Foundation Engineering

SECOND EDITION

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tendency exceeds the tendency to consolidate as a result of positive pore pressures induced by the shearing stresses, the water content of the clay may eventually increase and the shear strength correspondingly decrease. These conditions are not likely to occur beneath excavations one or two stories deep in normally loaded or slightly overconsolidated clays. If they do, however, the shear strengths derived from Q-tests no longer err on the side of safety.

The $\phi = 0$ concept and the use of Q-tests would also be valid for overconsolidated clays if in the field no opportunity existed for change in water content. However, the strong negative pore pressures associated with high overconsolidation ratios create a tendency for the soil to swell, whereupon the strength is reduced. Thus, in most practical problems the $\phi = 0$ concept for an overconsolidated clay leads to results on the unsafe side. Hence, except for overconsolidation ratios as low as possibly 2 to 4, the $\phi = 0$ concept should not be used for preloaded clays.

Many stiff saturated clays contain networks of hair cracks or slickensides. The shearing strength of deposits of this kind depends on the influence of such defects. Triaxial Q-tests on large specimens that include a representative number of defects have in some instances been found useful in determining the shearing strength of the mass. The cell pressure is usually taken as the overburden pressure on the sample as it existed in the ground. More reliable data can be obtained by means of large-scale loading tests or test excavations in the field.

c/p Ratio. The $\phi = 0$ concept leads to a useful corollary. According to eq. 4.5 the strengths of normally consolidated samples are defined by the rupture line

$$s = \bar{p} \tan \phi_d \qquad 4.5$$

An effective-stress rupture circle for one of a series of undrained tests is shown in Fig. 4.10*a*. The cell pressure under which all the samples in the series were consolidated is $\bar{p}_3$. The value of s corresponding to the $\phi = 0$ concept is the radius c of the circle. It is apparent that, for samples of a given material consolidated under different confining pressures, the ratio $c/\bar{p}_3$ is a constant.

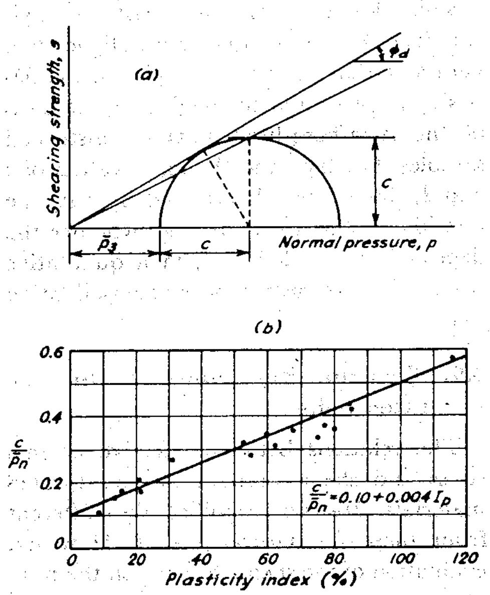

FIGURE 4.10. (*a*) Rupture diagram illustrating constancy of ratio c/p_3 for normally loaded samples of clay consolidated under different cell pressures. (*b*) Relation between ratio $c/\bar{p}_n$ and plasticity index.

In a natural deposit of normally loaded sedimented soil the consolidation stresses differ in horizontal and vertical directions. This condition introduces a complication into the interpretation, but it has nevertheless been found that a constant ratio exists between values of c determined by Q-tests and the effective vertical overburden pressure on horizontal planes. This ratio is designated as $c/\bar{p}_n$ or, for short, as the *c/p ratio*. Furthermore, a broadly valid statistical relation has been found between $c/\bar{p}_n$ and the plasticity index for normally loaded sedimentary clays (Skempton, 1948; Bjerrum and Simons, 1960). The relation is shown in Fig. 4.10*b*. It may be approximated by the equation

$$\frac{c}{\bar{p}_n} = 0.10 + 0.004 I_P \qquad 4.8$$

where I_P is expressed in per cent.

This relation is useful in at least two ways. If a deposit is known to be normally loaded, values of c for the various layers in the deposit can be estimated roughly on the basis of the Atterberg-limit tests on disturbed samples. On the other hand, if values of c and I_P have been determined by test, the relation can be used to judge whether the deposit is preloaded and, in a qualitative way, what the degree of overconsolidation may be.

4.9. Shearing Resistance of Unsaturated Soils

The relations between effective normal stress and shear strength for unsaturated materials are not significantly different from those for saturated soils. However, evaluation of the shear strength on the basis of these relations requires a knowledge of the pore pressure not only in the water contained in the voids but also in the air that occupies the remainder of the voids. The pore-air pressure and the porewater pressure may have quite different values on account of the surface tension at the air-water interfaces. Because of the difficulties in evaluating these pressures, it is current practice to investigate the strength of partly saturated soils by means of triaxial tests in which only total stresses are measured and in which the laboratory test conditions are made to duplicate, as closely as possible, those anticipated in the field. In many instances Q-tests are appropriate. The water content of each sample is kept constant. Volume changes occur, nevertheless, because of the compression of the air in the voids.

Typical results of several series of Q-tests on samples of an inorganic clay (CL) are shown in Fig. 4.11 (Casagrande and Hirshfeld, 1960). All samples were compacted to the same dry density. The rupture line for samples having a relatively low initial degree of saturation is markedly curved. For increasingly greater initial degrees of saturation, the strengths decrease. Moreover, for a given initial degree of saturation, increases in pressure cause compression of the air in the voids and, in addition, increase the solubility of air in water. Consequently, the

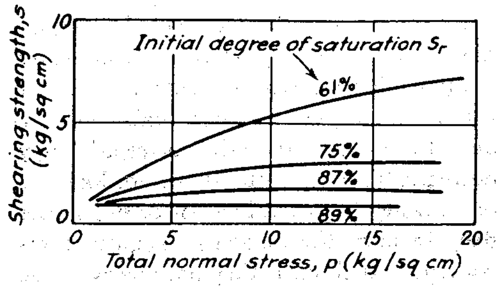

FIGURE 4.11. Results of Q-tests on partially saturated samples of an inorganic clay compacted to equal dry densities.

degree of saturation increases. For those samples with high initial degrees of saturation, S_r may reach 100 per cent at a comparatively low pressure, whereupon the $\phi = 0$ conditions are satisfied and the rupture line with respect to total stresses becomes horizontal.

A compacted fill is ordinarily placed at a moisture content close to the optimum value; this value corresponds to a partially saturated condition. The strength at the

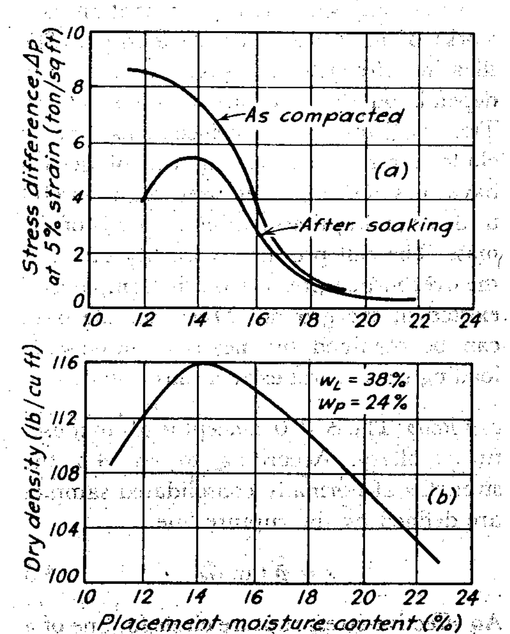

FIGURE 4.12. (a) Results of Q-tests on samples of a compacted silty clay tested as compacted and after soaking. (b) Standard Proctor moisture-density curve for material.