Soil Mechanics in Engineering Practice

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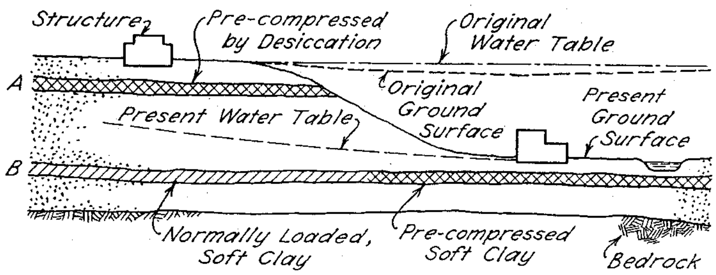

Fig. 13.7. Diagram illustrating two geological processes leading to precompression of clays.

For an ordinary clay of medium or low sensitivity, both the e–log p lines K_r and K are straight over a wide range of pressure, and the value of C_c corresponding to the field consolidation line K appears to be roughly equal to 1.30 C_c' (Eq. 13.10). That is,

$$C_c \sim 1.30C_c' = 0.009(L_w - 10\%) \tag{13.11}$$

If the value of C_c for a given layer of clay is known, the compression of the layer due to a surcharge Δp can be computed by means of Eq. 13.8. For normally loaded clays with low or moderate sensitivity the value of C_c can be estimated roughly by means of Eq. 13.11. Hence, the order of magnitude of the settlement of a structure located above a stratum of such clay can be determined without making any tests other than liquid-limit tests.

Undisturbed Precompressed Clay

A clay is said to be precompressed if it has ever been subjected to a pressure in excess of its present overburden pressure. The temporary excess pressure may have been caused by the weight of soil strata that were later eroded, by the weight of ice that later melted, or by desiccation due to temporary exposure. If the excess pressure Δp_0 was smaller than about 4 kg/cm², the clay may still be soft. If Δp_0 was much greater, however, the clay is stiff.

Two of the processes which lead to the precompression of clays are illustrated in Fig. 13.7. All of the strata located above bedrock were deposited in a lake at a time when the water level was located above the level of the present high ground. When parts of the strata were removed by erosion, the water content of the clay in the right-hand portion of stratum B increased slightly, whereas that of the left-hand

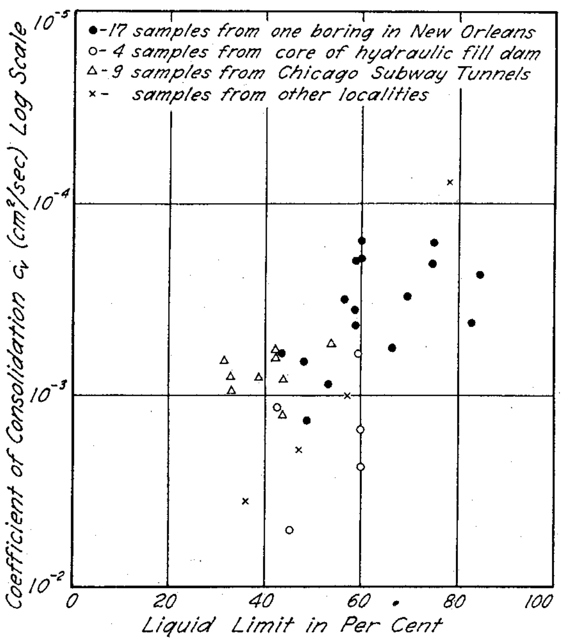

Fig. 14.3. Relation between liquid limit and coefficient of consolidation for undisturbed samples of clay.

of consolidation for clays with a given liquid limit varies within a wide range.

If the pressure in a natural clay stratum is relieved, for instance by the excavation of a shaft or tunnel, the corresponding volume expansion of the clay may not begin for a week or more after the excavation is completed. In a few instances it has even been observed that the consolidation of such strata under the influence of superimposed loads did not start for a few weeks after the load was applied. These delays in the reaction of clay to a change in stress, like the secondary time effect and the influence on c_v of the magnitude of the load increment, cannot be explained by means of the simple mechanical concept on which the theory of consolidation is based. Their characteristics and conditions for occurrence can be investigated only by observation.

In spite of the radical simplifications involved, the theory of consolidation serves a useful purpose, because it permits at least a rough estimate of the rate of settlement due to consolidation, on the basis of the results of laboratory tests. Therefore, the theory is presented briefly in Part II, Article 25.

ART. 18 SHEARING RESISTANCE OF COHESIVE SOILS

Normally Loaded Undisturbed Clays of Low to Moderate Sensitivity

The results of drained triaxial tests on normally loaded cohesive soils can be expressed with satisfactory accuracy by Coulomb's equation in which $c = 0$. Thus

$$s = \bar{p} \tan \phi \tag{18.1}$$

The values of ϕ for such materials, whether in a remolded or an undisturbed state, are related to the plasticity index. Approximate values may be estimated with the aid of Fig. 18.1, although the scattering from the curve for most clays may be on the order of 5° (Bjerrum and Simons 1960). However, the exceptionally high value $\phi = 47°$ was obtained (Lo 1962) for clay with a liquid limit of 426% from Mexico City. Hence, it is apparent that the statistical relation represented by Fig. 18.1 is not of general validity and should be used with caution.

Under conditions usually encountered in the field, the low permeability of clays greatly retards the drainage; as a consequence the pore pressures u_w associated with the forces tending to shear the clay may not dissipate readily. Since the pore pressures associated with shear are positive (Fig. 15.5*c*), the strength indicated by Eq. 18.1 may not be developed for a very long time; the time required for dissipation is governed by the consolidation characteristics and dimensions of the cohesive body (Articles 14 and 25).

The conditions associated with complete lack of drainage may be approximated in consolidated-undrained triaxial tests (Article 15). The results of such a test, in which $\bar{p}_1$ and $\bar{p}_3$ are the effective principal

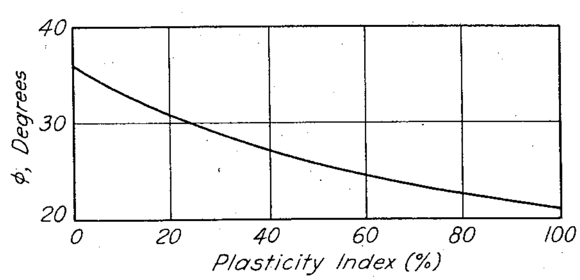

Fig. 18.1. Relation between ϕ and plasticity index for clays of moderate to low sensitivity under drained conditions.

stresses at failure, are represented by the rupture circle E, Fig. 18.2*a*; this circle is tangent to the rupture line defined by Coulomb's equation

$$s = \bar{p} \tan \phi \tag{18.1}$$

At the time of failure, positive pore pressures u_f act in all directions in the sample (see Fig. 18.2*a*). Hence, the total principal stresses at failure are

$$p_1 = \bar{p}_1 + u_f \tag{18.2}$$

and

$$p_3 = \bar{p}_3 + u_f \tag{18.3}$$

The rupture circle in terms of total stresses is then circle A; it has the same diameter as E but is displaced to the right a distance $\bar{A}_f\Delta p_f$ equal to the pore pressure u_f induced in the sample at failure.

If several tests are carried out under undrained conditions on the same clay initially consolidated under different cell pressures p_3, then, in terms of total stresses, the envelope to the rupture circles is also approximately a straight line passing through the origin (dash line in Fig. 18.2*a*), with the equation

$$s = p \tan \phi_{cu} \tag{18.4}$$

where ϕ_{cu}, known as the *consolidated-undrained angle of shearing resistance*, is appreciably smaller than ϕ. The relation between ϕ and ϕ_{cu} is determined by the value of the pore pressure induced by the stress difference $p_1 - p_3$ at failure; for normally loaded clays of low to moderate sensitivity this value is approximately equal to the stress difference itself.

It should be noted that the failure circle for a given test has the same diameter whether it is plotted in terms of effective stresses or total stresses. The pore pressure acts with equal intensity in all directions; hence the incremente of pore pressure is the same for both the major and minor principal stresses. This conclusion leads to an extremely useful concept, known as the $\phi = 0$ condition. In Fig. 18.2*b* the solid circle E is the effective stress circle shown in Fig. 18.2*a*. The total stress circle A corresponds to the consolidated-undrained test in which the pore pressure at the start of the test was zero and the pore pressure at the end of the test was u_f. If, however, after the initial consolidation under the cell pressure p_3, the cell pressure had been increased by an amount u_a without allowing drainage, the initial pore pressure in the sample would have been u_a and the pore pressure at failure $u_a + u_f$. The corresponding failure circle would have been B (Fig. 18.2*b*). The effective stress circle would, nevertheless, still be E.

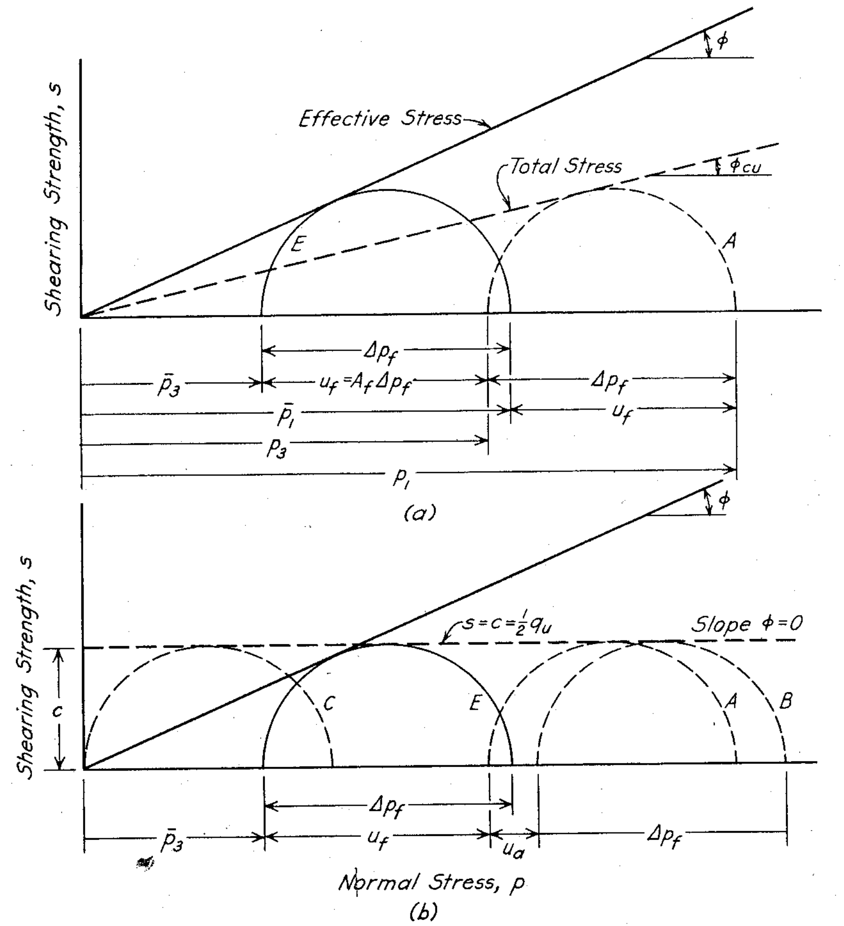

Fig. 18.2. **(*a*) Results of consolidated-undrained triaxial tests on normally loaded clay of moderate sensitivity. (*b*) Diagram illustrating $\phi = 0$ condition.**

Since any change u_a in cell pressure could have been chosen, it follows that if several samples are consolidated under the same cell pressure $\bar{p}_3$ and then are tested under undrained conditions at different cell pressures, the rupture line *with respect to total stresses* is horizontal. The line may be regarded as a special case of Coulomb's equation in which $s = c$ and $\phi = 0$. Hence, these particular circumstances are known as the *$\phi = 0$ condition* (Skempton 1948). Inasmuch as an unconfined compression test is merely a triaxial test in which the total minor principal stress p_3 is zero (circle C in Fig. 18.2*b*), the shearing strength under $\phi = 0$ conditions may be evaluated on the basis of unconfined compression tests as

$$s = c = \tfrac{1}{2}q_u \tag{18.5}$$

In connection with soils of such low permeabilities as those possessed by most clays and some silts, there are many practical problems in which we can assume that the water content of the soil does not change for an appreciable time after the application of a stress. That is, undrained conditions prevail. Moreover, if a sample is extracted at the same water content and is tested without allowing change in water content, either in unconfined compression or with a cell pressure $p_3 + u_a$, the strength of the soil with respect to total stresses will be approximately (within the limitations imposed by Eq. 15.4) the value c as determined readily from Eq. 18.5. Hence, as a consequence of the $\phi = 0$ concept, the unconfined compression test assumes unusual practical importance.

Moreover, when undrained conditions can be expected to prevail in deposits of saturated clay in the field, other expedient types of tests can often be used advantageously for evaluating c. Foremost among these are several varieties of *vane shear tests* as shown in Fig. 44.17. (The equipment for performing vane shear tests in the field is described in Article 44). Similar vanes of smaller size are often used in the laboratory, especially for investigating the strength of samples of very weak or remolded clays. Among the most convenient modifications (Fig. 18.3) is the portable *torvane* (Sibley and Yamane 1965). The vanes are pressed to their full depth into the clay, whereupon a torque is applied through a calibrated spring until the clay fails along the cylindrical surface circumscribing the vanes and, simultaneously, along the circular surface constituting the base of the cylinder. The value of c is read directly from the indicator on the calibrated spring. By means of such a device, rapid and detailed surveys of c can be carried out (see Fig. 45.5).

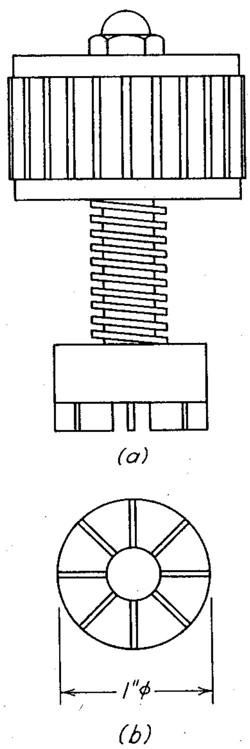

Fig. 18.3. Torvane for determining shear strength of materials for which $s = c$. (*a*) Side view. (*b*) Bottom view of vanes.

Several examples of the use of the $\phi = 0$ concept will be developed in Part III.

If a normally loaded clay is consolidated under an all-around pressure p_3 and then failed under undrained conditions, the failure circle with respect to total stresses is represented by A in Fig. 18.2*a*. The shearing strength under $\phi = 0$ conditions is measured by the radius c of this circle. By geometry (Fig. 18.4*a*)

$$\frac{c}{p_3 + c} = \frac{b}{c}$$

whence

$$\frac{c}{p_3} = \frac{\sin \phi_{cu}}{1 - \sin \phi_{cu}}$$

which, for a given clay, is a constant. This relation has suggested (Skempton 1957) that a similar constant ratio should exist between the undrained shear strength of normally loaded natural deposits, as determined by means of unconfined compression or vane tests, and the effective overburden pressure at the depths corresponding to the strength tests. It has been found that this ratio, designated as $c/\bar{p}$, is indeed constant for a given normally loaded deposit, provided the plasticity index is approximately the same throughout the deposit. Moreover, it has also been found that the field $c/\bar{p}$ values for various deposits or fairly homogeneous portions of deposits are correlated closely with the plasticity index, as shown in Fig. 18.4*b*. Like all statistical relations, Fig. 18.4*b* entails the possibility that exceptions may appear, but so far the relation has been found applicable over a wide range of types of sedimented clays.

The $c/\bar{p}$ ratio, estimated by means of Fig. 18.4*b*, makes possible a rough evaluation of the undrained shear strength of normally loaded deposits on the basis of the results of Atterberg limit tests. Conversely, if the undrained strength has been determined by independent tests, comparison with values based on Fig. 18.4*b* may indicate whether the clay is normally loaded or precompressed.

Extrasensitive and Quick Clays

Most natural clay deposits consist of more or less well graded mixtures of particles of sizes intermediate between those of fine sand and clay, and are relatively insensitive. However, clays consisting primarily of clay-size particles in an edge-to-face structure, or possessing a flocculent structure (Article 4) are likely to experience appreciable loss of strength upon remolding and may exhibit at least moderate sensi-

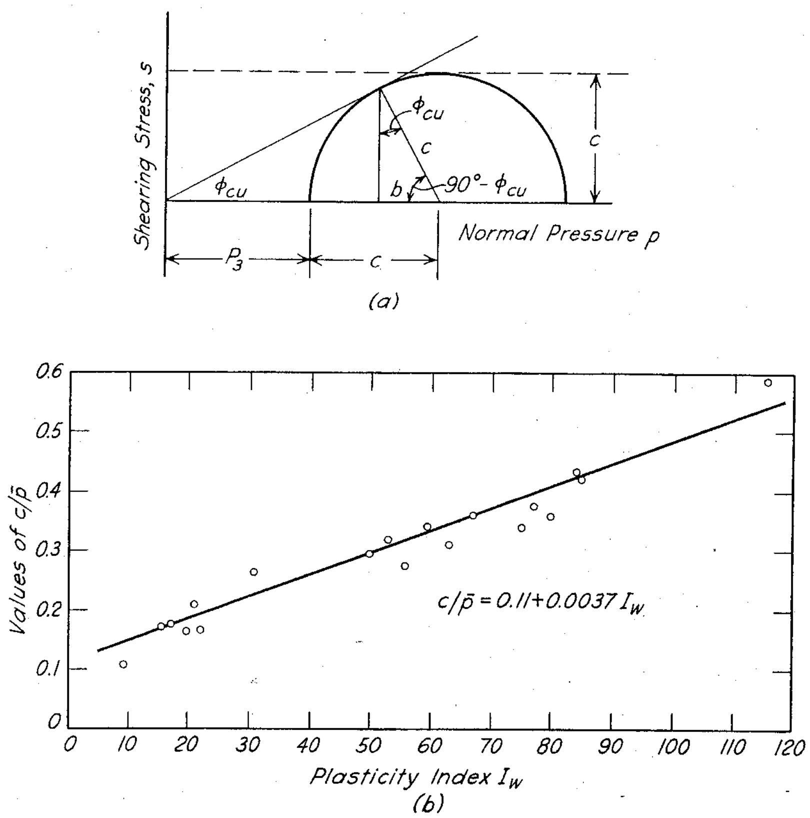

Fig. 18.4. (*a*) **Mohr rupture diagram for calculating relation between** c **and** $\bar{p}_3$ **for consolidated-undrained test.** (*b*) **Statistical relation between** $c/\bar{p}$ **ratio and plasticity index (after Skempton 1957).**

tivity. Some natural clay deposits, moreover, consist of a mixture of particles of fairly uniform fine sand and clay. While sedimentation proceeds, the simultaneous deposition of the flaky constituents of the finest fraction and of the equidimensional sand grains interferes with the rolling of the sand grains into stable arrangements. Therefore, if the sand grains touch each other, their configuration may be as metastable as that of true quicksands. However, the interstices between the sand grains are occupied by the clay-size materials which acquire, as a result of such physico-chemical processes as thixotropy and syneresis, appreciable strength as sedimentation proceeds. As a consequence, although the clay is sensitive, it does not exhibit the properties of true quicksands. In many respects, the states of transition from loose sand to true quicksands have their counterparts in the states between clays with low and very high sensitivity.

The failure of extrasensitive clays, like that of true quicksands, appears to be progressive. However, instead of turning throughout into a viscous liquid, extrasensitive clays break up into relatively solid chunks floating in a viscous liquid that can travel on valley floors to distances of several miles at a rate up to 10 miles per hour. One eyewitness, who had the misfortune to be standing on top of one of the chunks in such a slide, graphically described the character of the material in the following words (Terzaghi 1950):

". . . . after reaching the bottom I was thrown about in such a manner that at one time I found myself facing upstream toward what had been the top of the gully . . . . The appearance of the stream was that of a huge, rapidly tumbling, and moving mass of moist clayey earth. . . . At no time was it smooth looking, evenly flowing or very liquid. Although I rode in and on the mass for some time my clothes afterwards did not show any serious signs of moisture or mudstains . . . as I was carried further down the gully away from the immediate effect of the rapid succession of collapsing slices near its head . . . it became possible to make short scrambling dashes across its surface toward the solid ground at the side without sinking much over the ankles."

Quick clays are normally consolidated marine clays that differ from other extrasensitive clays inasmuch as they have acquired their present degree of sensitivity in two steps: the first during deposition, and the second, far more important, by leaching after being lifted above sea level as described in Article 4. In an undisturbed state such clays are as brittle as other extrasensitive clays. A slope failure on such clays commonly starts at the foot of the slope and proceeds by progressive failure in an uphill direction, even on very gentle slopes. Examples of quick-clay flows are discussed in Article 49.

Intact Overconsolidated Clays

The shear-strength characteristics of an overconsolidated clay under drained conditions are illustrated by Fig. 18.5*a*. The rupture line corresponding to the peak strengths of normally loaded samples is represented by the straight line Od. We may, however, consolidate a number of identical samples under the same cell pressure $\bar{p}_3$. If one such sample is tested under drained conditions by increasing the vertical pressure, the stress on the failure plane at failure is represented by point a on the

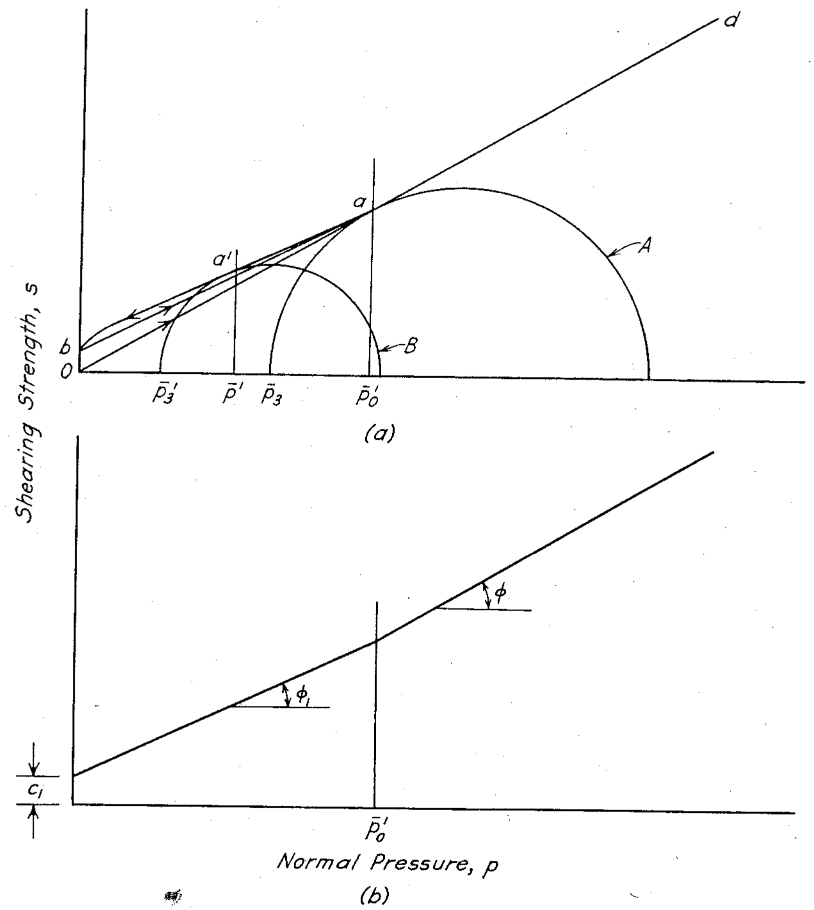

Fig. 18.5. (*a*) **Rupture diagram for clay under drained conditions, preconsolidated to $\bar{p}_0'$.** (*b*) **Simplified rupture lines for same clay.**

circle of stress A. The normal stress on the failure plane is $\bar{p}_0'$. Circle A corresponds to a normally loaded sample.

If one of the samples previously consolidated to $\bar{p}_3$ is allowed to swell under a cell pressure $\bar{p}_3'$ and is then tested under drained conditions, the strength of the sample (circle B) exceeds that of a normally loaded sample tested under the same conditions. The failure envelope $aa'b$ for such samples lies above the line Oa representative of the normally loaded material. The curve $aa'b$ corresponds to the rebound curve bc_1 in the e-log p diagram (Fig. 13.4). If several samples are first consolidated to the stress $\bar{p}_3$, then allowed to swell under zero pressure, and finally are consolidated under various pressures before the performance of drained tests, it is found that the rupture line resembles the lower line ba for pressures less than $\bar{p}_0'$, but for greater pressures is nearly identical to the rupture line Od for the normally loaded clay. The lower line ba corresponds to the reloading portion of the e-log p curve (Fig. 13.4).

As a rough approximation, the rebound and reloading branches $aa'b$ and ba of the rupture line may be replaced (Fig. 18.5b) up to the pressure $\bar{p}_0'$ by the straight line

$$s = c_1 + \bar{p} \tan \phi_1 \tag{18.6}$$

in which, for a given clay, ϕ_1 is found to be nearly constant whereas c_1, known as the *cohesion intercept*, is found to depend on $\bar{p}_0'$. For pressures greater than $\bar{p}_0'$, the expression

$$s = \bar{p} \tan \phi \tag{18.7}$$

is applicable.

Since for most clays the value of c_1 is very small and ϕ_1 is only slightly smaller than ϕ, a small error on the safe side is made if Eq. 18.7 is considered applicable for all values of $\bar{p}$. Hence, the strength of an intact, moderately overconsolidated clay under drained conditions does not differ significantly from that of normally loaded clays.

In contrast, under undrained conditions the strength of a preloaded clay may be smaller or larger than the drained strength, depending on the value of the overconsolidation ratio. If the overconsolidation ratio lies in the range between 1.0 and about 4 to 8, the volume of the clay tends to decrease during shear and the undrained strength, like that of a normally loaded clay, is less than the drained strength. On the other hand, for values of overconsolidation ratio greater than about 4 to 8, the volume tends to increase, the value of u_w correspondingly decreases, and the undrained strength exceeds the drained value. For high overconsolidation ratios the excess may be very large. However, the strong negative pore pressures associated with high overconsolidation ratios tend to draw water into the soil and cause it to swell, whereupon the strength is reduced. For this reason the undrained strength often cannot be depended upon. Moreover, in most practical problems the attempt to apply the $\phi = 0$ concept for an overconsolidated clay would lead to results on the unsafe side, whereas for a normally loaded clay the tendency toward consolidation would lead to errors in the conservative direction. Hence, except for overconsolidation ratios as low as possibly 2 to 4, the $\phi = 0$ concept should not be used for preloaded clays.

Heavily overconsolidated clays and clay shales are likely to exhibit high peak strengths even when tested under fully drained conditions because of the strong bonds that have developed between the particles

(Article 49). However, after a surface of sliding forms and extensive slip occurs, the bonds are destroyed and the particles along the surface of sliding assume an orientation favorable to a low resistance to shear along the surface. The ultimate shearing resistance after very large displacements under fully drained conditions is known as the *residual strength* (Skempton 1964). It cannot be investigated in conventional triaxial tests because the amount of slip in such tests is restricted; special direct shear or torsional shear devices are required (Haefeli 1950). The residual shear strength may be expressed as

$$s_r = \bar{p} \tan \phi_r \tag{18.8}$$

where ϕ_r varies from about 30° for clays having low plasticity indices and a small clay-size fraction to as little as 5° to 12° for some highly plastic clays with a large percentage of clay-size particles (<0.002 mm). Because of the nearly complete destruction of the structure of the natural clay along the surface of sliding, it is likely that the values of ϕ_r are the same irrespective of the stress-history of the clay, and can be determined with sufficient accuracy on remolded specimens (Skempton 1964).

Fissured Overconsolidated Clays

The continuity of heavily overconsolidated clays is commonly disrupted by a network of hair cracks. If the average pressure in such clays has been reduced, either by excavation or by geological processes such as erosion, the shearing resistance decreases at constant shearing stress; it may ultimately become as small as 0.2 ton/ft² irrespective of its original value. Therefore, the failure of slopes in open cuts underlain by such materials may occur many years after the cut is made.

The mechanics of the process of softening is explained in Article 49. At any given time the shearing resistance of the clay increases rapidly with increasing depth below the surface. After a slide occurs the material underlying the newly exposed surface begins to soften and the process continues until another slide occurs. Hence, the side slopes of valleys located in such clays are subject to intermittent superficial landslides from the time the valleys originate; the process does not stop until the slope angle becomes compatible with the softest consistency the clay can acquire. Thus the slopes become gentler. In some regions, such as the valley of the Saskatchewan River south of Saskatoon in Canada, slides still occur without provocation on slopes rising at 1 vertical on 15 horizontal. The problem of determining the shear characteristics of such clays for design purposes has not yet been solved (Peterson et al. 1960).

The coefficient c_v represents the coefficient of consolidation (Eq. 14.2). Hence,

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \tag{25.7}$$

The solution of this equation must satisfy the hydraulic boundary conditions. These conditions depend on the loading and drainage conditions as shown in the diagrams in Fig. 25.2. The boundary conditions that determine the consolidation of a half-closed layer and a uniform pressure distribution may serve as an example. According to Fig. 25.3*b*, the boundary conditions are as follows:

1. At $t = 0$ and at any distance z from the impervious surface, the excess hydrostatic pressure is equal to Δp.
2. At any time t at the drainage surface $z = H$, the excess hydrostatic pressure is zero.
3. At any time t at the impervious surface $z = 0$, the hydraulic gradient is zero (that is $\partial u/\partial z = 0$).
4. After a very great time, at any value of z, the excess hydrostatic pressure is zero.

Equation 25.7 combined with the boundary conditions determines the degree of consolidation $U\%$ for a given time t. The equation for $U\%$ is

$$U\% = f(T_v) \tag{25.8}$$

In this expression,

$$T_v = \frac{c_v}{H^2} t \tag{25.9}$$

is a pure number called the *time factor*. Since the soil constants and the thickness of the compressible layer enter Eq. 25.8 only in the combination represented by the dimensionless time factor T_v, the value $U\% = f(T_v)$ is the same for every layer that consolidates under specified conditions of loading and drainage. It has been determined for every condition of practical importance by means of the differential Eq. 25.7. The results have been presented in the form of graphs or tables. By means of these graphs and tables, all the problems likely to be met in practice can be solved without any computation other than the evaluation of Eq. 25.9. Figure 25.4 represents the solutions of the problems illustrated in Fig. 25.2. The following instructions serve as a guide for using the graphs.

For every open layer (thickness $2H$) the relationship between $U\%$ and T_v is determined by the curve C_1, regardless of the slope of the zero isochrone *de*. Therefore, the curve C_1 represents the solution for all the consolidation problems represented by Fig. 25.2*a*, *b*, *c*, and *e*. If the zero isochrone is horizontal, indicating a uniform distribution of the consolidation pressure throughout the consolidating layer, curve C_1 also represents the process of consolidation for a half-closed layer with thickness H. The following example illustrates the procedure for using the graph (Fig. 25.4*a*).

The coefficient of consolidation of an open layer with thickness $2H$ is c_v. We wish to determine the time t at which the degree of consolidation of the layer due to the weight of a superimposed building becomes equal to 60%. From Eq. 25.9 we obtain

$$t = T_v \frac{H^2}{c_v}$$

According to curve C_1 in Fig. 25.4*a*, a degree of consolidation of 60% corresponds to the time factor 0.28, whence

$$t = 0.28 \frac{H^2}{c_v} \tag{25.10}$$

regardless of the slope of the zero isochrone. If the zero isochrone for a half-closed layer of clay with thickness H is horizontal, the degree of consolidation of this layer after time t (Eq. 25.10) will also be equal to 60%.

If the consolidation pressure for a half-closed layer decreases from some value Δp_t at the top to zero at the bottom, as shown in Fig. 25.2*d*, the relation between U and T_v is given by the curve C_2. If it increases from zero at the top to Δp_b at the bottom, as in Fig. 25.2*f*, curve C_3 furnishes the required information. For intermediate types of vertical distribution of consolidation pressure, sufficiently accurate results can be obtained by interpolation. Figure 25.4*b* shows the curves C_1 to C_3 plotted to a semilogarithmic scale. Small values of U can be obtained somewhat more accurately from the semilogarithmic curves. The semilogarithmic plot of C_1 corresponds to the solid curve in Fig. 14.2*b*.

Because of the simplifying assumptions listed at the outset of the preceding analysis, the computation of the rate of settlement has the character of a crude estimate. The most important discrepancy between theory and reality has been referred to as the secondary time effect (Article 14). According to the theory of consolidation, the time–settlement curve should approach a horizontal asymptote whereas in reality it merges into an inclined tangent as shown in Fig. 14.2*a*. At present the secondary settlement cannot be predicted reliably on the basis of test results. Experience shows that the rate of the secondary settlement of

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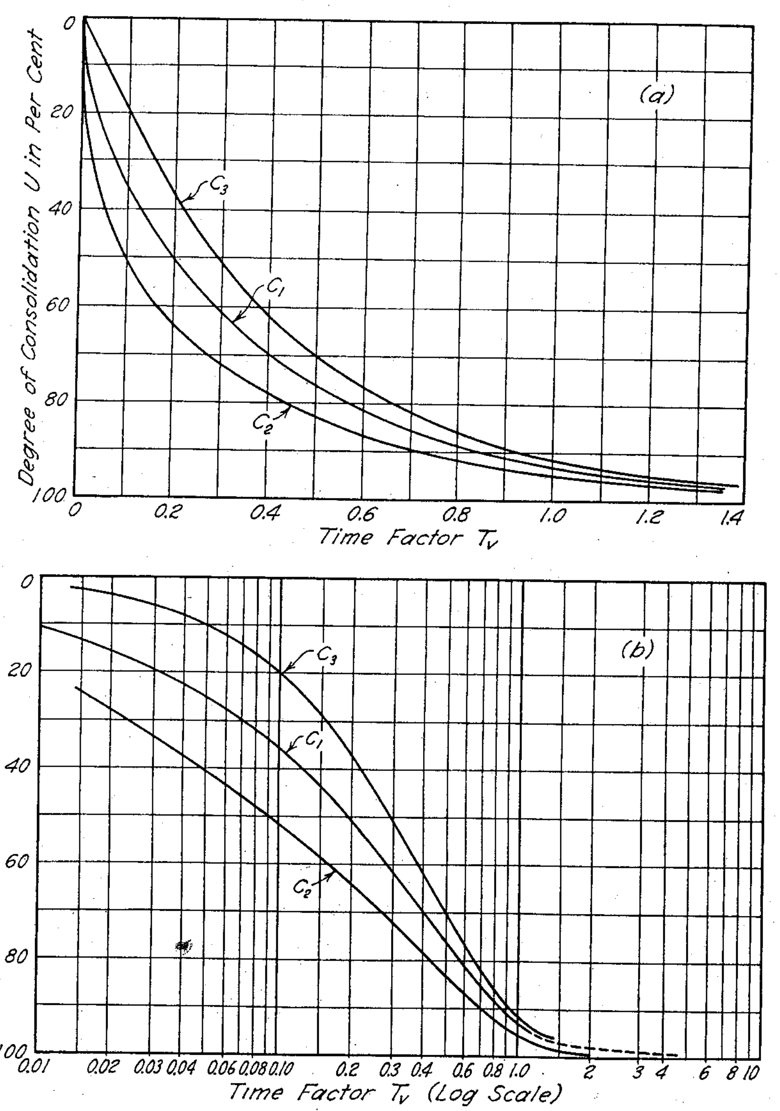

Fig. 25.4. Relation between time factor and degree of consolidation. In (*a*) the time factor is plotted to an arithmetic and in (*b*) to a logarithmic scale. The curves C_1, C_2, C_3, correspond to different conditions of loading and drainage, represented by *a*, *d*, and *f*, respectively, in Fig. 25.2 (after Terzaghi and Fröhlich 1936).

buildings resting on normally loaded clay ranges, during the first decades after construction, between $\frac{1}{8}$ and $\frac{1}{2}$ in per year. Exceptional rates as high as one inch per year have been observed.

It is obvious that the results of a settlement computation are not even approximately correct unless the assumed hydraulic boundary conditions are in accordance with the drainage conditions in the field. Every continuous sand or silt seam located within a bed of clay acts like a drainage layer and accelerates consolidation of the clay, whereas lenses of sand and silt have no effect. If the test boring records indicate that a bed of clay contains partings of sand or silt, the engineer is commonly unable to find out whether or not these partings are continuous. In such instances the theory of consolidation can be used only for determining an upper and a lower limiting value for the rate of settlement. The real rate remains unknown until it is observed.

Furthermore, in reality water escapes from the clay beneath a loaded foundation not only in vertical directions, but also by flow in horizontal or inclined directions. Problems of three-dimensional consolidation have been solved for relatively simple boundary and stress conditions (Biot 1941, Gibson and McNamee 1963). For more complex conditions, solutions can be obtained by numerical procedures (Abbott 1960, Gibson and Lumb 1953).

Problems

1. Representative samples were obtained from a layer of clay 20 ft thick, located between two layers of sand. By means of consolidation tests, it was found that the average value of c_v for these samples was 4.92×10^{-4} cm^2/sec. By constructing a building above the layer, the average vertical pressure in the layer was everywhere increased and the building began to settle. Within how many days did half the ultimate settlement occur?

Ans. 438 days.

2. If the clay layer in problem 1 contained a thin drainage layer located 5 ft below its upper surface, how many days would be required to attain half the ultimate settlement?

Ans. 127 days.

3. A layer of clay 30 ft thick rests on an impermeable rock base. The consolidation stress along a given vertical line is assumed to vary uniformly from a maximum at the top of the layer to zero at the rock surface. The value for c_v for the clay is 9.5×10^{-5} cm^2/sec. How many years will elapse after the construction of a building until the settlement becomes equal to 30% of the final value? Solve the same problem on the assumption that the clay rests on a pervious sand bed instead of rock.

Ans. 6.5; 4.9 years.

ratio of horizontal to vertical permeability can be judged on the basis of Eqs. 11.10 and 11.11.

Elaborate investigations of this nature are rarely justified economically. The determination of the permeability of natural deposits below the water table by *in situ* permeability tests is always more reliable than that by laboratory tests.

Procedures have been developed to evaluate the permeability of sand strata located above the water table on the basis of the quantity of water that percolates into the soil from that part of a drill hole extending below the casing. The results are at best no more than crude estimates and may be unreliable, because the flow pattern into the soil remains unknown and the formation of a filter skin at the entrance surface can hardly be avoided. The procedure (Zangar 1953) is similar to that described in connection with *in-situ* permeability tests in drill holes below the water table.

Shearing Resistance of Saturated Clays

If a project involving clay soils calls for an investigation of the stability of slopes, the computation of the lateral pressure against the bracing of open cuts, or an estimate of the ultimate bearing capacity of footings or rafts, the shearing resistance of the clay must be determined. If the water content of the clay will not change significantly during the period when the slopes are unsupported or during the lifetime of the temporary bracing of the open cuts, or if the factor of safety of footings is a minimum before the water content can decrease on account of the loading, the $\phi = 0$ conditions are applicable (Article 18). The undrained shearing strength on the basis of total stresses is then equal to one-half the unconfined compressive strength q_u of undisturbed samples of the clay. The shearing strength can also be determined directly by means of the vane (Fig. 44.17) or torvane (Fig. 18.3). Inasmuch as many practical problems of outstanding importance fall into the $\phi = 0$ category, means for evaluating the undrained shearing strength of saturated clay soils deserve special consideration.

During the drilling of the exploratory holes the shearing resistance of the clay can be crudely estimated on the basis of the record of the standard penetration test. Table 45.2 shows the approximate relation between the unconfined compressive strength and the number of blows per foot of penetration of the sampling spoon. However, at a given number N of blows per foot, the scattering of the corresponding values of q_u from the average is very large. Therefore, compression tests should always be made on the spoon samples. The other

Table 45.2
Relation of Consistency of Clay, Number of Blows N on Sampling Spoon, and Unconfined Compressive Strength

	q_u in tons/ ft²					
Consistency	Very Soft	Soft	Medium	Stiff	Very Stiff	Hard
N	<2	2–4	4–8	8–15	15–30	>30
q_u	<0.25	0.25–0.50	0.50–1.00	1.00–2.00	2.00–4.00	>4.00

routine tests on the spoon samples, listed in Table 9.1, are also obligatory because their results are required for comparing the clay with others previously encountered on similar jobs. The values of q_u obtained by means of compression tests are likely to be somewhat too low because spoon samples are appreciably disturbed. The supplementary investigations required on important jobs depend on the character of the soil profile.

If the soil profile is simple and regular, it is commonly possible to evaluate the average shearing resistance of the clay strata on the basis of the results of laboratory tests. The samples are secured by means of tube sample borings (Article 44) which furnish continuous cores. To obtain fairly reliable average values, the spacing between the sample borings should not exceed 100 ft. If it is known in advance that the soil profile is fairly regular and that tube sample borings will be required, continuous samples are taken in all those sections of the exploratory holes that are located within clay strata. In the sections located between clay strata spoon samples are extracted, and standard penetration tests are made.

The samples are delivered to the laboratory in sealed tubes commonly 30 or 36-in. long. Preferably, all the clay samples from one hole should be tested in the sequence in which they followed each other in the drill hole in a downward direction. Each sample is ejected from its tube by means of a close-fitting plunger in such a manner that the sample continues to move with respect to the tube in the same direction as it entered; if excessive side friction causes too much disturbance during ejection, the tube is cut into 6-in. sections, the soil itself is cut by means of a wire saw, and each section is ejected.

For routine testing each sample is cut into sections with lengths equal to about three times the diameter; that is, 2-in. tube samples

Since the source of instability is the pressure of the water trapped in the sand pockets, stabilization can be accomplished by means of drainage. However, the geological profile is likely to be very irregular, and the spacing of drains may be very difficult to determine in advance even after the soil and hydraulic conditions have been thoroughly investigated by boring, testing, and periodic surveys of the water table. Under these circumstances an expedient and effective procedure may be the insertion of *horizontal auger drains* (Smith and Stafford 1957). Such drains commonly consist of perforated or slotted metal or plastic pipe of about 2-in. diameter, inserted into holes drilled nearly horizontally into the soil beneath the slope. The lengths of the drains range from a few feet to more than 200 ft. Their horizontal spacing depends on the local conditions; it often ranges from about 15 to 50 ft. Several rows at different elevations may also prove effective. The drains are usually given a small downward slope toward the face of the cut to facilitate the removal of the water by gravity.

The holes for the drains are commonly made by continuous-flight hollow-stem augers (Fig. 44.3) which permit insertion of the drain pipe without collapse of the hole. In some materials a filter may be required to prevent underground erosion and clogging; the filter material under favorable circumstances may be transported into the hole around the drain by means of the auger upon reversing its direction of rotation and gradually withdrawing it from the hole. Holes have also been formed successfully by a modification of rotary drilling wherein a casing terminating in a hollow bit is advanced by rotation while water is supplied through the interior of the casing and returns around the outside of the casing. The bit is abandoned when the hole reaches its final length, the drain is inserted, and the casing withdrawn.

The technique of installation of horizontal auger drains requires adaptation to local conditions, but such drains can often be installed so rapidly and economically that the length and spacing are established on the basis of trial. Several drains may be nonproductive, but those that encounter the pervious pockets may be remarkably effective. Once drainage has been accomplished, the terrain may become so stable that the cut can be made with standard slopes.

Slides in Stiff Clay

Almost every stiff clay is weakened by a network of hair cracks or slickensides. If the surfaces of weakness subdivide the clay into small fragments 1 in. or less in size, a slope may become unstable during construction or shortly thereafter. On the other hand, if the spacing of the joints is greater, failure may not occur until many years after the cut is made.

Slides in clay with closely spaced joints occur as soon as the shearing stresses exceed the average shearing resistance of the fissured clay. Several slides of this type took place in a long railway cut at Rosengarten, near Frankfurt in Germany. The slope of the sides was 3:1. The greatest depth of the cut was 100 ft, and the average shearing stresses along the surfaces of sliding adjoining the deepest part of the cut were roughly 10 tons/ft^2. The clay was very stiff, but large specimens broke readily into small angular pieces with shiny surfaces. Slides started immediately after construction, and continued for 15 years (Pollack 1917).

If the spacing of the joints in a clay is greater than several inches, slopes may remain stable for many years or even decades after the cut is made. The lapse of time between the excavation of the cut and the failure of the slope indicates a gradual loss of the strength of the soil. Present conceptions regarding the mechanics of the process of softening are illustrated by Fig. 49.5. Before excavation, the clay is very rigid, and the fissures are completely closed. The reduction of stress during excavation causes an expansion of the clay, and some of the fissures open. Water then enters and softens the clay adjoining these fissures. Unequal swelling produces new fissures until the larger chunks disintegrate, and the mass is transformed into a soft matrix containing hard cores. A slide occurs as soon as the shearing resistance of the weakened clay becomes too small to counteract the forces of gravity. Most slides of this type occur along toe circles involving a relatively shallow body of soil, because the shearing resistance of the clay increases rapidly with increasing distance below the exposed surface. The water seems to cause only the deterioration of the clay structure; seepage pressures appear to be of no consequence.

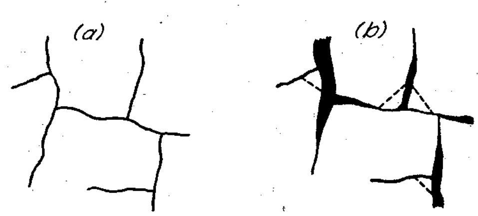

Fig. 49.5. Section through fissured stiff clay mass. (*a*) Old fissures closed before relief of stress by excavation. (*b*) Relief of stress causes fissures to open, whereupon circulating water softens clay adjoining the walls.

Fig. 49.6. Photograph of slide in very stiff fissured clay.

Figure 49.6 shows a slide in very stiff fissured clay beside a railroad cut having side slopes of 2.5:1. The height of the slope was 60 ft. The characteristic S-shape of the slope after failure is apparent. Failure occurred about 80 years after the cut was excavated. No springs or other indications of percolating water were present.

A study of the records of several delayed slides in stiff clays with widely spaced joints has shown that the average shearing resistance of the clay decreases from a high initial value at the time of excavation to values between 0.20 and 0.35 ton/ft^2 at the time of the slide. Since the process of deterioration may require many decades, it would be uneconomical to select the slope angle for the sides of cuts in such clays on the basis of the ultimate value of the shearing resistance. However, it is desirable to delay the deterioration as much as possible by draining the strip of land adjoining the upper edge of the cut for a width equal to the depth of the cut and by treating the ground surface of the cut area to reduce its permeability. Should local slides occur at a later date, they can be remedied by local repairs. If delayed slides would endanger life or cause excessive property damage, the slope should be provided with reference points and periodic observations should be made, inasmuch as slides of this type are always preceded by deformations that increase at an accelerated rate as a state of failure is approached. When the movement becomes alarming, the slopes in the danger section should be flattened.

Hard core drains have also been successfully used to prevent movements at danger sections. These drains consist of ribs of dry masonry installed in trenches running up and down the slope at a spacing of about 15 or 20 ft. The trenches are excavated to a depth somewhat greater than that to which the clay has been softened. A concrete footwall supports the lower ends of all of the ribs. The beneficial effect of this type of construction is commonly ascribed to the action of the ribs as drains, but it is more likely that the principal function of the ribs is to transfer part of the weight of the unstable mass of clay through side friction to the footwall.

The behavior of poorly bonded clay shales is governed by many of the same considerations as that of stiff clays. Hence, further information concerning slides in heavily overconsolidated clays is contained in the next section.

Slopes on Shale

From an engineering point of view, shales are of outstanding importance because they constitute about 50% of the rocks that are either exposed at the earth's surface or are buried beneath a thin veneer of sediments. All the rocks of this category consist of deposits of clay or silt that have acquired their present characteristics under the influence of relatively moderate pressures and temperatures.

As the thickness of the overburden increases from a few tens of feet to several thousands, the porosity of a clay or silt deposit decreases; an increasing number of cohesive bonds develops between particles as a result of molecular interaction, but the mineralogical composition of the particles probably remains practically unaltered. Finally, at very great depth, all the particles are connected by virtually permanent, rigid bonds that impart to the material the properties of a real rock. Yet, all the materials located between the zones of incipient and complete bonding are called shale. Therefore, the engineering properties of any shale with a given mineralogical composition may range between those of a soil and those of a real rock.

The most conspicuous differences among the shales produced by the compaction of identical sedimentary deposits have their origin in the number of permanent interparticle bonds per unit of volume of shale. A relative measure of the degree of bonding is provided by the performance of intact specimens obtained from a depth of several hundred feet. Upon submersion, all of these gradually break up into fragments. However, depending on the degree of bonding, the sizes of the fragments may be as great as a large fraction of an inch or as small as the individual mineral particles themselves. Between these limits, shales may be said to range from those categorized as well-bonded, of which the extreme types are rock-like shales, and those described as poorly bonded, of which the extreme