2. WATER IN SOIL

2.1 ADSORBED WATER

As will be discussed in a later chapter, a clay particle carries a net negative charge as a result of isomorphous substitutions. This negative charge is compensated by cations which are located on the surface of the clay particle. In the presence of water these cations tend to diffuse away from the clay surface to form what is referred to as the “double layer”. The cations also attract water because of the dipolar character of the water molecule. The cations tend to diffuse towards the region of smaller cation concentration in the bulk fluid but this tendency is balanced by the electrostatic attraction to the clay particle surface. The water close to the clay mineral surface is also under strong electrostatic attraction. This “adsorbed” water does not have the same properties, such as density, as ordinary free water, but the exact nature of adsorbed water is not fully understood. Martin (1960) has summarised the results of several investigations into the density of adsorbed water. As shown in Fig. 2.1., the density of the adsorbed water immediately adjacent to the clay mineral surface is much higher than that of free water.

The relative importance of adsorbed water increases as the particle size decreases. This is because of the relatively large particle surface area (specific surface) of the fine grained soils. For example, the adsorbed water content of a sand is negligible; that of a kaolinite is almost negligible but in the case of a montmorillonite it may amount to more than 50%.

Because of the high adsorbed water content in some clay soils, some problems relating to the definition of water content may arise. In Chapter 1 water content was defined as the ratio of the mass of water in a sample of soil to the mass of soil solids in the sample. In the laboratory this mass of water is determined by a standard technique which involves measuring the decrease in mass of a soil sample after drying for a period of 24 hr. in a temperature controlled oven at 105°C. If the soil is not noticeably organic it is assumed that the decrease in mass is caused by removal of water.

It is important to realise that this standard technique does not necessarily yield a measurement of all of the water that may be present in a soil. This point is illustrated in Fig. 2.2 which shows the effects of drying oven temperature on the measured water content of samples of wet soils. This figure indicates that all of the soil water is not removed from some clay soils when the soil is subject to the standard technique at 105°C. A portion of the adsorbed water remains and can only be removed by the application of heat much greater than can be provided by an oven set at 105°C.

In routine calculations relating to geotechnical engineering problems, soil water is usually considered in a fairly simplistic way. It is considered to have the same density as ordinary
water and in the standard oven treatment for determination of water content, all of the water in the voids of a soil is assumed to be removed.

Fig. 2.1 Density of absorbed water on Sodium Montmorillonite (After Martin, 1960)

Fig. 2.2 Drying Curves
Fig. 2.3 Air-water Interface

Fig. 2.4 Interface in a Capillary tube

Fig. 2.5 Water between two Spheres
2.2 CAPILLARITY

At a horizontal air-water interface the pressures in the air and in the water are equal. In the field the water surface at which this occurs is known as the water table. In other words the pore water pressure (i.e. pressure of the water in pores or voids of a soil) at the water table is zero with respect to atmospheric pressure. At a curved air-water interface the pressures in the air and water are unequal and surface tension maintains equilibrium. The water that is held in a soil by surface tension forces is referred to as capillary water.

Let Fig. 2.3 represent a hemispherical air-water interface. For vertical force equilibrium.

\[ T. \ 2\pi r + p_w \cdot \pi r^2 = p_a \cdot \pi r^2 \]

\[ \therefore \ p_w - p_a = \Delta p = \frac{-2T}{r} \quad (2.1) \]

where \( T \) is the surface tension and equals 72.75 m N m\(^{-1} \) at 20°C and \( \Delta p \) is the subatmospheric pressure or suction in the soil water. The pressure is greater on the concave side of the interface.

If Fig. 2.4 represents the curved air-water interface in a small diameter tube (a capillary tube) an expression for \( \Delta p \) can be found in a fashion similar to that used above. In this case however the surface tension force does not act vertically but at an angle \( \alpha \) to the vertical where \( \alpha \) is the wetting angle. for this situation

\[ \Delta p = \frac{-2T \cos \alpha}{r} \quad (2.2) \]

with water in a clean glass capillary tube the angle \( \alpha \) is zero so equation (2.1) can be used without error.

For an air-water interface having unequal curvatures in two directions the value of \( \Delta p \) is given by (Adam, 1941):

\[ \Delta p = -T \left( \frac{1}{r_1} + \frac{1}{r_2} \right) \quad (2.3) \]

where \( r_1 \) and \( r_2 \) are the two radii of curvature at a point of the air-water interface and which are measured at right angles to each other.
EXAMPLE

Referring to Fig. 2.5 determine the pressure in the water forming the meniscus between the two equal size spheres.

If Fig. 2.5 is considered as a vertical section then the radius of curvature at point A of the air water interface in the vertical plane is 0.8 mm and the radius of curvature in the horizontal plane is 0.7 mm in the opposite sense. Applying equation (2.3)

\[
\Delta p = -T \left( \frac{1}{r_1} + \frac{1}{r_2} \right)
\]

\[
\Delta p = p_w - p_a = -72.75 \times 10^{-3} \times \left( \frac{1}{0.8} - \frac{1}{0.7} \right) \times 10^3
\]

= -72.75 (1.250 - 1.429)

= -72.75 \times (-0.179)

= 13.0 \text{ N/m}^2

The pressure \(\Delta p\), is positive which means that the pressure in the water is greater than atmospheric pressure.

When a capillary tube, open at both ends is placed with its lower end in water as illustrated in Fig. 2.6(a) the water will rise in the tube a distance \(H_c\), which depends upon the radius of the tube. This distance \(H_c\) is referred to as the capillary rise. The magnitude of \(H_c\) can be determined as follows:

At point A in fig. 2.6(a) the pressure difference across the curved air-water interface is

\[
\Delta p = p_w - p_a = \frac{-2T}{r}
\]

\[
\therefore p_{wA} = p_a - \frac{2T}{r} \text{ at point A}
\]

The pressure in the water in the capillary tube at point B can be determined from a knowledge of the pressure in the water at point A, the density of water \(p_w\) and the capillary rise \(H_c\).

\[
p_{wB} = p_{wA} + p_w g H_c
\]

but since point B is level with the free water surface

\[
p_{wb} = p_a', \text{ the atmospheric pressure}
\]
Fig. 2.6 Pressures in a Capillary tube

Fig. 2.7 Wetting and Draining of a Sandy soil.
\[ p_a = p_{WA} + p_w g H_c \]

\[ = p_a - \frac{2T}{r} + p_w g H_c \]

\[ \therefore p_w g H_c = \frac{2T}{r} \]

or \[ H_c = \frac{2T}{p_w g r} \quad (2.4) \]

The pressure in the water at any point C, a distance h above the free water surface can also be found

\[ \rho_{wC} = \rho_{wB} - \rho_w g h \]

\[ = \rho_a - \rho_w g h \]

\[ = -\rho_w g h \] with respect to atmospheric pressure.

The distribution of pressure along a capillary tube is shown in Fig. 2.6(b).

### 2.3 CAPILLARY RISE IN SOILS

In order to illustrate capillary behaviour in soils the capillary tube analogy is sometimes applied. With this analogy it is assumed that the pores or voids in the soil can be represented by equivalent vertical tubes and that the capillary rise \( H_c \) and the water pressure distribution above the water table depends only upon the average radius of these pores. Unfortunately the true situation in soils is much more complex and reliance cannot be placed upon the capillary tube analogy.

The capillary rise in soils depends upon the direction of movement of the water. The capillary rise for a soil which is being drained by allowing downward movement of water is greater than that for the same type of soil which is being wetted by upward (capillary) flow of water. This point is illustrated in Fig. 2.7 which shows the variation of degree of saturation at various points above the free water surface. Curves A were obtained from a sand sample that was initially saturated. the free water surface was then lowered allowing the sample to drain. As a result the upper portion of the soil became partly saturated as shown in Fig. 2.7. Curves B were obtained from the sand sample that was initially dry. The base of the sample was placed in contact with water and water moved upwards into the soil as shown by the curves.
This behaviour may be simply explained in terms of the shapes of the voids in a soil. For a soil that is being drained the water flows through a series of relatively large voids and narrow openings. The curvature of the air-water interface increases as the size of the opening decreases. The stage will ultimately be reached where the drainage will stop because the curvature of the interface will be increased sufficiently as the water passes through a narrow opening that the height of the water above the free water surface can be supported by surface tension forces.

In the wetting case the water will continue to rise as long as the height of water above the free water surface can be supported by surface tension forces. The water will not rise to the same height as in the draining case since the rising water will at some stage be unable to rise through the large voids to reach the narrow openings above.

For finer soils (smaller grain size) the voids will be smaller and the values of capillary rise become greater. The following approximate values of capillary rise will give some impression of the relative effect of grain size:

- capillary rise for a coarse sand = 0 approx.
- capillary rise for a fine sand = 2m approx.
- capillary rise for a silty soil = 10m approx.
- capillary rise for a clayey soil = 50m approx.

### 2.4 SOIL SUCTION AND pF

An alternative method of representing sub-atmospheric water pressures or soil suction is by means of the pF scale (Schofield, 1935) in which

\[ pF = \log_{10} h \]  

where \( h \) is the height in centimetres of a column of water which would give a pressure numerically equal to the suction. For example, the pF at a point 1 m above the water table in a saturated clay soil is

\[ pF = \log_{10} (100) = 2 \]

the total suction \( h \) in equation (2.5) is made up of two components, the matrix suction and the solute suction.

\[ h = h_m + h_s \]  

where \( h_m \) = matrix suction which is governed by the physical environment of the soil water
\( h_s \) = solute suction or osmotic pressure due to dissolved salts.
A wide variety of techniques (Croney, Coleman and Bridge, 1952) has been developed for the measurement of soil suction over various ranges of pressure and some of these are shown in Fig. 2.8. Brief descriptions of these methods are given below.

2.4.1 Suction Plate

This is a laboratory technique in which the soil is placed in close contact with a moist porous plate (Fig. 2.9), the suction of which can be controlled. If the soil initially has a higher suction that the plate, moisture enters the soil until the soil suction becomes equal to that of the plate. This device may be used to determine the suction - water content relationship for a soil although it suffers the disadvantage of having a slow time response.

2.4.2 Tensiometer

This device (Fig. 2.10) is used to measure the in-situ suction of a soil. When the porous tip is surrounded by soil, water is drawn from the tip reducing the pressure in the measuring system. Equilibrium is reached when the pressure in the porous tip is equal to the soil suction.

2.4.3 Pressure Membrane

The principle of operation of this laboratory device is very similar to that of the suction plate except that air pressure is applied to remove moisture from the soil sample. As shown in Fig. 2.11 the soil is placed inside a pressure chamber and in contact with a saturated porous membrane. The porosity of the membrane determines the range of suction over which the device can be used. The apparatus can be used to measure the soil suction - water content relationship for a soil, but, as in the case with the suction plate the response time is slow.

2.4.4 Centrifuge

The range of suction over which a sample can be subject may be increased by increasing the gravitational acceleration in a centrifuge. The soil is placed on a porous block which is in contact with water a distance h below the soil sample as illustrated in Fig. 2.12. In the earth’s gravitational field the suction on the sample after equilibrium is reached is given by the negative head of water, h. By the application of a centrifugal force the suction can be increased in proportion to the increased acceleration.
Fig 2.8 Methods of Measuring Soil Suction

Fig 2.9 Suction Plate Apparatus
Fig. 2.10 Tensiometer

Fig. 2.11 Pressure Membrane Apparatus

Fig. 2.12 Centrifuge Cup
2.4.5 Vacuum Dessicator

With this laboratory device a soil sample is placed inside a chamber in which the humidity can be controlled. The suction (h cm) can be determined from the relationship between humidity and suction

\[ h = - \frac{RT}{Mg} \ln \left( \frac{H}{100} \right) \]  \hspace{1cm} (2.7)

where

- H is the relative humidity (%)
- R is the universal gas constant \((8.315 \times 10^7 \text{ ergs/mole/}^\circ\text{C})\)
- M is the molecular weight of water \((18.0 \text{ gm/mole})\)
- T is the absolute temperature

Equation (2.7) can be expressed in terms of the pF scale as

\[ \text{pF} = 6.502 + \log_{10} (2 - \log_{10} H) \] \hspace{1cm} (2.8)

The relationship between pF and H is shown in Fig. 2.13. Because the response time with this technique is extremely slow the time is shortened significantly by the application of a partial vacuum and removal of most of the air from the sealed chamber.

2.4.6 Porous blocks

When a moist porous material is placed in contact with soil, moisture transfer occurs until the suction in the porous material is equal to that in the soil. The suction in the porous material is related to the electrical resistance so the suction may be determined from a measurement of resistance between two electrodes which are placed in a block of the porous material as shown in Fig. 2.14. The porous block technique may be used either in the field or in the laboratory to measure matrix suction but its response is very slow, and as shown in Fig. 2.15, the porous materials which are usually used exhibit hysteresis effects. This figure also indicates that alumina cement may be used as the porous block material over a much larger range of suction than is possible with Plaster of Paris.

2.4.7 Psychrometer

As shown in Fig. 2.8 this instrument (Fig. 2.16) can operate over a wide suction range. (Richards, 1965). It is primarily a laboratory device for measurement of total suction but portable versions have been used in the field. The instrument operates as a wet and dry bulb thermometer,
with temperature being measured by the voltage output of the thermocouple junction. The wet bulb condition is achieved by placing a drop of water on the junction, or, preferably, by causing condensation following cooling of the tip. This is achieved by the Peltier effect on current reversal. The psychrometer is relatively inexpensive, easy to use and has a rapid response. The suction is determined from equation (2.7).
2.5 APPLICATION OF SUCTION MEASUREMENTS

In many situations measurements of soil suction may yield better indications of potential soil behaviour than would measurements of water content. This is particularly true in partly saturated soils. For example soil suction measurements can indicate direction of flow of moisture whereas water content measurements cannot. Poor correlations are often found between water content and soil parameters such as strength and compressibility, however correlations with the soil suction are usually much better. (Richards, 1967.)

Many of the techniques described briefly in section 2.4 may be used to determine the suction - water content relationships for a soil under conditions of wetting and drying. This relationship typically shows hysteresis effects as illustrated in Fig. 2.17. This figure is for a clayey soil which was oven dried from an initial water content of 34%. The soil was rewetted to a pF value of 0.5, then it was dried a second time. As shown in Fig. 2.17 three different curves were obtained for these three stages.

Measurements of suction in the field at different times of the year may yield information of the type shown in Fig. 2.18. In conjunction with information of the type shown in Fig. 2.17 the maximum water content changes at various depths may be determined. If it is assumed that the soil remains substantially saturated during these water content changes calculations of volume change and surface movement may be carried out, as illustrated in the example in section 2.6.

EXAMPLE

Determine the directions of moisture flow in the following cases in which two soils are placed in contact with each other

(a) clay (1) with water content, \( w = 18\% \) and suction, \( h = 800 \text{ kN/m}^2 \) in contact with clay (2) with \( w = 21\% \) and \( h = 3000 \text{ kN/m}^2 \).

(b) sand with \( w = 13\% \) and pF = 2.4 in contact with a clay with \( w = 25\% \) and pF = 4.0.

In case (a) moisture will move from clay (1) to clay (2) and in case (b) moisture will move from the sand to the clay in spite of the fact that in both cases moisture is moving from the drier to the wetter soil. The reason for these movements is that water moves from a soil having the lower suction to the soil having the higher suction.
Fig. 2.15 Relationships between suction and water content  
(after Croney, Coleman and Bridge, 1952)

Fig. 2.16 Psychrometer
Fig. 2.17 Suction-Water content curves for a clayey soil  
(after Croney and Coleman, 1953)

Fig. 2.18 Measured Suction Profiles  
(after Richards, 1967)
2.6 SWELLING AND SHRINKAGE OF SOIL

As a saturated clay soil dries it shrinks in volume by an amount equal to the volume of water removed. This process continues as the water content is decreased until air invades the voids of the soil and the volume change ceases. The water content at which this occurs is called the shrinkage limit. A typical experimental plot from which the shrinkage limit may be determined is shown in Fig. 2.19.

When some dry soils are wetted they increase in volume but this behaviour is not characteristic of soils such as gravels, sands and silts. The amount of swelling that may occur depends upon the type and orientation of clay minerals within the soil. Generally soils having greater values of activity and greater mineral surface areas display the greater swelling characteristic. A comparison of swelling and shrinkage characteristics for three common clay minerals is given in Table 2.1.

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Typical Surface Area m²/g</th>
<th>Shrinkage Limit (%)</th>
<th>Relative Swelling Behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montmorillonite</td>
<td>600</td>
<td>8 - 15</td>
<td>High</td>
</tr>
<tr>
<td>Illite</td>
<td>60</td>
<td>15 - 17</td>
<td>Medium</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>10</td>
<td>25 - 29</td>
<td>Low</td>
</tr>
</tbody>
</table>

The volume change of a swelling soil can be prevented by applying a sufficiently high confining pressure - known as the swelling pressure. Swelling pressures for some soils can reach values of 1000 kN/m². The probable expansion as a result of saturating an originally dry soil, can be estimated from the experience of the U.S. Bureau of Reclamation, which has been summarised in Table 2.2. The expansion values quoted here are based on the assumption that the expanding soil is subjected to a nominal surcharge pressure of 6.9 kPa (1 psi).
TABLE 2.2

Probable Expansion Estimated From Classification Test Data
(after U.S.B.R. (1974))

<table>
<thead>
<tr>
<th>Degree of Expansion</th>
<th>Colloidal Content (% &lt; 1µm)</th>
<th>Plasticity Index (%)</th>
<th>Shrinkage Limit (%)</th>
<th>Probable Expansion (% of total volume)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt; 15</td>
<td>&lt; 18</td>
<td>&gt; 15</td>
<td>&lt; 10</td>
</tr>
<tr>
<td>Medium</td>
<td>13 - 23</td>
<td>15 - 28</td>
<td>10 - 16</td>
<td>10 - 20</td>
</tr>
<tr>
<td>High</td>
<td>20 - 31</td>
<td>25 - 41</td>
<td>7 - 12</td>
<td>20 - 30</td>
</tr>
<tr>
<td>Very High</td>
<td>&gt; 28</td>
<td>&gt; 35</td>
<td>&lt; 11</td>
<td>&gt; 30</td>
</tr>
</tbody>
</table>

EXAMPLE

The subsoil beneath a house includes an extensive layer of saturated expansive clay 1.5m thick. The density of the clay is 2200 kg/m³ and the water content is 14%. The shrinkage limit of the clay is 11.0%. At the end of a season of extensive rainfall it is found that the water content of the saturated expansive clay has increased to 16%.

Estimate the amount of heave the house will experience.

The first step in the solution will be to determine the relative volumes occupied by water and by solid in the saturated soil before the rainfall. This may be done with the assistance of a phase diagram.

Select a total volume of 1 m³ for calculation purposes

\[
M_W + M_S = 2200 \text{ kg}
\]

but \[M_W = 0.14 \times M_S\]

\[
\therefore 1.14 \times M_S = 2200 \text{ kg} \\
M_S = 1930 \text{ kg} \\
M_W = 270 \text{ kg}
\]

consequently \[V_W = \frac{M_W}{\rho_W} = \frac{270}{1000} = 0.27 \text{ m}^3\]

\[
\therefore V_S = 1 - 0.27 = 0.73 \text{ m}^3
\]

when the water content has increased to 16%
new $M_W = 0.16M_S$

$= 0.16 \times 1930 = 309 \text{ kg}$

$\therefore$ increase in volume of water per m$^3$ of total original volume

$= 0.309 - 0.270 = 0.039 \text{ m}^3$

In other words the volumetric expansion (strain)

$= \frac{0.039}{1} = 0.039$

Since the clay deposit is extensive it can be safely assumed that the expansion will occur in the vertical direction.

$\therefore$ Heave = expansive strain x clay thickness

$= 0.039 \times 1.5$

$= 0.0585 \text{ m}$

$= 58.5 \text{ mm}$

**EXAMPLE**

In a uniform deposit of expansive clay ($G = 2.70$), measurements of soil suction at various depths were made over a period of years to determine the maximum and minimum suction profiles. Using the appropriate soil suction - water content relationship for the clay, these suction profiles were converted into maximum and minimum water content profiles, which have been plotted in Fig. 2.20.

**Fig. 2.19 Determination of Shrinkage Limit**
Fig. 2.20

Fig. 2.21
After one particular wet season, a small residential structure was erected on the surface of this clay. Making appropriate calculations, estimate the maximum likely movement of the structure that would occur as the clay dries out during a subsequent dry season, assuming that the clay remains saturated.

The movement of the ground surface will occur as a result of volume changes in the soil down to a depth of 2m. Since there is no water content change below this depth, nor will there be any volume change. The solution procedure is to determine the volumetric strain at various depths corresponding to a water content change from the maximum to the minimum value.

It may be shown with the assistance of the phase diagram that the total volume \( V \) of a sample of soil containing a dry mass \( M_S \) of 1000kg is

\[
V = (0.370 + w) \, m^3
\]

in which the water content \( w \) is expressed as a ratio (not a percentage). On the same basis, it is evident that the volume change \( \Delta V \) is numerically equal to the water content change \( \Delta w \)

\[
\Delta V = \Delta w = (w_{\text{max}} - w_{\text{min}})m^3
\]

The calculations for volumetric strain \( \Delta V/V \) are now carried out as tabulated below:

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>( w_{\text{max}} )</th>
<th>( V ) (m³)</th>
<th>( w_{\text{min}} )</th>
<th>( \Delta V ) (m³)</th>
<th>( \Delta V/V )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.320</td>
<td>0.690</td>
<td>0.140</td>
<td>0.180</td>
<td>0.261</td>
</tr>
<tr>
<td>0.2</td>
<td>0.291</td>
<td>0.661</td>
<td>0.162</td>
<td>0.129</td>
<td>0.195</td>
</tr>
<tr>
<td>0.4</td>
<td>0.272</td>
<td>0.642</td>
<td>0.178</td>
<td>0.094</td>
<td>0.146</td>
</tr>
<tr>
<td>0.6</td>
<td>0.258</td>
<td>0.628</td>
<td>0.188</td>
<td>0.070</td>
<td>0.111</td>
</tr>
<tr>
<td>0.8</td>
<td>0.246</td>
<td>0.616</td>
<td>0.194</td>
<td>0.052</td>
<td>0.084</td>
</tr>
<tr>
<td>1.0</td>
<td>0.237</td>
<td>0.607</td>
<td>0.200</td>
<td>0.037</td>
<td>0.061</td>
</tr>
<tr>
<td>1.2</td>
<td>0.229</td>
<td>0.599</td>
<td>0.203</td>
<td>0.026</td>
<td>0.043</td>
</tr>
<tr>
<td>1.4</td>
<td>0.222</td>
<td>0.592</td>
<td>0.207</td>
<td>0.015</td>
<td>0.025</td>
</tr>
<tr>
<td>1.6</td>
<td>0.216</td>
<td>0.586</td>
<td>0.209</td>
<td>0.007</td>
<td>0.012</td>
</tr>
<tr>
<td>1.8</td>
<td>0.213</td>
<td>0.583</td>
<td>0.210</td>
<td>0.003</td>
<td>0.005</td>
</tr>
<tr>
<td>2.0</td>
<td>0.210</td>
<td>0.580</td>
<td>0.210</td>
<td>0.000</td>
<td>0.000</td>
</tr>
</tbody>
</table>

The volumetric strain has been plotted as a function of depth in Fig. 2.21. Assuming that, as the shrinkage takes place, the volume change occurs equally in lateral and vertical directions the vertical strain at a point will be equal to one third of the volumetric strain at that
point. It is evident that the ground surface movement is equal to one third of the hatched area shown in Fig. 2.21 and this may be determined by graphical integration.

∴ Ground movement = 54mm

2.6.1 Experimental Data Relating to Swelling and Shrinkage

While there is a standard test procedure for the measurement of the shrinkage limit, this test is subject to considerable uncertainty, has some undesirable features and is often not conducted. As an alternative some approximate procedures have been devised to assist in the estimation of the shrinkage limit. For example, Karlsson (1977) has shown (Fig. 2.22) that for highly sensitive clays the shrinkage limit of undisturbed samples is greater than the plastic limit. For medium sensitivity clays the shrinkage limit is close to the plastic limit and for organic soils the shrinkage limit is significantly less than the plastic limit.

A very simple procedure for estimating shrinkage limit using the U-line and the A-line on the plasticity chart has been suggested by Casagrande (Fig. 2.23). The U-line or upper limit line represents the upper range of plasticity index and liquid limit coordinates that has been found for soils at the present time. The U-line and the A-line are extended to intersect at point 0. A line is then drawn from point 0 to the coordinates of the liquid limit and plasticity index on the plasticity chart. The shrinkage limit is where this line crosses the liquid limit axis.

Unsaturated soils having low in-situ dry densities may undergo large volume changes following saturation. These metastable or collapsing soils are often aeolian in origin. The volume change may or may not be the result of the application of additional load. Based largely on the experience of the U.S. Bureau of Reclamation (now the U.S. Water and Power Resources Service) Fig. 2.24 has been developed to assist in identifying in-situ soils that may be collapsing or expansive in behaviour.

For compacted clays, Seed et. al. (1962) have developed Fig. 2.25 to assist in identifying potentially swelling soils. This chart has also been found to be applicable to natural soils. The swelling potential in this figure has been defined as the percent swell of a laterally confined sample on soaking under a 6.9kPa surcharge and can be related to soil properties by means of the expression

\[
S = (3.6 \times 10^{-5}) (A^{2.44}) (C^{3.44})
\]

(2.9)

where

\[S\] is the swelling potential in percent

\[A\] is the activity of the soil
C is the percent clay

![Graph showing the relation between the shrinkage limit of undisturbed and remoulded samples and the plastic limit for several Swedish clays.

Fig. 2.22 Relation between the shrinkage limit of undisturbed and remoulded samples and the plastic limit for several Swedish clays (after Karlsson, 1977)

![Diagram illustrating the Casagrade procedure for estimating shrinkage limit.

Fig. 2.23 Casagrade procedure for estimating shrinkage limit]
Fig. 2.24 Guide to Collapsibility and Expansion based on insitu dry densities and the liquid limit (adopted from Mitchell and Gardner, 1975)

Fig. 2.25 Classificaton chart for Swelling Potential (after Seed et. al, 1962)
REFERENCES


