Clay particles themselves are somewhat tough and flexible. Unlike sand grains, they can be deformed or bent before they break. As they bend, however, they develop more and more resistance to further deformation.

The interstices, or pores, surrounding the clay particles are extremely small. The porewater is in thin layers and strips and is partially bonded to the clay. The partially bonded porewater closest to the clay particles is quite viscous, acting like a very "thick" fluid, becoming "thinner", or less viscous, as distance from the particle increases. The water is trapped, of course, but the thinner water can be forced out of the clay by the introduction of a pressure gradient across the deposit.

Depending on the way a clay is deposited, the clay particles may settle into a very loose *flocculent* or *honeycomb* structure as shown in Fig. 7-2[36]. In slowly deposited sediments, the buildup of the clay deposit can occur with very high void ratios as shown. When loaded, the clay structure can collapse into the large void spaces, producing unusually large and rapid settlements.

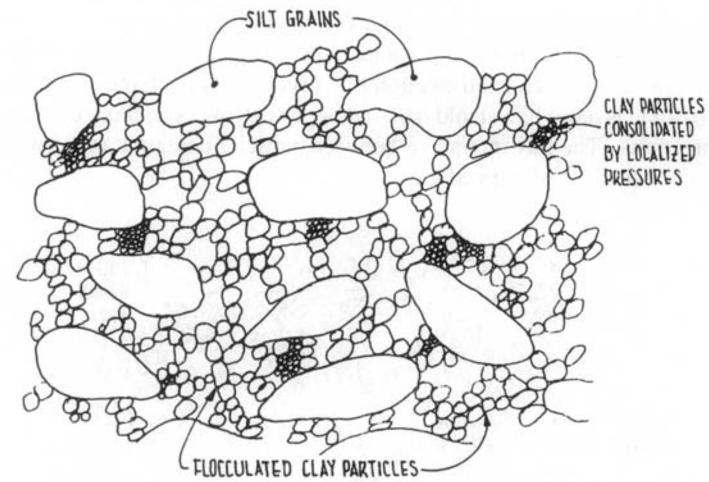

Figure 7-2 Flocculent Silt-Clay Soil Structure

Clay soils having a flocculent structure are called *sensitive* clays. Typically, sensitive clays are subject to relatively rapid large-magnitude settlements under load. Mechanically remolding a deposit of sensitive clay will destroy the flocculent structure, restoring the clay to a more dense state. Such remolding, however, will produce a large reduction in volume and a reduction in strength.

Almost all clay soils exhibit some degree of sensitivity, which means that as a rule, almost any clay will lose some degree of strength when remolded. The effects of a low to moderate degree of sensitivity are usually of little consequence, however, and are usually ignored. The only soils considered in this elementary text are these clays of low-to-moderate sensitivity.

While not rare, clays having high levels of sensitivity are only occasionally encountered in routine practice. When encountered, however, they should be regarded with caution; these highly sensitive clays can be adversely affected by external influences at a particular site. In some cases, for example, driving piles through a deposit of such clay can cause a localized remolding within the deposit, producing unpredictable settlement patterns. Also, the effects of seismic shaking can produce remolding and rapid settlement of foundations in such clays. Specialized textbooks in soil mechanics treat the subject of foundations on sensitive clays in considerable detail[12,14].

A clay that has never been subjected to any pressure greater than the pressure it now sustains, whether from overburden or from foundation loads, is said to be *normally consolidated*. Such lightly condensed soils, compressed only by their own weight, occur in delta regions of large rivers when clay deposits are laid down by periodic flooding. The response of these relatively soft compressible soils to a foundation load is reasonably predictable in comparison to other types of clay.

A normally consolidated clay can often be identified by its in-place moisture content. If the in-place moisture content is just below the liquid limit, the clay is very likely a normally consolidated clay.

Degree of Consolidation

A load placed on any clay soil, including a normally consolidated clay, will at first be carried entirely by an increase in pressure in the trapped porewater, as shown in the time-pressure curve of Fig. 7-3. Over the following months or years, the "thinner" less viscous water is extruded away from the region of increased pressure. As that happens, the load is slowly transferred to the "thicker" water

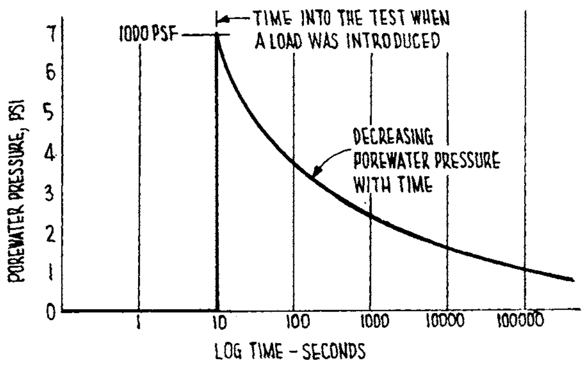

Figure 7-3 Time-Pressure Curve for Load on a Clay Soil

and to the clay particles until equilibrium is reached under the increased pressures and higher viscosities.

When equilibrium is reached, the clay is said to be *100% consolidated at the increased pressure*. At 100% consolidation, the increase in porewater pressure has been completely dissipated; the porewater pressure has returned to its original state. The *degree of consolidation U%* at any time is the percent complete at that time.

The process of making the very slow transfer of load from the porewater to the clay particles, with its corresponding decrease in volume, is given the generic name *consolidation*. It is the mechanism whereby clay deforms under load. It explains why deformations occur so slowly in clays, sometimes taking several months or even years before settlements finally stabilize under an increase in load.

Consolidation settlements in clays are viewed as a long-term phenomenon. Under short-term loads such as wind or earthquake, there simply is not enough time for the porewater to escape outward through the tiny pore spaces. As indicated in Fig. 7-3, all of the increase in pressure from short-term loads can be considered to be sustained entirely by the porewater.

Too, since the voids in clay soils are usually filled with water (or nearly so), and since water is relatively incompressible compared to soil, very little deformation will accompany short-term loads on clay soils. Where air pockets exist, the entrapped air will of course be compressed by the increased pressure, resulting in a small amount of short-term deformation. When the load is released, the air immediately expands, thus producing a small amount of "elastic" rebound. Otherwise, the trapped porewater will sustain the entire short-term load with very little deformation.

Some examples will illustrate some uses of porewater pressure in estimating such things as degree of consolidation and settlements.

Example 7-1 Porewater pressure, settlements and degree of consolidation.

Given : Footing on a stratum of clay soil with a pressure gage installed immediately below the footing to measure porewater pressure. A load is placed on the footing, which causes an increase in porewater pressure of 6.2 psi. At the end of 6 months, the increase in pressure has dropped to 0.96 psi.

To find: The degree of consolidation at 6 months.

Solution:

At the end of 6 months, the increase in pressure has dropped by an amount (6.2 - 0.96)/6.2 or an 84.5% drop, that is, 84.5% of the porewater that is going to be extruded out of the soil has been extruded out. The degree of consolidation $U\%$ is therefore 84.5% at the end of 6 months.

Example 7-2 Porewater pressure, settlements and degree of consolidation

Given : Footing on a stratum of clay soil with a pressure gage installed immediately below the footing to measure porewater pressure. A load is placed on the footing, which causes an increase in porewater pressure of 5.8 psi. At the end of 3 months, the increase in porewater pressure has dropped to 4.3 psi.

The settlement of the footing at that time is measured at 0.45 in.

To find: The total settlement to be expected

Solution:

At the end of 3 months, the increase in pressure has dropped by an amount (5.8 - 4.3)/5.8 or a 25.9% drop, that is, 25.9% of the porewater that is going to be extruded out has been extruded out. As a corollary, 25.9% of the change in volume (and therefore the settlement S) that is going to occur has occurred by this time, or

$$0.259 \times S = 0.45 \text{ in.}$$
$$S = 1.74 \text{ in.}$$

A total settlement of 1.74 in. can be expected at some point in the future.

Example 7-3 Porewater pressure, settlement and degree of consolidation

Given : Spread footing 6 ft. square on a stratum of clay soil. The footing is loaded relatively quickly by a load of 70 kips. Settlement S is monitored monthly thereafter and found to be:

At 1 month : S = 2.30 in.
At 2 months : S = 3.45 in.
At 3 months : S = 4.31 in.
At 4 months : S = 4.60 in.
At 5 months : S = 4.60 in.

To find: Porewater pressure at the beginning of the test and at the end of each month thereafter.

Solution:

At the start of the test, the porewater carries the entire load of the footing. The increase in porewater pressure Δp is computed as:

$$\Delta p = \frac{P}{A} = \frac{70{,}000}{6 \times 6} = 1944 \text{ psf}$$

$$\Delta p = 13.5 \text{ psi at the start of the test}$$

At one month, the settlement is 2.3/4.6 = 0.50 or 50% complete. The increase in porewater pressure has similarly dropped to:

At 1 month: Δp = [(4.60 - 2.30)/4.60]13.5 = 6.75 psi
At 2 months: Δp = [(4.60 - 3.45)/4.60]13.5 = 3.38 psi
At 3 months: Δp = [(4.60 - 4.31)/4.60]13.5 = 0.85 psi
At 4 months: Δp = [(4.60 - 4.60)/4.60]13.5 = 0

Overconsolidated Clay

It is again noted that there is a distinct reduction in the volume of the clay as it consolidates. The loss in volume is of course the volume of porewater that has been extruded outward and lost, with a corresponding reduction in the volume of voids. Such a flow of porewater in a stratum of clay is shown in Fig. 7-4. The clay particles themselves, along with the remaining "thicker" water, are in a state of increased stress with a reduced volume.

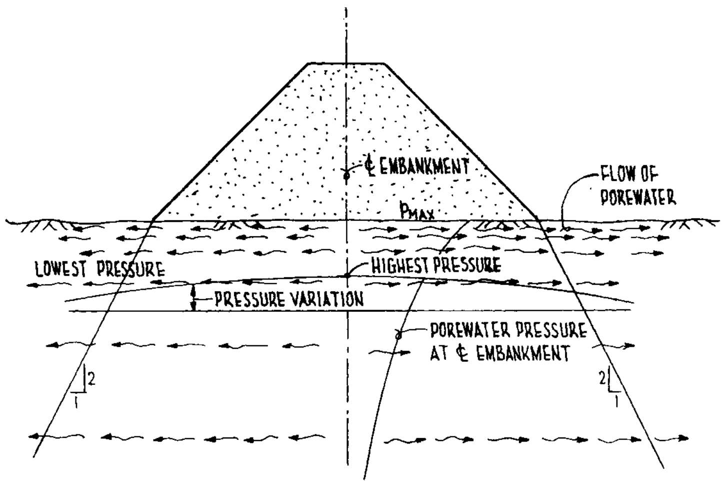

Figure 7-4 Flow of Porewater due to Increased Pressure

When a clay stratum has been loaded by thousands of feet of overburden (or ice-age glaciers) and when this load is eventually eroded (or melted) away, the release of the load creates restoring forces in the clay that can be quite large. The clay remains compressed, like a compressed spring, unable to expand until conditions permit an ingress and restoration of some part of the porewater that had been extruded eons ago.

In this precompressed state, the clay is said to be *overconsolidated*. An overconsolidated clay is defined as any clay that has been subjected to a greater level of consolidation pressure in its past than it now experiences. Of the two possible cases of consolidation that can be found in clay soils, either normal consolidation or overconsolidation, the more common case by far is that of overconsolidation.

An overconsolidated clay can often be identified by its in-place moisture content. If the in-place moisture content is significantly below the liquid limit, the clay is very likely an overconsolidated clay. In some cases, the in-place water content of an overconsolidated clay can even be below the plastic limit.

When a source of water is eventually made available, the ingress of water allows the particles to unbend and to return closer to their original configuration. Such expansion typically happens in small pockets at random locations, producing unpredictable localized heaving. Such small isolated pockets of expanding soil can devastate buildings built on the stratum.

In some geologically overconsolidated clays, the expansion and heaving of the clay can be a serious and ever-present problem. A well-known example of such a clay is the Red Permian clay of Texas, Oklahoma and Kansas. Horror stories about heave in the Red Permian clay are well known to the local foundation designers.

A clay can also become overconsolidated by severe drying, or *desiccation*. As the clay dries, the capillary tension in the porewater can become quite large and can cause severe amounts of volume change due to shrinkage. Desiccation is one of the more common causes of overconsolidation.

Since the change in volume due to desiccation occurs in three directions, a desiccated clay will shrink laterally, which will in turn produce vertical cracking. As indicated in Fig. 7-5, cracks may be several inches wide, up to several feet deep and several yards long, forming permanent planes of weakness in the clay mass.

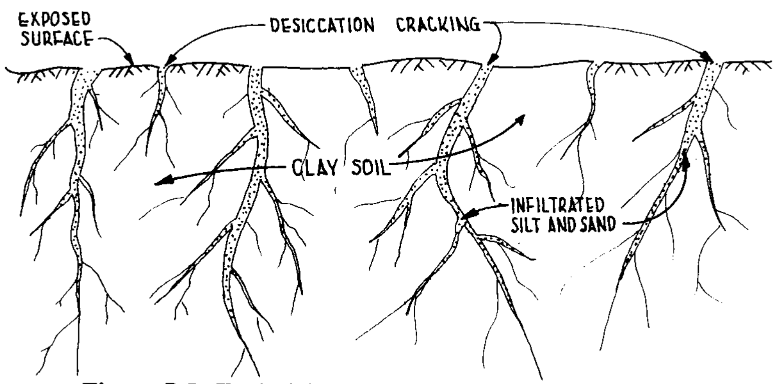

Figure 7-5 Typical Crack Penetration in Desiccated Clay

Desiccated clays are known for their rapid and erratic expansion when a water source finally becomes available. The criss-crossing pattern of open cracks will commonly become filled with windblown silt. Since the silt is more permeable than clay, the filled cracks thus provide a permanent means of ingress for water. Such an ingress of water can of course produce rapid and unpredictable large-scale heaving; the effects on structures can be crippling.

The Consolidation Test for Clay Soils

As with other engineering materials such as steel or concrete, the calculation of deformations in soils begins with a stress-strain curve. In soils, however, the stress-strain curve takes a somewhat different form due simply to the nature of soil. The strain is best viewed as a change in void ratio and the stress is best viewed as the log of the vertical pressure p_1. The reason for using a log scale for pressures comes as a result of the extremely slow rate of consolidation as porewater pressure approaches zero.

Some typical stress-strain curves for plastic soils are shown in Fig. 7-6. Such curves are usually called *e*-log *p* curves rather than stress-strain curves. As indicated, the curves are those for a normally consolidated clay, an overconsolidated clay and a sensitive clay. Shown in dashed lines is an idealized curve for the sensitive clay after it has been remolded.

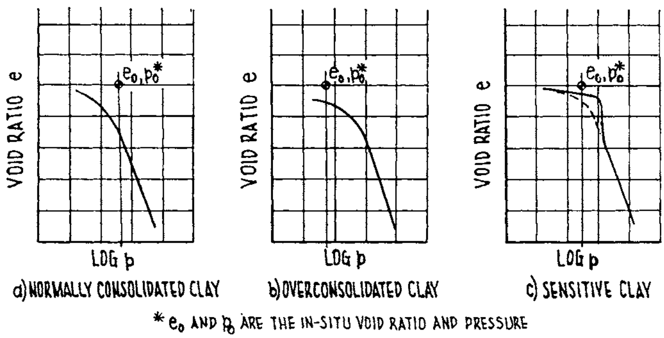

Figure 7-6 Typical *e*-log *p* Curves

Note that the *e*-log *p* curve of the sensitive clay in Fig. 7-6 has a very steep slope when the flocculent structure first begins to break down. The effect is somewhat akin to the "plastic range" in steel, where deformations increase with little or no increase in load. The settlement of footings on such sensitive clays can be quite large and can occur quite rapidly.

All deposits of clay begin their geologic lives as normally consolidated clays. It should not be inferred, however, that pressures in a deposit of normally consolidated clay are low. While it is true that pressures have never been higher than they are now, the pressures at the bottom of a deep deposit of normally consolidated clay can be quite high.

In a consolidation test, a sample of the clay soil is fitted snugly into a circular confining ring and is loaded by a uniform pressure as indicated schematically in Fig. 7-7. The apparatus allows free flow of water out of the sample at top and bottom.

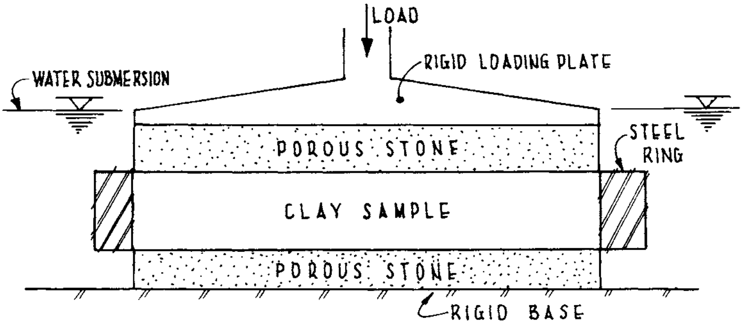

Figure 7-7 Schematic Diagram of a Consolidation Test

Before the test is started, the maximum anticipated increase in the soil pressure is established. Also, the *in situ* void ratio e_0 and the *in situ* overburden pressure p_0 are determined. At the start, the sample is loaded until the *in situ* pressure p_0 on the sample is restored; deformations are allowed to come to rest. The void ratio at that point is taken to be e_0.

The actual test procedure then consists of a step-by-step incremental increase in load on the confined sample until the anticipated maximum pressure on the soil has been reached. At each step of loading, the compressive deformations at various times are recorded for that increment of load. Deformations are allowed to come to a stop before the next increment is added. The procedure is repeated for a number of increments of load until the anticipated maximum pressure is reached.

As an optional part of the test, the rebound deformations of the clay may also be included. At some point well into the procedure, preferably about halfway, all of the added increments of load are removed. The sample will then undergo "rebound" back to pressure p_0. When deformations stabilize, the vertical deformation at this rebound position is recorded. All of the increments are then replaced. When deformations again stabilize, the deformation is recorded and the regular test procedure is resumed. These results will be used later to reproduce the rebound-reload curve of the soil. (Technically speaking, the sample is now overconsolidated.) When the test is complete, results at each increment of load are plotted, providing a graph similar to that of Fig. 7-8a.

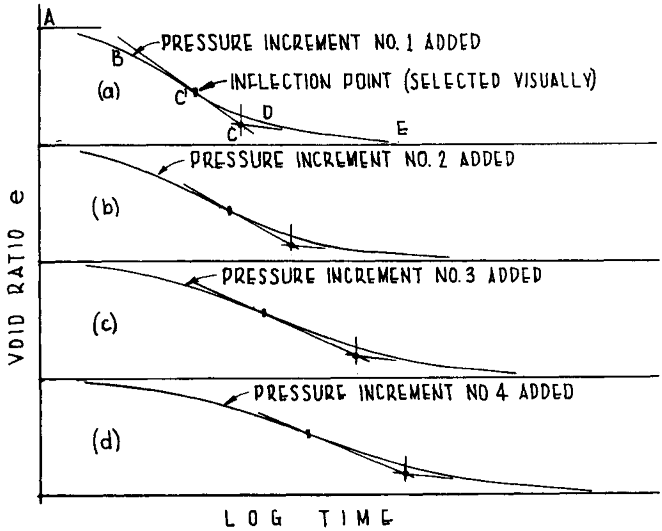

Figure 7-8 *e*-log time Consolidation Curves

The set of curves shown in Fig. 7-8 are commonly called the *e*-log time curves. A complete set of these *e*-log time curves is necessary, one curve for each increment of pressure up to the maximum anticipated pressure.

There are two types of consolidation reflected in the *e*-log time curves of Fig. 7-8a. The first type is the primary consolidation, defined by the initial portion of the curve having a steep downward slope, ABC'. The second type is the secondary consolidation, defined by the final portion of the curve having a shallow upward asymptotic curve C'DE.

A graphical method is used to find the theoretical point C' where the two curves meet. As shown in Fig. 7-8a, a line is drawn through the inflection point C' tangent to both the upper part of the curve and the lower part of the curve. Another line is drawn tangent to the secondary consolidation curve along its straightest portion. The intersection of these two lines occurs at point C, which is taken to be the point where primary consolidation is 100% complete and secondary consolidation begins.

Secondary consolidation is of interest only rarely. The time involved can be upward of 50 to 100 years, far beyond the service life of today's commercial structures. In general, references to consolidation will mean only primary consolidation, or that part of the curve shown as ABC in Fig. 7-8a.

It is emphasized that Fig. 7-8a is drawn using a beginning pressure p_0, with an added increment of pressure producing an increase to $p_0 + \Delta p$. At that pressure, the clay reaches 100% consolidation when void ratio and pressure reach point C. At this point, essentially all the water has been forced out of the clay that is going to be forced out at this increase in pressure.

The initial portion of the primary consolidation curve (up to 50% to 60% of the total) can be considered to be essentially a second order parabola. For all increments of load after the first one, the void ratio at the initial point A corresponding to 0% consolidation can be projected as indicated in Fig. 7-9.

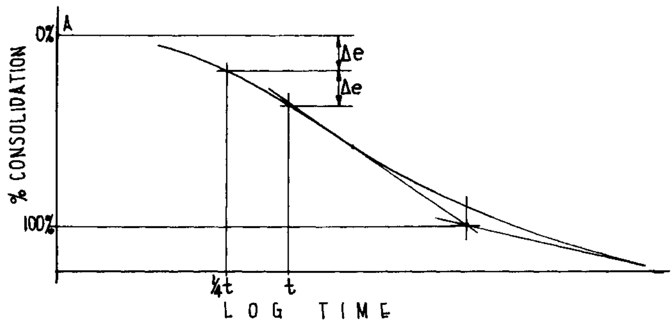

Figure 7-9 Projected Point of Zero Consolidation

To project the curve of Fig. 7-9 to point A, find values of e and t for some arbitrary point in the first half of the curve. Find the point $^1/_4t$ as indicated and draw a horizontal line through that point. Lay off the distance Δe between t and $^1/_4t$ as indicated. Then draw a second horizontal line at a distance Δe above the first horizontal line. The intersection of this second horizontal line with the vertical ordinate is the point of zero consolidation.

Once the points of 0% consolidation and 100% consolidation are known, the scale for the vertical ordinate is readily set. With a 0% to 100% scale on the vertical ordinate, each e-log time curve represents a consolidation-time curve at the given incremental increase in pressure. This concept is developed further in the next section.

Once the set of e-log time curves are complete, the final consolidation curve can be drawn. The values of e and p corresponding to 100% consolidation are taken from the e-log$p$ curves and plotted as a separate graph. The end result of the consolidation test is the consolidation curve shown in Fig. 7-10 above.

The consolidation curve is commonly called the e-log$p$ curve. The e-log$p$ curve for clay is roughly equivalent to the stress-strain curve for steel or timber. Each point of the curve defines the void ratio e at 100% consolidation under the indicated vertical pressure. It should be noted, however, that a consolidation