basic distinction needs to be drawn between marine and fresh water clays. As described by Van Olphen (1963), where the cation concentration of the water in which the particles are suspended is high, they become flocculated into various configurations. When freshly deposited, such clays have low density. In contrast, clays deposited from fresh water form a more dispersed structure, with a void ratio as low as 0.5; they are dense, relatively impermeable materials. Estuarine clays may display a mixture of these micro-structural types.

Once deposited the structure of marine clays, in particular, may be modified by increases in overburden pressure. Immediately after deposition the void ratio of the deposit is likely to be as high as two and the water content will be very high (Bjerrum 1967). Increases in the overburden pressure reduces both the pore volume and the water content by causing the breakage of flocculation bonds and particle rearrangement. If during consolidation, the cationic concentration of the pore water remains high, the particles are liable to remain randomly orientated. On the other hand, dispersive conditions created by the presence of fresh pore water causes the particles to become preferentially...
orriented in a horizontal direction. Obviously this imparts anisotropy on the material. In some marine clays, a lack of consolidation after deposition, coupled with leaching of the pore water to reduce its cation concentration, has resulted in deposits which display significant metastability (Gillott 1979).

Clay deposits which are buried and subsequently exhumed are liable to be found in an overconsolidated condition. This contrasts with normally consolidated clays which have not borne an overburden pressure greater than the present one. Due to clay mineral bonding and the precipitation of intergranular cements during diagenesis, overconsolidated deposits are more dense, stiffer and less compressible than their normally consolidated counterparts. These changes are accompanied by other modifications to the engineering properties.

Most recently deposited clays are in a normally consolidated condition although some have become overconsolidated due to desiccation after deposition. This is because the removal of water produces surface tension forces within the inter-particle capillaries. Such negative pore water or suction pressures draw the soil particles into closer proximity to each other and thus increase the density of the material. Even in the wetter temperate areas, a desiccated crust one or more metres thick may overlie weaker and more compressible normally consolidated clay.

An overconsolidated clay is considerably stronger than a normally consolidated one at the same pressure. In both cases a peak shear strength, higher in the case of an overconsolidated clay, is mobilized at low values of strain and the strength gradually decreases to a residual value with further strain. The latter value of strength depends primarily on the grain size and mineralogy of the deposit; it is not strongly influenced by the consolidation history. It has been found (Skempton 1964) that the residual shear strength is lowest in clay-rich deposits and increases as the proportion of detrital or diagenetic minerals increases. During shearing overconsolidated clays are dilatant so that under undrained conditions, pore water suction are produced. Thus, the undrained shear strength is greater than the value determined under drained conditions. This behaviour contrasts with that displayed by a normally consolidated clay in which, due to consolidation during shear, the shear strength obtained by testing under drained conditions exceeds that for undrained conditions.

When tested in an undisturbed condition the shear strength of most normally consolidated clays is higher than the value obtained for the material tested in a remoulded state at the same water content and stress conditions. Destruction of the framework of clay minerals and of bonds at the points of contact during remoulding or shearing renders the material weaker. Such behaviour was described by Skempton and Northey (1953). It is a feature of marine clays, which due to edge to face floculation structures, have low density and high values of water content. Quick-clays display extreme sensitivity (Gillott 1979) (see above). Some types are capable of flow if subjected to only slight disturbance. On the other hand heavily overconsolidated clays are insensitive. Some clays with moderate to high sensitivity display thixotropy in that they recover some strength after remoulding.

Reduction of the overburden pressure borne by a clay formation by erosion or due to excavation, is accompanied by elastic rebound. However, overconsolidated clays do not necessarily realize their total heave potential nor lose their greater strength and lower compressibility immediately. As explained by Bjerrum (1967), these changes depend on the strength of diagenetic bonding imparted as a result of overconsolidation. Elastic rebound will accompany the relief of overburden pressures in materials in which the bonds are weak, whereas, if strong bonds are present, this process may continue over long time scales and give rise to time dependent heaving. The strength, compressibility and other engineering properties of the material slowly change as this process takes place. Thus, steep-sided excavations in overconsolidated clays may remain stable for some time following construction, but then eventually undergo rotational failure.

As explained below, changes to the pore water pressure conditions also affect the long term strength characteristics of clays. Excessively steep slopes in normally consolidated clays are liable to fail by slumping or rotational failure. In soft clays basal heave may be a problem in excavations.

Expansion due to the removal of overburden gives rise to the development of systems of fissures which significantly modify the mass properties of overconsolidated clays. As explained by Taylor and Cripps (1984), the behaviour of small laboratory specimens can be significantly different from that of the material en masse. In addition, obtaining suitable samples of fissured deposits can be very difficult. Softening of the clay adjacent to fissures causes a reduction in shear strength to a value approaching the residual value. Fissures also cause the concentration of shear stresses which may
locally exceed the peak strength of the clay so leading to progressive failure. Under these circumstances the operational strength of a clay deposit lies between the peak and the residual shear strengths.

The engineering properties of clay deposits are dependent both on their total water content and on the situation of the water. Water can be held in both inter-particle and intra-particle spaces. Removal of this water due to drying or drainage leads to shrinkage of the material. In relatively dry clays water is held in the inter-particle voids by capillary forces which are relaxed as the clay is wetted. As a result the clay expands and, due to the reduction in inter-particle force, its strength is reduced.

Intra-particle swelling is a characteristic of smectitic clays. Such components are liable to be present in some clays derived from volcanic terrains or mudrocks containing these minerals. In swelling clays, water is incorporated into the crystal lattice so that as the water content rises the actual particle expands. The process is reversible so that dehydration due to evaporation or transpiration leads to volume reduction. The magnitude of these volume changes can be very large.

Besides the availability of water and the type of dissolved cations it contains, the amount of swelling and shrinkage also depends on the inter-layer cations present in the clay mineral and the overburden pressure. The swell potential of all clays can be increased due to the action of osmotic pressures produced by infiltration of rain water into clays in which the pore waters contain a high salt concentration. Swelling is also greater in clays in which the particles display particle orientation.

Moisture migration and pressure changes caused by freeze-thaw action also can increase the swell potential. Significant seasonal changes in soil moisture are liable to occur in clay formations in southern England to depths of 1.0 to 1.5 m. The presence of trees can increase this depth (Driscoll 1984) and it is also greater in the drier temperate climates. The clearance of vegetation can lead to significant swelling, particularly in more arid areas.

Successive reductions in water content to the shrinkage limit cause volume decrease, but once this limit has been attained no further shrinkage occurs. In soils with a high clay content, the water content at the wilting point for plants is greater than the shrinkage limit whereas in low clay content soils this water content is liable to be lower. Hence the desiccating effect of vegetation is likely to be more notable in low to moderately expansive clays than for highly expansive ones.

Plasticity is an important fundamental property of clays. The extent to which they display plasticity depends on their mineralogy, water content, pore water composition and the crystallinity of the constituent clay particles. Increases in water content and decreases in crystallinity both tend to increase the plasticity of a clay. Clays containing smectite are generally more plastic than ones containing illite or kaolinite. As little as 10% of smectite may make a significant difference.

In the case of smectitic clays the type of interlayer cation present in the crystal lattice is a further cause of variation in the plasticity. In terms of liquid limit, the plasticities of kaolinitic and illitic clays are liable to lie in the ranges of 30 to 75% and 60 to 90% respectively, whilst the value for smectitic clay exceeds 100% and may reach very high values. Since the plasticity of clays depends on mineralogy, expansive clays can be recognized from their plasticity characteristics (Holtz and Gibbs 1956).

Skempton (1953) defined activity as the ratio of the plasticity index to the percentage clay fraction (that is, particles less than 2 microns in size) where the plasticity index is the difference between the liquid and plastic limits of the material. Clays with normal activity have values in the range 0.75 to 1.25, while for active clays with high swell potential, the activity is higher than 1.25 but it is lower than 0.75 in inactive clays.

The activity of many British soils lies within the normal range, although lacustrine deposits tend to be inactive, estuarine and marine clays are liable to be active. Data quoted by Skempton (1953) indicated that, due to a swelling clay component, London Clay is an active clay deposit.

The application of a load to a clay soil causes it to compress by the reduction of the void ratio. In the case of saturated soils, since initially the load is carried by the water, a pore water pressure in excess of the hydrostatic stress develops. This excess pressure is dissipated at a rate which depends on the permeability of the soil. In the case of clay deposits, owing to their low permeability, this is a slow process. The rate of consolidation can be significantly increased if the permeability of the deposit is enhanced by the presence of even quite minor silty seams, as for example in varved clays. During dissipation, neglecting any small reduction in volume of the soil particles or pore fluid, the volume of water expelled from the pore space is equal to the compression of the deposit. Reduction in volume caused by the exclusion of pore water is termed
primary consolidation.

Due to their less dense structure marine clays tend to be more compressible than fresh water ones. Some clays, particularly those containing organic matter, or if they are interbedded with organic rich layers, undergo significant secondary consolidation due mainly to the rearrangement and distortion of soil particles. Both primary and secondary consolidation occur concurrently. Lightly overconsolidated clays, in particular, are liable to undergo time dependent secondary consolidation.

Since in a saturated clay an increase in load is initially borne by the pore water, the loading does not result in an immediate increase in strength of the material. As a result, in undrained shear, clays behave in a purely cohesive manner so that the angle of internal friction is zero. Once dissipation of the excess hydrostatic pressure has occurred, for the drained or effective stress parameters, the material displays both cohesion and friction. This means that the shear strength parameters of the material depend on the drainage conditions. Temporary structures, such as trenches, and poorly designed permanent structures may be stable immediately following construction but become unstable as the pore water pressures equilibrate with hydrostatic values.

The bearing capacity of a soil depends mainly on its shear strength, the depth and the ground water conditions (Tomlinson 1982). Normally consolidated clays can sustain a shear stress greater than the in situ value but less than a critical value without undergoing excessive strain. The allowable shear stress decreases with increases in plasticity and strain rate. Marine clays tend to have low density but relatively high strength. The bearing capacity of normally consolidated clays is low near to the ground surface where they tend to be soft. However, due to increases in the in situ stress with depth, an enhancement of a strength value occurs so that they can become stiff at depth.

In cases in which normally consolidated clays have a crust of over-consolidated clay, foundations may be satisfactorily supported on this upper layer provided that the underlying softer material is not over stressed. Provided the water content of the deposit does not change, the ultimate bearing capacity of a clay is a function of its undrained shear strength. Any reduction in pore space due to consolidation increases the factor of safety against failure of the foundation. Most overconsolidated clays have high bearing capacity. Although a bearing capacity calculation may indicate that the support for a particular structure is adequate, in clays an excessive amount of differential or total settlement may still occur. Therefore, it is necessary to consider the settlements expected from a particular foundation design. Clays which have undergone swelling or heave, as described above, are liable to be subject to increased settlement on loading.

Generally, unless they contain large stones, stiff overconsolidated clays can be sampled using standard U100 sample tubes which are driven into the ground. Deposits which, because they are too hard, become disturbed during driving may be sampled by means of triple core barrel rotary drilling methods (Clayton et al. 1982). Softer clays, particularly those interbedded with silts and varved clays, are difficult to sample in an undisturbed condition. Various types of lined thin-walled piston samplers are available. Very long continuous samples have been obtained by the use of sampling devices that incorporate a foil or stockinette sheath which is stretched over the sample during the sampling operation.

ORGANIC SOILS AND PEAT

Peat is an accumulation of partially decomposed plant material in which complete oxidation has been prevented by incomplete aeration and high water content. Most peats which occur in temperate zones have been formed during the last 20,000 years. In addition, some buried peats were formed during interglacial periods. Peat has formed in both highland and lowland situations, producing upland bogs on the one hand and fens on the other. In the latter case, and also in estuarine and lacustrine situations, peat is liable to be interbedded with deposits of clay, organic clay, silt and, more rarely, sands. Catt (1988) reviewed the classification of peat. The degree of humification of the original vegetable matter can be assessed by squeezing the water out of a handful of the peat and observing both its amount and colour. In amorphous granular peat the original plant structures are completely destroyed; the material can be squeezed between the fingers and there is no free water. Peat in which breakdown of the vegetable matter is incomplete contains varying amounts of recognizable plant material with some brown-coloured free water. In addition to organic matter most peats also contain a percentage of mineral material. In highly organic peats such material may constitute as little as 2% of the dry weight. Increases in the non-organic constituents make the material more soil-like and a material in which the organic constituents exceed 50% would be considered to be an organic soil rather than
a peat.

As indicated in Table 6, depending on type, peats have very high values of void ratio and water content. Removal of the water from peat causes shrinkage with a minimum volume achieved at the point of almost complete dehydration. The amount of shrinkage can vary between 10 and 75% of the original volume. The dry density of peat is a useful indicator of the behaviour of the material under load. Hanrahan (1952) recorded values in the range 65 to 120 kg/m³. Higher values than these are liable to be found in peats containing high quantities of non-organic material.

The shear strength of peat increases with the degree of humification, the amount of mineral matter and reduction of water content. During drainage, initial decreases in the water content have little effect on the strength but at the later stages removal of the same volume of water has a large effect. Loading of peat causes high pore water pressures to be generated. Due to high initial permeability, primary consolidation proceeds rapidly as these pressures are dissipated. Significant secondary consolidation relating to the removal of water from micropores and creep deformations result in erratic increases in compression for constant loading for protracted time intervals. In addition, loading has been found to cause a large reduction in the value of the permeability with time (Adams 1965). Thus it is difficult to predict the settlement characteristics of this material. The removal of load does not usually result in a recovery of the compression. The installation of drainage does not increase the rate at which full consolidation is achieved since it only assists primary consolidation which is a rapid process in any case.

The permeability of peat varies over a wide range. Most deposits display significant anisotropy, with horizontal values exceeding vertical ones by at least one order of magnitude. The bearing capacity of peat is low with very large total and differential settlements commonplace. Organic soils are also weak, highly compressible materials. If the organic content of a soil rises above 20% of the weight, the consolidation behaviour is increasingly dominated by the organic material.

Peats and organic soils are liable to undergo rotational failure in slopes. A low submerged density renders peat prone to spreading failure under the action of seepage forces. Drainage of peat gives rise to long term volume reduction due to increases in effective stress, the removal of water and oxidation of organic matter.

Due to the weak and delicate structure of peats and organic soils, they are difficult to sample in an undisturbed condition. Thin-wall fixed piston sampling as described for soft clays is likely to be the most successful method.

ARID DEPOSITS

Most arid deposits consist of the products of the physical weathering of bedrock formations. The process gives rise to a variety of rock and mineral fragments which then may be transported and deposited under the influence of gravity, wind or water. Arid conditions may give rise to a variety of evaporative deposits and cemented layers due to the precipitation of soluble minerals. Although biological activity tends to be limited within arid zones, it can be a significant agency of deposition or alteration, for example in the formation of algal mats in coastal lagoons. The activities of burrowing animals, such as termites, and the growth of roots also may modify the engineering behaviour of soils.

DISTRIBUTION OF ARID DEPOSITS

Arid environments may be defined as those in which net evapo-transpiration exceeds total precipitation. Such conditions exist extensively, though not exclusively, within belts at latitudes approximately 15° to 45° north and south of the equator. Arid and extremely arid zones encompass much of north Africa, the Middle East, the western United States, parts of South Africa and much of non-coastal Australia. Due to controls exerted by altitude and the distribution of rain-bearing winds, some high mountainous areas in central Asia, equatorial parts of both west South America and much of East Africa also experience such conditions.

Although some regions of Europe, the eastern United States, central America, China and the eastern half of the Indian sub-continent lie within the latitudes in which arid zones commonly occur, total rainfall in these areas is usually sufficient to overcome evapo-transpiration losses.

Most of the arid areas of the world are characterized by high annual mean temperatures, perhaps in excess of 30°C, coupled with an annual precipitation totalling less than 800 mm. The rainfall tends to be seasonal and highly sporadic, both in the short- and long-term. In extremely arid areas the total precipitation may be considerably less than 250 mm per annum. These conditions give rise to a lack of surface water and run-off. Areas
deprived of water due to cold conditions at high latitudes or great altitudes are considered within the context of glacial environments (above).

Collinson (1978) indicated that various landforms typify arid regions. These include bare rocky outcrops in upland and mountainous areas, large alluvial plains, some with internal drainage basins, salt flats, dry deltas and areas of sandy and stony desert. In coastal areas, sabkha and lagoonal environments of low clastic input give rise to sand grade, bioclastic carbonates and finer grained sediments, bioturbated and laminated clays and algal mat or stromatolite deposits with evaporative minerals.

ENGINEERING CHARACTERISTICS

A number of engineering problems arise with Quaternary arid deposits. Some of these are common features of the materials themselves and others occur due to the climatic conditions.

Settlement may occur due to the withdrawal of groundwater in some arid areas, especially those underlain by confined aquifers. The reduction in pore water pressure leads to an increase in effective vertical stress and results in the consolidation of sediments. Poland and Davis (1969) reported that abstraction of water for irrigation in the San Joaquin Valley, California caused extensive settlement in that area.

Gravels and sands

The shortage of surface water, together with the harsh conditions, inhibit biological and chemical activity such that in most arid zones there is only a sparse growth, or perhaps absence, of vegetation. This makes the surface regolith of upland areas and slopes highly vulnerable to intense denudation and redistribution by gravitational or aeolian processes or the action of ephemeral water. Weathering activity tends to be dominated by the physical breakdown, in places intense, of minerals, rocks and rockmasses into poorly sorted assemblages of fragments ranging in size down to silts.

The rapid erosion of loose surface material by hill-wash or ephemeral stream and river channels during rainfall events often leads to debris flows, especially in alluvial fans. 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. Much of the gravel-sized material is liable to consist of relatively weak, low durability materials.

Many arid areas are dominated by the presence of large masses of sand, usually derived from coasts, permanent or temporary streams and by direct attrition of rocks, particularly sandstones. Depending on the rate of supply of sand, the wind speed, direction, frequency and constancy and the nature of the ground surface, it may be transported and/or deposited in mobile or static dunes. Cooke et al. (1978) illustrated the analysis of a site of a proposed airport site in Dubai with respect to the threat posed by dune movement.

For the most part, aeolian sands are poorly graded, have undesirably smooth surfaces and may be too impure for use as fine aggregate. Fookes and Higginbottom (1980) explained that as far as the availability of sand and gravel aggregates are concerned, in the absence of downward leaching, surface deposits become contaminated with precipitated salts, particularly sulphates and chlorides. However, active wadi channels in areas of high relief may yield acceptable uncontaminated aggregates. Alluvial plain deposits tend to be less suitable due to the presence of gypsum particles and cement and also of fragments of weak, weathered rock and clay. Beach sands are used as a source of fine aggregate in various parts of the Middle East. Most of these materials are carbonate sands with smooth, well-rounded, porous, chemically unstable particles. Again, contamination with evaporative salt can cause problems.

Low lying coastal zones and inland plains with a high water table are areas in which sabkha conditions commonly develop. These are extensive flat areas which, in the coastal setting, tend to slope gently seawards. Groundwater is recharged directly from the sea, from inland sources or by infiltration of sea water blown inland by on-shore winds. The ground surface is held at a particular level by capillary soil moisture. The height to which water may rise from the water table is a function of the size and continuity of the pore spaces in the soil. Lane and Washburn (1946) demonstrated that capillary rises of up to 3 m can occur in some fine grained deposits. Soil above this level is subject to aeolian erosion whereas the deposition of wind-borne sand or silt and evaporative minerals occurs below this level. Within coastal sabkas the dominant minerals are calcite, dolomite and gypsum with lesser amounts of anhydrite, magnesite, halite and carnalite together with various other sulphates and chlorides. In the case of inland sabkas or salinas, the minerals precipitated are much more variable since they depend on the composition of the local...
groundwater. The same applies to inland drainage basins and salt playas that are subject to periodic desiccation (Fookes and Collis 1975).

Ellis (1973) considered the engineering properties of sabkha materials. He concluded that although standard tests for assessing material suitability often gave misleading results, the deposits can be satisfactorily used as sub-base and base material for surfaced and unsurfaced roads. In the case of unsurfaced roads, the hydroscopic properties of salt assist with the binding of the running surface so a proportion of this mineral is advantageous. The surfacing material of surfaced roads can be subject to disruption if the construction material becomes contaminated with soluble salts, so the base of the bitumen should be raised well above the zone of capillary moisture. Sabkha materials have very variable, but for the most part low, bearing capacities.

Silty deposits and loess

Silt-sized material may be deposited in temporary lakes or by aeolian action. Lacustrine and fluvial silts that occur within alluvial plains and other areas often bear many similarities with other water-borne silts except that they may be interbedded with evaporite deposits and have been affected by periodic desiccation. The latter process leads to the development of a stiffened crust or, where this has occurred successively, a series of hardened layers within the formation. An aeolian silty deposit that occurs extensively in arid areas, or areas which were arid during Quaternary times. Significant deposits occur in the dry continental areas of central and western Europe, North America, Russia and Asia. Great thicknesses occur in some parts of China, with reported depths exceeding 300 m in Gansu Province.

Most loess comprises a well-sorted assemblage of silt-sized grains (50 to 90%) with some sand and clay mineral winnowed either from fine grained glacial outwash deposits or desert basins, wadis and playas. It consists predominantly of quartz grains with lesser amounts of feldspar, clay minerals and calcite. Work by Grabowska-Olszewska (1988) indicated that in Poland the clay, which was derived either by the in situ weathering of feldspars or transportation in silt-sized aggregations, includes a high percentage of mixed-layer clays (50-80%) with some illite (5-40%), kaolinite (2-5%) and fine quartz (2-10%). Most deposits contain between 4% and 20% calcium carbonate that occurs at grain contacts or as the linings of fossil root holes.

Following a study of loess from various parts of China, Derbyshire and Mellors (1988) reported that the samples they studied had similar composition, grain size distribution, open texture and low degree of saturation. In addition, the material was loosely bonded and was easily disaggregated by water action.

Typically, the evenly distributed inter-granular pore space is only partially filled with clay sized quartz, clay minerals and calcite. The mechanical behaviour of loess is heavily dependent on the amount and bonding properties of this inter-granular material. Apart from some apparently alluvially derived deposits, most loess lacks any stratification. However, the presence of systems of fossil root holes are a characteristic occurrence in many deposits. Due to these features, unlike fluvial and lacustrine silts, the vertical permeability of loess is much greater than the value in a horizontal direction.

The bulk and dry densities of loess are invariably low with values typically in the ranges of 2.65 to 2.70 Mg/m³ and 1.20 to 1.46 Mg/m³ respectively. The corresponding void ratio and porosity values are respectively 0.81 to 0.89 and 45 to 47% for Chinese loess. Loess tends to display moderate plasticity with liquid limit values in the range between 25 and 35%, exceptionally values of 45% have been observed in clayey loess.

The behaviour of loess is very sensitive to change in the water content and, in the case of shear strength, the value is also dependent on the initial porosity, the degree of deterioration of inter-particle bonds and the increase in grain contacts brought about by consolidation. The response to applied stress is complicated since, at low levels, load is borne by the bonds, the breakage of which leads to a reduction in the cohesion of the material. With further loading the grains are brought into direct contact with one other, thereby increasing the inter-particle friction and producing a hardening effect. Tan (1988) noted that in triaxial tests the style of failure observed changed from a brittle to a plastic one at a confining pressure of about 100 kPa. In consequence of this, loess can support heavy structures with small settlements provided the water content is low and the imposed stress does not exceed the apparent pre-consolidation pressure. Considerable compression occurs for higher stresses with primary and secondary consolidation behaviour similar to that of saturated clay.

A number of silty deposits, including loess, formed under arid conditions are liable to undergo considerable volume reduction or collapse when wetted. Such meta-stability arises due to the loss of...
strength of inter-particle bonds resulting from increases in water content. Thus, infiltration of surface water, including that applied in the course of irrigation, leakage from pipes and rise of water table may cause large settlements to occur. Popescu (1986) defined a "truly collapsing" soil as one in which collapse on saturation occurs under self-loading, in other words, at a vertical stress equal to, or less than, the prevailing overburden pressure. On the other hand, a "conditionally collapsing" loess undergoes collapse only if loaded to a greater pressure that this.

A number of authors have suggested criteria to identify soils that are liable to undergo collapse upon saturation. For instance, Gibbs and Bara (1962) suggested that the dry density and liquid limit values can be analysed to identify soils which have sufficient void space to hold their liquid limit moisture content. Such soils are liable to collapse. Experience with Polish loess led Grabowska-Olszewska (1988) to suggest that collapse on saturation is most likely to occur in loess in which the natural water content is less than 6%. The same work indicated that most loess with a water content of more than 19% is liable to be stable.

Lin and Wang (1988) proposed a coefficient of collapsibility \( (i_c) \) which depends on the values of void ratio determined in oedometer tests. This is expressed in terms of the change in void ratio \( (\Delta e) \) on wetting and the initial void ratio \( (e_1) \) as follows:

\[
i_c = \frac{\Delta e}{1 + e_1}
\]  

The Chinese Code on Construction in Regions of Collapsible Loess considers that a collapsible soil is one for which the index is 0.015 or greater. On the other hand, Lutenegger and Hallberg (1988) indicated that, in the United States, soils displaying a value of more than 0.02 are susceptible to collapsing behaviour.

The amount of settlement due to wetting depends on the texture, density and mineralogy of the loess. It also depends on the thickness of the collapsing layer. Irrigation in Tashkent was reported by Dearman et al. (1989) to have caused over 2.5 m of settlement in some places. Construction of the California aqueduct system required pre-treatment by wetting to induce hydrocompaction of the material prior to construction. Other methods of densifying loess before construction include dynamic consolidation, compaction and the injection of grouts.

The subsidence of loess can be accompanied by fissuring at the edge of the subsidence area and over buried cliff features. The availability of water, for instance if irrigation is poorly controlled, can lead to gullying, landslipping and the formation of sinkholes due to internal erosion. This can lead to the rapid undermining of structures and loss of ground support. Poland and Davis (1969) reported that the formation of pipes can be controlled by deep ploughing to mix the soil.

Clays

Various types of calcareous silty clay typically form in arid and sub-arid regions when clay is deposited in saline or lime-rich waters. These are characterized by the presence of a desiccated surface layer typically up to 2 m thick, which may be capable of supporting lightly-loaded structures; although care needs to be exercised to ensure that bearing capacity failure does not occur in the underlying softer soils. Also, the reduction in strength that typically occurs due to wetting should be borne in mind.

Apart from those derived directly by the weathering of mudrocks, clay minerals tend to be a rare constituent of arid deposits. However, aeolian clay dunes are found in situations in which the parent material has been derived as sand or silt-sized aggregations from desiccated alluvial clays. The presence of clay minerals may significantly change the engineering behaviour of some deposits consisting predominantly of other materials. As explained above, loess usually contains a proportion of clay minerals.

Evaporative minerals and carbonates

Recently deposited limestones, of various types, occur in some arid areas, including the Middle East. Most of these consist of poorly cemented, chalky and clayey deposits. Although these materials will usually have adequate bearing capacity for moderate loadings, they suffer high rates of solution and are often too weak for crushing into aggregates.

A common feature of arid deposits 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 respectively unconsolidated deposits or bedrock are cemented by gypsum, calcite, silica, iron oxides or other compounds. Thus, cementation by gypsum would give rise
to gypcrust or gypcrete respectively. Cretes and crusts may form continuous sheet- or isolated patch-like masses at the ground surface where the groundwater table is at, or near, this level, or at some other position within the ground profile. Alternations in the climatic conditions in north Africa and elsewhere during Quaternary times have given rise to an inter-layering of calcrust or caliche and gypcrust deposits, as well as the calcification of gypcrust. In some particularly arid zones dehydration of gypsum has resulted in the formation of anhydrite.

Well cemented crusts and cretes may provide adequate bearing capacity for structures. Care must be taken that the underlying uncemented material will not be overloaded. Also, possible changes in the engineering behaviour of the material with any changes in the water conditions must be borne in mind. Crusts and cretes are often used as construction materials to form crushed rock aggregates for concrete making and road building. However, care needs to be taken to ensure that the material is not contaminated with deleterious soluble salts, particularly chlorides and sulphates (Evans 1978). The presence of a surface cemented layer leads to a lack of infiltration of precipitation so that high rates of run-off may give rise to intense floods and debris flows.

As well as the effect of change in water content on the inter-particle swelling/shrinkage behaviour of arid soils, ground movements are liable to occur as a consequence of the solution and/or precipitation of soluble minerals. Changes in the hydration state of minerals, such as swelling clays and calcium sulphate, also causes significant volume changes. High rates of evaporation in arid areas results in saline groundwater and the precipitation of mineral previously held in solution. Such a process may lead to ground heave due to the precipitation of minerals within the capillary fringe. For instance, Netterburg (1977) documented many cases in which the surfaces of roads in South Africa have been badly disrupted although, in some cases, the problem has been attributed to soluble minerals derived from calcrete used as a construction material.

Minerals that are precipitated from groundwater in arid deposits also have high solution rates so that flowing groundwater may lead quickly 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 which result in the decrease in the salt concentration of groundwaters. Hence, care needs to be exercised with irrigation, bridge, tunnel and harbour works that may lead to the removal of soluble minerals from the ground.

Where evaporation is particularly intense, for example in internal drainage basins, thick sequences of evaporative deposits such as gypsum, anhydrite, halite and potash may be produced. Most deposits formed in this way have very low strengths, also they may be subject to creep or plastic deformation at relatively low stress levels.

**Tropical Soils**

Engineers from temperate climate countries have experienced many difficulties when dealing with certain tropical soils because it has been assumed that they will behave in a similar manner to soils in the temperate zone. Consequently, methods of test and engineering classification schemes have been used which were not designed to cope with the different conditions found when dealing with soils formed in a tropical environment (Gidigasu 1988). Therefore, it is essential to appreciate that new test methods may be needed and that new classification systems must be derived.

Of course, it would be wrong to assume that all tropical soils are different from those found in other climatic zones. Vargas (1985) pointed out that many soils such as alluvial clays and sands or organic clays will behave in the same manner and have similar geotechnical properties regardless of the climatic conditions of the area of deposition. Therefore, in this section, only those soils that are peculiar to tropical regions are considered, the other soils being dealt with elsewhere in this paper. Consequently, most of the soils considered here are residual that is, formed in situ by the weathering of a parent rock (or soil) in tropical climatic conditions.

Vargas (1985) also stated that "what is important to engineers and geotechnologists is the mechanical properties of such soils." Whilst this is undoubtedly true, a full understanding of the mechanical properties and the most appropriate test methodologies to be applied will not be obtained without a fundamental appreciation of the processes that cause the soils to form in the way that they do, and of the mineralogy and micro-fabric which control the observed geotechnical properties. Without such an appreciation, there is a great danger that the apparent soil properties will be misunderstood and that incorrect engineering assumptions will be made, leading to inadequate design or even failure.
Table 8 Grouping of residual soils which can be expected to have similar engineering properties (after Wesley 1988)

<table>
<thead>
<tr>
<th>MAJOR DIVISION</th>
<th>SUB-GROUPS</th>
<th>COMMENTS</th>
</tr>
</thead>
</table>
| Group A        | i) strong macro-structure influence | Nature of macro-structure needs definition  
- stratification  
- fractures, fissures, voids etc  
Remoulding likely to strongly influence behaviour  
- sensitivity should be a useful indicator  
| ii) strong micro-structure influence | Probably a rather minor sub-group |
| iii) little or no structure influence | |
| Group B        | i) Smeectite (montmorillonite) group | Problem soils, characterized by low strength, high compressibility, high shrink/swell behaviour  
(similar characteristics to any montmorillonite soil)  
| ii) Other minerals (?) | ?? |
| Group C        | i) Halloysite | Low activity soils, with good engineering properties  
| ii) Allophane | Low activity soils, with good engineering properties, characterized by very high water contents and large irreversible changes on drying  
| iii) Sesquioxide (lateritic) | Extremely variable group, ranging from silty clay to gravel  

CLASSIFICATION OF TROPICAL SOILS

It is conventional for engineers to classify soils on the basis of their engineering properties. For non-tropical soils this has resulted in a number of fairly well established classification systems which are based on geotechnical index properties, principally grading and plasticity, (for example, the British Soil Classification System [Anon. 1981] and the Unified Soil Classification [Wagner 1957]). Once a soil has been classified by means of one of these systems it is then possible to infer many of the ways in which the soil will behave from an engineering point of view. This is not to say that all the engineering characteristics of the soil can be inferred but such a classification will go a long way towards providing the engineer with information required for preliminary design assumptions. In addition, investigation of temperate climate soils has a long history and the literature contains a wealth of data on a wide variety of the soils. Many engineering problems arise where the classification systems inadequately represent certain geotechnical properties (for example, the swelling characteristics of UK Gault clay or London Clay and the "collapse" characteristics of loess) but, in the main, the systems work for most practical purposes.

With many tropical soils this is not the case and the use of such systems for tropical soils has been criticized (for example, Degraft-Johnson and Bhatia 1969). For some soils the tests used are inadequate, in themselves, to determine the property being measured. For example, the liquid and plastic limit values obtained from "standard" tests for some tropical red clay soils are dependent upon the precise test method employed and, in particular, the amount of energy used in mixing the soil prior to carrying out the test. Consequently, two quite different values of liquid limit could be obtained by two operators testing the same soil depending upon how much effort was put into the pre-mixing of the sample. Different results also will be obtained depending upon whether the soil was pre-dried prior to testing or kept close to its natural moisture content. For other soils, the geotechnical parameters used in the classifications do not represent the most important engineering
characteristics of the soils. The swelling characteristics of Black Cotton Soils are an example of this.

A further problem for engineers working with tropical soils is that the amount of published, or easily available, information on the soils is small (compared with soils from temperate countries). Consequently, there is a tendency to apply temperate soil assumptions and methodologies to these soils, even when these are wholly inappropriate.

Therefore, it seems necessary that the approach to classifying tropical residual soils should be rather different from the accepted methods for temperate sedimentary soils. It has been indicated above that mineralogy and micro-fabric are the main factors which control the material geotechnical properties. For some tropical residual soils macro-fabric also plays an important part in controlling the soils' engineering properties. This factor also needs to be taken into account in deriving a classification system. Therefore, it is logical that the first level of classification should be on this basis. Such an approach is difficult for engineers because the information required is geological but when dealing with tropical residual soils this approach is essential.

Wesley (1988) proposed just such a classification system and his summary table is reproduced here in Table 8.

This classification, in turn, only deals with some of the tropical residual soils. Cook and Newill (1988) divided residual soils into two major groupings: (i) Mature Soil, (ii) Duricrusts. Wesley's (1988) classification is only concerned with the mature soils.

A further problem arises from the consideration of saprolites. These are incompletely weathered materials which retain the parent rock structure. In discussing a review of international practice for sampling and testing of residual soils which summarized comments from a number of countries (Brand and Phillipson 1985), Brand (1985) concluded that the term residual soil included not only mature soils but saprolites and certain transported soils (such as colluvium) as well. These various classifications and divisions are summarized in Table 9.

The classification of Cook and Newill (1988) of duricrusts and mature soils has been adopted by the Working Party of the Engineering Group of the Geological Society in their report on tropical residual soils (Anon. 1990). This classification is principally pedologically based following work by Duchaufour (1982). The main groupings are given in Table 10.

Other proposed classifications have been discussed by De Carvalho et al. (1985)

<table>
<thead>
<tr>
<th>Table 9 Major classification divisions of residual soils</th>
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<tbody>
<tr>
<td><strong>MAIN DIVISIONS</strong></td>
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<tr>
<td></td>
</tr>
<tr>
<td>Residual Soil</td>
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<tr>
<td></td>
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<tr>
<td>Duricrust</td>
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<tr>
<td></td>
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<tr>
<td>Mature soil</td>
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<td></td>
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<tr>
<td>Saprolite</td>
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<tr>
<td>Transported Soil</td>
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<table>
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<tr>
<th><strong>SUB DIVISIONS</strong></th>
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<tr>
<td>Cook &amp; Newill 1988 (Table 10)</td>
</tr>
<tr>
<td>Wesley 1988 (Table 8)</td>
</tr>
</tbody>
</table>

### Duricrusts

Duricrusts are described by Goudie (1973) and are not considered in detail here as they are discussed under arid climate soils, though Ferricretes (including laterites - this term is not recommended because of misuse in the past and confusion over its "correct" usage) and Alcretes

<table>
<thead>
<tr>
<th>Table 10 Classification of residual soils (after Cook and Newill 1988)</th>
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<tbody>
<tr>
<td><strong>MATURE SOILS</strong></td>
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<tr>
<td>Vertisols</td>
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<tr>
<td>Fersiallitic andosols</td>
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<tr>
<td>Fersiallitic (sensu stricto)</td>
</tr>
<tr>
<td>Ferruginous (sensu stricto)</td>
</tr>
<tr>
<td>Ferrallitic</td>
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<tr>
<td></td>
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<tr>
<td><strong>DURICRUSTS</strong></td>
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<tr>
<td>Silcrete</td>
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<tr>
<td>Calcrite</td>
</tr>
<tr>
<td>Gypcrete</td>
</tr>
<tr>
<td>Ferricrete</td>
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<tr>
<td>Alcrete (Alicrete)</td>
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</tbody>
</table>
Vertisols ("Black" Soils)
These are dark mature soils and are characterized by the presence of clay minerals of the smectite group which typically have a high swell and shrink potential and possess contraction cracks and slickensides. The soils (of which Black Cotton Soils are probably the most common) are found on lower slopes and in hollows or valleys, where drainage is restricted, in regions with well defined wet and dry seasons where annual rainfall is not less than 1250 mm. Accumulation of silica and bases, lost in solution up slope, combine with other weathering products to form the smectite clays. Humus is often incorporated into the soils with the help of the swell/shrink process.

Generally, the clay fraction in these soils exceeds 50%, silty material varying between 20 and 40%, and the remainder being sand. Organic content is usually less than 2%. Liquid limits may range between 50 and 100%, with plasticity indices of between 25 and 70%. The shrinkage limit is frequently around 10 to 12% (Clare 1957).

Ola (1978) described some black cotton soils from NE Nigeria as being highly plastic, silty clays formed by the weathering of basaltic rocks and shaly and clayey sediments. This soil contained up to 70% of montmorillonite, kaolinite and quartz making up most of the remainder. Shrinkage and swelling of these soils is a problem in many regions of Nigeria which experience alternating wet and dry seasons. However, the volume changes are confined to an upper critical zone in the soil which frequently is less than 1.5 m thick. Below this, the moisture content remains more or less constant, for example, around 25%. Ola (1980) noted an average linear shrinkage of 8% for some of these soils, with an average swelling pressure of 120 kPa and a maximum of around 240 kPa. In such situations the dead load of a building should be sufficient to counteract the swelling pressure.

Anon. (1990) indicated that vertisols are formed mainly from basic igneous and metamorphic rocks, containing dark minerals such as olivine, augite, hornblende and biotite, and from mudrocks, and that the clay minerals in the vertisols are usually illites, chlorites and smectites. Anon. (1990) also indicated that the soils are prone to erosion and dispersion as well as swell/shrink problems.

Vargas (1985) suggested that expansive soils, as well as other tropical residual soils, could be usefully classified by using a combination of Casagrande's plasticity (Casagrande 1948) and Skempton's (Skempton 1953) activity charts with a common vertical plasticity index axis.

Olo et al. (1987) attempted to identify the soil classification parameters that were most relevant for the classification of expansive soils in terms of their intrinsic expansiveness. They concluded that the shrinkage limit was more appropriate than plastic limit and that the most consistent trends were found when plotting shrinkage index against clay fraction or activity. They also observed that there was a lack of published data including all relevant classification parameters.

"Red" Soils
The remaining soils in the classification of Cook and Newill (1988) can be grouped together, for convenience, as "red" soils. However, this simplistic approach disguises significant differences between different soils formed from different parent rocks under differing climatic conditions over varying periods of time.

As indicated above, Anon. (1990) has proposed a pedological approach to the classification of these soils. Duchaufour (1982) distinguished three phases in the development of residual soils in tropical areas. These phases produce soils termed fersiallitic, ferruginous and ferralitic. Typically, each phase relates to a broad set of climatic conditions:

Fersiallitic soils: Subtropical or mediterranean climates. Mean temperatures 13 - 20°C, annual rainfall 500 - 1000 mm; hot dry season but sub tropical and tropical sub-types known. Main clay mineral is smectite but on older surfaces, well-drained sites or silica-poor parent rocks, kaolinite may be present. Young volcanic ashes produce fersiallitic andosols characterized by allophane, which alters to imogolite or halloysite on weathering.

Ferruginous soils: Either more humid (without dry season) or slightly hotter
than mediterranean areas. Soils more strongly weathered than fersiallitic ones. Kaolinite is the dominant clay mineral; gibbsite is usually absent and smectite is subordinate. On older surfaces and more permeable and base-rich parent rocks, soils intermediate between ferruginous and ferrallitic soils, and known as ferrisols, may be formed.

Ferrallitic soils: Hot, humid tropics, mean temperature $>25^\circ\text{C}$, annual rainfall $>1500$ mm, little or no dry season - tropical rain forest type environment. All primary minerals except quartz are weathered. Much silica and bases removed in solution. Any residual silica converted to kaolinite. Gibbsite may be present. Soils can be divided into ferrites and allites depending upon whether iron oxides or aluminium oxides dominate.

With this form of classification the different groupings should be seen as part of a weathering continuum from fersiallitic soils through to ferrallitic soils. This leads to practical problems in that, for example, fersiallitic soils can be found in a climatic environment conducive to the formation of ferrallitic soils when, for example, the parent material has not been exposed to the climatic conditions for a long enough period of time. In the same way, different soils may occur within the soil profile. With older soils, past (Quaternary) climatic conditions may influence the type of soil found at a particular location.

Practical problems do arise in the use of this classification, which takes parent rocks and the environmental conditions as its basis, because of the difficulty in identifying the variation of climate, at a particular location, with time. In other words, the present environmental conditions are not necessarily a good indicator of the soils to be expected if climate has changed since the formation of the parent rock.

To be able to predict the type of soil to be expected at a site the following information is required: parent rock type, geological age, climatic conditions and history, other environmental conditions (for example, relief, drainage conditions, vegetation). Pedological maps also may provide useful information as Anon. (1990) has given an indication of the various pedological groupings into which their classes fall. However, pedological maps should be treated with caution as a number of different pedological classifications are used and cross-correlation between them is difficult. Two of the main classifications are the FAO/UNESCO system (Anon. 1985) and the USA System (Anon. 1975).

Following the suggestions of Wesley (1988) it may be possible to divide "red" soils into three groupings. At the present time the boundaries of these groupings are imprecise and further divisions may be necessary as research continues. The three groups are as follows:

1. Soils on young Quaternary basic and intermediate volcanic ashes; Andosols; Allophane-rich in a high rainfall, high temperature environment.
2. Soils on older Quaternary and Tertiary basic and intermediate volcanic ashes and lavas in a high rainfall and high temperature environment with or without a pronounced dry season; Nitosols, Alfisols, Uxisols, Andosols, Latosols; Halloysite/metahalloysite rich. Can be subsidiary allophane, goethite and disordered kaolinite present.
3. Soils on other parent rocks such as granites, gneisses, schists, metasediments and sedimentary rocks. High temperature, medium to high rainfall with pronounced dry season; Disordered kaolinite and/or some halloysite present.

On the Casagrande chart these soils fall into three areas (based on Wesley 1988 and Northmore, pers. comm.) (Fig. 5).

### Group 1

This group is characterized by the presence of allophane, an amorphous clay mineral. The soils are have very high moisture contents (c. 80% to $>250\%$) and liquid limits (c. 80% to c. 300%). Plasticity index may be correspondingly high (c. 20% to c. 90%); Moore and Styles (1988) gave values of over 100% for some soils from Papua New Guinea. Wesley (1973) demonstrated similar values for soils from West Java. Plasticity values vary considerably if the soils are tested with or without air drying prior to testing, values of plasticity index and liquid limit being reduced by more than 50% in the air dried state. The plasticity values obtained also are dependant upon the degree of working of the soil during pre-test mixing. In the case of allophanic soils, plasticity decreases with increased working. The soils are also characterized by very low dry densities and high void ratios (sometimes as high as 6). Moore and Styles (1988) also noted that the soils are very strength sensitive, have low CBR values (around 1%) and low dry density values on compaction when tested at natural moisture contents. Because laboratory compaction and CBR tests are normally carried out on air dried soils, design values based on these laboratory tests will be impractical as drying of the soils in a humid tropical climate is normally very difficult. Therefore, tests on samples initially at natural moisture...
content are recommended. These soils are discussed further by Tuncer and Lohnes (1977) and Wesley (1977).

Group 2

These soils contain halloysite, or its partially dehydrated form metahalloysite, which is a kaolin group clay mineral which can occur in a variety of forms including laths, tubes, barrel-shapes, polygons and spheres. Terzaghi (1958), Newill (1961), Matyas (1969), Wesley (1973, 1977) and Hobbs et al. (1988) have all discussed aspects of the geotechnical properties of this group of soils.

Characteristically, these soils have high moisture content (c. 30% to c. 65%) and liquid limits (c. 40% to >100%). Plasticity indices also can be high (10% to c. 50%). Air drying of the soils causes a reduction in plasticity, approximately parallel to the A-Line but by a much smaller amount than for Group 1 soils. The plasticity values obtained are dependant upon the degree of mixing (that is, the amount of energy put into "working" the soil). Plasticity increases with mixing. Complete mixing of the soils using a greaseworker, in which both inter-ped and (probably) intra-ped bonds are broken down to produce a sticky clay paste, causes liquid limit to increase by about 30%. The plasticity values obtained using a greaseworker may be considered to be the maximum values obtainable. The values are also independent of operator mixing effort and consequently, are reproducible. Clay contents are generally high (50% to >80%) and pre-drying appears to have little effect on the particle size determination provided complete dispersion of particles is achieved. Specific gravity values range from 2.65 to 2.85. Newill (1961) suggested that specific gravity could be used to distinguish between the presence of halloysite and metahalloysite in these soils. This is because for halloysitic soils, specific gravity will vary depending upon whether the sample is pre-dried (105°C) or tested at natural moisture content. Drying may result in the loss of intra-particle water (as opposed to "free" water) and a drop in specific gravity.

This group of soils are susceptible to collapse, that is rapid consolidation on flooding with water at constant load. However, the phenomenon does not appear to occur universally in these soils and collapse susceptibility appears to be dependant on factors such as sample depth and disturbance, voids ratio, degree of saturation and test duration prior to flooding.

As with Group 1 soils, compaction tests should be carried out without initial pre-drying. Individual samples should be used for each stage of the test allowing partial drying from the natural moisture content as necessary. Over compaction may increase the susceptibility of these soils to shrinkage. In a number of embankments in Kenya longitudinal shrinkage cracks on shoulders have been observed to coalesce to form tension cracks, sometimes running for tens of metres, which may lead to a risk of slope failure.

Group 3

These soils also may contain halloysite and/or metahalloysite but kaolinite is likely to be present probably in a disordered form. Typically the soils are formed on acid igneous rocks, metasediments and sedimentary rocks. This group of soils is restricted to that part of the soil profile in which none of the parent rocks' structure or texture remains. It corresponds to Grade VI of the weathering classification of Anon (1990) and to the "soil" of Wesley's (1988) gradual weathering profile. Consequently, this Group excludes weathering grades V and IV of Anon (1990) which are termed Saprolite.

In western Kenya, this Group of soils is found developed in granites and is only a few metres thick. The same is probably true in Singapore where Chang (1988) describes weathering grades IV, V and VI as being between about 10 and 35 m thick.

The soils have lower clay contents than for Groups 1 and 2, being usually less than 50%. Quartz contents are higher and the soils range from silty clays to clayey and silty sands. They are usually of medium to high plasticity (liquid limit 30% to c. 60%; plasticity index <10% to c. 25%) and much less sensitive in plasticity tests to pre-drying or to the degree of mixing. This is because the proportion of halloysite in the soils is much lower than in Group 2 soils and the mineral probably has little influence on engineering performance, the silt and sand fractions controlling behaviour (Lumb and Lee 1975). In addition, the iron oxide in these, and other, "red" soils probably plays a part in influencing the engineering behaviour. However, the role of the iron oxide in providing bonds between clay particles or peds is poorly understood.

Jennings and Brink (1978) and Knight (1963) have indicated that some of these soils may be prone to collapse when wetted under certain loads. As with Group 2 soils, further work is required to determine more precisely the nature of this collapse and the control upon it.
Fig. 5 Plasticity chart showing envelopes for tropical red clay soils from Kenya and Indonesia; Group 1: residual soils on younger Quaternary volcanics; Group 2: residual soils on older Quaternary and Tertiary volcanics; Group 3: residual soils on basement granites and gneisses.

Saprolites

As, by definition, saprolites are incompletely weathered materials, it is difficult to generalize on the geotechnical properties and engineering characteristics of this group of soils. Their properties and characteristics are dependent on the nature of the parent rock and the degree of weathering that they have sustained. Anon. (1990) included saprolites in their definition of residual soils, and did not distinguish them from weathering grade VI residual soils. By inference, saprolites are those soils falling in weathering grades IV (highly weathered rock) and V (completely weathered rock).

DISCUSSION

Tropical soils are not as fully understood, from the engineering viewpoint, as temperate soils. This has led to attempts being made to extend classification systems and test methods from the temperate to the tropical soils. For many tropical soils this has been inappropriate. Work in recent years has begun to move towards classification based on a better understanding of the soils and more information is becoming available as engineers and engineering geologists take a greater interest in these materials (see, for example, the Proceedings of the 1st and 2nd Conference on Geomechanics in Tropical Soils, held in Brasilia and Singapore in 1984 and 1988 respectively).

Since 1984, when it was proposed to divide tropical soils into lateritic or saprolitic types (Vargas 1988), much research has been carried out and a number of new systems have been proposed, some of which are outlined above. These systems have many different bases (Anon. 1990). However, as the geotechnical properties of the soils and hence their engineering behaviour appear to be controlled, largely, by the soil composition (mineralogy) and micro-structure, this seems a logical place to start in any classification. It is then necessary to identify the environmental factors (climate, parent rock type, vegetation, drainage conditions etc) and the geological processes that lead to the formation of particular soils. At the same time, the geotechnical properties of the soils, based on tests appropriate to the soils themselves, and the engineering use to which the soils are to be put, must be determined.

Such a classification based on soil-forming processes, soil composition and
microstructure, and geotechnical properties is likely to be complex. However, only when all the elements are more fully understood and documented will a workable classification system be derived. Work, so far, has taken the study of tropical soils only part way down this road.

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