Pore Pressure and Strength of Underconsolidated Clay Soils

Pression interstitielle et résistance au cisaillement des sols argileux sous-consolidés

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SUMMARY

The efficiency of the compacting effect of natural loads and pressures under laboratory conditions on clay soils depends greatly upon the resistance offered by cementation bonds between soil particles. The formation of these bonds at early stages of sedimentation of clay deposits and their subsequent preservation leads to the formation of underconsolidated soils. Different kinds of loesses, permafrost soils, and post-glacial quick clays of sea and lake origin belong to this category of soils. The degree of sensitivity of such soils depends on the degree of their underconsolidation. A characteristic feature of underconsolidated soils is their ability to be further consolidated under constant stresses as a result of the destruction of their natural structure alone. Pore pressure in such soils may spring up and increase not only as a result of the external pressures becoming greater but also after removing the effects of the cementation bonds at constant pressure. The pore-pressure parameter for underconsolidated soils, A_f, may be greater than unity. The peculiarity of the process of the development of the pore pressure in underconsolidated soils gives evidence of this pressure exerting no direct effect on the strength of clay soils. The value of the pore pressure is an indicator of the degree of the discrepancy between the density and strength of clay soils, on one hand, and the value of pressure applied to them, on the other; it indicates also the degree of removing the effects of the cementation bonds in such soils (the degree of peptization). The value of the pore pressure shows what proportion of the total pressure is not balanced by the resistance offered by the bonds between soil particles.

SOMMAIRE

La consolidation des sols argileux sous l'effet des charges naturelles et sous les pressions développées en laboratoire dépend considérablement de la résistance provoquée par des cohésions de cimentation entre les particules. La formation de ces liaisons au premier stade de la sédimentation des dépôts argileux et leur conservation subséquente amène la création de sols non consolidés. Les différentes sortes de loess et les sols congélés ainsi que les dépôts postglaciaires des argiles marines appartiennent à cette catégorie de sols. Le degré de sensibilité de ces sols dépend de leur degré de consolidation insuffisant. Ces sols ont une particularité caractéristique: ils peuvent continuer leur consolidation supplémentaire dans des conditions de contrainte constante si la destruction de la structure naturelle a lieu. La pression de l'eau interstitielle peut se former et s'accroître par suite de la pression extérieure, de même qu'après l'élimination de l'influence des cohésions de cimentation sous pression constante. Le paramètre de la pression de l'eau interstitielle A_f pour des sols qui ne sont pas assez consolidés peut dépasser considérablement l'unité. Les particularités de ce processus de développement de la pression de l'eau interstitielle dans ces sols prouvent que cette pression n'a pas d'influence directe sur la résistance des sols argileux. La pression de l'eau interstitielle est un indice d'une différence de densité et de résistance dans le sol, ainsi que de l'ordre de grandeur de la pression qui leur est appliquée. Cette pression indique de plus la perte de l'effet des liens de cimentation (le degré de "peptization").

La valeur de la pression de l'eau interstitielle montre quelle partie de la pression totale n'est pas équilibrée par la résistance offerte par la cohésion de cimentation entre les particules de sol.

SOILS WITH HIGHLY SENSITIVE STRUCTURES attract the attention of many investigators (Bjerrum, 1961; Crawford, 1963; Denissov, 1963; Gorkova, 1963; Keinonen, 1963; Simons, 1960). Because they drastically decrease their strength when their natural structure is destroyed and because they have some anomalies of development of the pore pressure which are reflected in the value of the pore-pressure parameter, A_f, being greater than unity. According to present ideas the strength of clay soils is governed both by attraction forces between the particles (Van der Waals' and London's forces) and by cementation bonds (Denissov, 1946, 1956; Rosenqvist, 1959; Seed, Mitchell, and Chan, 1960; Bjerrum and Lo, 1963; Denissov and Reltov, 1957, 1961).

The effect of the interparticle attraction forces and the strength of clay soils caused by this effect depends upon their density. The particular significance of these forces becomes clear when one takes into consideration that the effect of the cementation bonds during the period of existence of soils may be greatly diminished by the peptizing action of surrounding media—low mineralized waters (loesses and different clay soils), rising temperature (permafrost), and by remoulding of the soils. The effect of the interparticle attraction forces is not affected by the surrounding media to the same significant degree. The strength of clay soils caused by these forces may diminish only when the density of the soils decreases.

In evaluating clay soils for construction purposes, the data concerning their strength caused by the attraction forces are of great interest. Therefore the data concerning natural consolidation of clay soils, that is the degree of their consolidation as compared with the state of a recently formed sediment, are significant. The state of a clay sediment with the moisture content (density) corresponding to the liquid limit, w_L, may be taken as a standard of such initial state. The void ratio for this state may be calculated from the values of the liquid limit, w_L, and the specific gravity, γ:

$$e_L = w_L \gamma / 100. \qquad (1)$$

Laboratory investigations show that clay soils of a very high initial moisture content, greatly exceeding the value of w_L, will have a moisture content lower than w_L at pressures of the order of only 0.8 to 1.0 kg/sq.cm.

pression curves for samples of such soils with undisturbed

Comparison of the value of the void ratio for a soil in its natural condition, e_0, with that of e_L, assumed to be the standard of the initial state, may be accomplished by means of the coefficient of natural consolidation, K_c:

$$K_c = e_L/e_0. \tag{2}$$

The lower the value of K_c as compared with unity, the less consolidated is the soil, and *vice versa*—the values of K_c that are much greater than unity indicate significant consolidation of the soil.

Quick clays in Norway and some other countries with the characteristic K_c values of the order of 0.7 to 0.9 are usually referred to as normally consolidated. The data relating to the geological history of the areas of their occurrence are taken as the formal basis of this statement; these data give evidence that the clays were never subjected to natural pressures greater than now. Since such soils, like loesses, may be further consolidated after their structure is destroyed (at a constant stress state), and consequently must be distinguished as a special category of underconsolidated soils (Denissov, 1946, 1961).

When considering the reasons for the low degree of consolidation in many kinds of clay soils, especially quaternary ones, one must consider that in some cases this low degree of consolidation may reflect only the absence of the effect of any substantial pressures during the whole period of the existence of the soil. This fact explains why the values of K_c for the upper horizon of silts do not usually exceed 0.6 to 0.8. There are, however, well-known clay soils which have the same values of K_c but are subject to natural pressures exceeding 1.5 to 2.0 kg/sq.cm. Different kinds of loesses and permafrost soils which are capable of being further consolidated when moistened and thawed respectively, and post-glacial clays of sea and lake origin belong to this category of soils. The reason for the extremely weak reaction of the above-mentioned types of deposits to the pressure of overburden may lie solely in some peculiarities of the process of their natural consolidation.

First of all, it is to be noted that for incomplete natural consolidation of the types of deposits in question cannot be justified in terms of insufficient time. Had such an explanation been correct, considerable differences in strength between samples with undisturbed structure and those in the remoulded state which are characteristic of all the soils having low values of K_c, could not have occurred so frequently.

The value of K_c of the order of 0.6 to 0.8 for soils subjected to great pressures indicates that such soils retain high porosity which is a characteristic of recently precipitated deposits. The conditions of density and moisture content which correspond to very low pressures proved to be preserved for the whole subsequent period of time when natural pressure had been increasing. Such "preservation" of high moisture content might be only a consequence of the effects of the cementation bonds which after their emergence lowered the efficiency of the natural pressure (which had always been increasing) towards zero.

To confirm the significance of the cementation bonds we refer to the well-known phenomenon of frozen soils retaining low density under fairly high pressures (up to 2 to 3 kg/sq.cm. and higher). Ice plays the role of a cement in such soils. The absence of compaction under the action of rising pressures is characteristic for the region of low pressures as well as for loesses and post-glacial clays. Com-

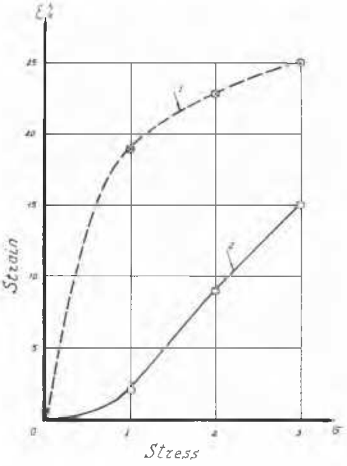

FIG. 1. Consolidation curves for a post-glacial clay in remoulded (Curve 1) and undisturbed (Curve 2) states.

structure have a distinctive bulge directed upward, initial parts of these curves often being horizontal. The compression curves slope more gently as pressures increase; this is characteristic only of remoulded samples of such soils. Fig. 1 is given as an illustration: curve 1 shows the relation between strain and pressure for a post-glacial clay in the remoulded state; curve 2 shows the same relation for the same post-glacial clay with undisturbed structure. It is to be noted, that as the pressure increases the slope of the curve for the sample with undisturbed structure becomes steeper, and that for the remoulded sample becomes gentler, so that the two curves approach one another. Since the paste is in the maximum peptized state, and the sample with undisturbed structure is in the aggregate state, the above-mentioned approach of the two curves may be explained by a higher degree of peptization of the soils due to a pressure rise. The degree of peptization may also rise without any rise of pressure, however, due to the action of low mineralized waters (post-glacial clays), increased moisture content (loesses), higher temperatures (permafrost), and remoulding.

The effects of the peculiarities in the natural structure of clay soils may result not only in the above-mentioned differences in the rate of the straining of samples with undisturbed structure and in the remoulded state, but may also bring about differences in the pore pressure which develop while loading undisturbed and remoulded samples of some clays. Tests with Norwegian post-glacial clays showed that the pore-pressure parameter, A_f (Skempton, 1954) for remoulded samples does not usually exceed unity, whereas that for samples of the same clays with undisturbed structure may often be well over unity (Bjerrum, 1961; Simons, 1960).

To understand the reasons for the pore pressure anomaly during the loading of soil samples, changes in the state of the clay soils undergoing such loading must be considered. In shear the density of clay as well as sandy soils approaches the state corresponding to the critical void ratio or critical density. The critical density is the flow density at which the volume and the shape of pores conform to the shape of mineral particles displacing them. Zero value of the pore-pressure parameter, A_f, corresponds to this state. Soils with densities lower than critical, that is normally consolidated and underconsolidated soils, are further consolidated when subject to shear, surplus pore pressure ($A_f > 0$) arising in them. If the density of a soil exceeds the critical density, it swells under shear, with negative pore pressure ($A_f < 0$) arising in it.

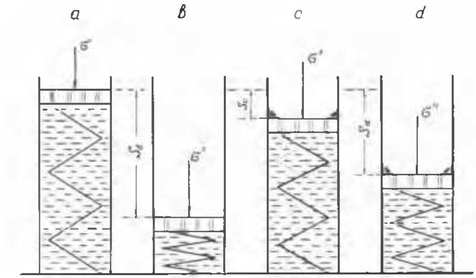

FIG. 2. Mechanical models of clay soils in different states: (a) initial state; (b) normally consolidated state at the pressure σ''; (c) and (d) underconsolidated state (black triangles show oxide film representing cementation bonds).

The development of pore pressure during the loading of soil samples can be illustrated by a simple mechanical model. A model of a clay soil is given in Fig. 2a. By compressing the spring (consolidation of soil) the sample is subjected to a low pressure σ'. Increasing the external pressure to σ'' results in the springing up of pressure in the water equal to $\sigma'' - \sigma'$; under the compression the spring is shortened by the value of S_b (Fig. 2b). In this state, the model illustrates a normally consolidated soil (Fig. 2a), which is further consolidated (Fig. 2b), provided there are no detaining agents like cementation bonds.

To show the effects of such bonds, assume that by the time the piston drops through the distance S_c (S_c is much smaller than S_b), an oxide film is formed on the surface of the metal (triangles in Fig. 2c) which attaches the piston to the walls of the cylinder. The pressure in the water will drop to zero, and the model (Fig. 2c) will illustrate the underconsolidated state of the soil, the oxide film representing cementation bonds. The degree of underconsolidation of the soil, corresponding to the model (Fig. 2c), may be described by the ratio:

$$D_{uc} = (S_b - S_c)/S_b. \quad (3)$$

The spring of the model (Fig. 2c) retains a potential ability to be further compressed after the removal of the effects of the oxide film (cementation bonds in a soil). The removal of the oxide film may be accomplished either by its intentional destruction by acid, for example (corresponding to the destruction of the cementation bonds in loesses by water, and in frozen soils by heat), or by increasing the pressure.

In the first case, after the destruction of the film in the model (cementation bonds in soils) the water is subject to a pressure the value of which depends upon the degree of the preceding underconsolidation:

$$\Delta u = [(S_b - S_c)/S_b](\sigma'' - \sigma'). \quad (4)$$

In the second case the destruction of the film in the model (the cementation bonds in soils) may take place due to the external pressure having reached some value, σ''', sufficient to cause the destruction. At the moment of destruction the pore pressure will be equal to

$$\Delta u = (\sigma''' - \sigma'') + [(S_b - S_c)/S_b](\sigma'' - \sigma'), \quad (5)$$

i.e., will be much greater than the increment of the stress ($\sigma''' - \sigma''$). Thus, the pore-pressure parameter, A_f, will be equal to

$$A_f = \frac{(\sigma''' - \sigma'') + [(S_b - S_c)/S_b](\sigma'' - \sigma')}{\sigma''' - \sigma''} = 1 + \frac{S_b - S_c}{S_b} \cdot \frac{\sigma'' - \sigma'}{\sigma''' - \sigma''}. \quad (6)$$

The fact that the parameter A_f exceeds unity when post-glacial clay samples are loaded is connected with "mobilization" of the effects of previous stages of the development of pressure. Such a condition may arise when a sample before loading is in the state of underconsolidation due to the effects of the cementation bonds. The destruction of the bonds opposing external pressure takes place under shear

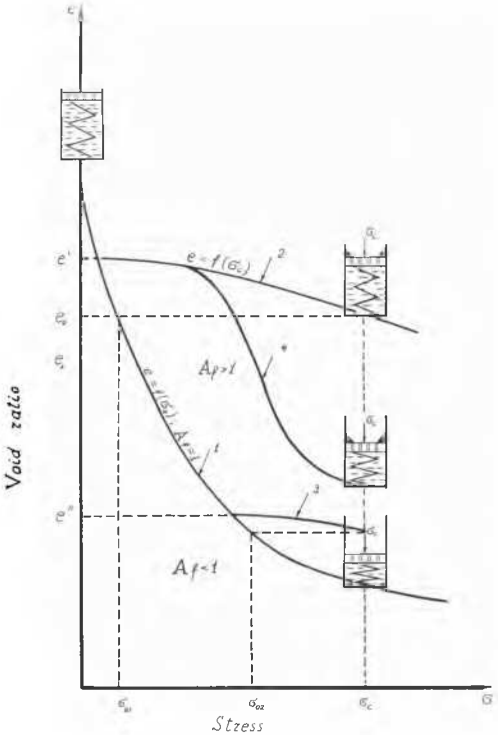

FIG. 3. Curves for natural consolidation of clay sediments: 1, without any influence of the cementation bonds (normal consolidation curve); 2, 3, 4, with the effects of the cementation bonds.

and consequently causes the pore pressure to rise at this time.

After the destruction of the oxide film in the model, roughness may develop along the surfaces of contact of the piston and the cylinder; as a result the piston may drop only through a distance $S_d < S_b$ (Fig. 2d). Under these circumstances the model (Fig. 2d) will represent a clay soil whose cementation bonds are not destroyed completely, i.e., an incompletely peptized soil. As a result the soil retains, to a certain degree, its state of underconsolidation.

The value of the pore pressure depends not only on the degree of under- or overconsolidation, but also on the relation between the resistance to filtration of water and the resistance to displacement of particles along their surfaces of contact. If the resistance to filtration of water is great, the value of the pore pressure will be relatively great. If the resistance to filtration of water is low, and the resistance along the contact surfaces of particles is great, the value of the pore pressure will be negligible, if any. It is in the case of such a relation of the resistances that so-called secondary consolidation of soils takes place. This relationship of resistances causes the role of the secondary consolidation to become more important when pressure increases, with the value of pressure stages decreasing (Leonards and Girault, 1961).

The foregoing ideas concerning the significance of the cementation bonds and the effects of their destruction upon the development of the pore pressure may be used in calculating the value of the pore pressure. Assume that a soil in its natural condition is underconsolidated (or is such by the completion of preliminary consolidation in carrying out undrained-consolidation tests), and under the pressure σ_c it has a void ratio of e_0 (Fig. 3). The cementation bonds and the interparticle attraction forces resist the pressure. After complete destruction of the soil structure and the removal of the effects of the cementation bonds the strength of the soil will be reduced to a value corresponding to the pressure σ_{01}. If the cementation bonds are partly retained, the strength of the soil will be of the value of σ_0, intermediate between σ_{01} and σ_c.

Thus, the pore pressure will be:

$$u = \alpha\beta(\sigma_c - \sigma_{01}), \quad (7)$$

where

α is a coefficient describing the condition of drainage and the relation between the filtration resistance and the contact resistance. When there is no drainage and the filtration resistance is great, $\alpha = 1$; when drainage is good, and the filtration resistance is low, $\alpha = 0$.

β is a coefficient describing the degree of the removal of the effects of the cementation bonds (the degree of the peptization of soil). When these bonds are completely destroyed, $\beta = 1$; when they are completely retained, $\beta = 0$.

The possibility of a rise of the pore pressure due to the destruction of the natural structure of underconsolidated clay soils disproves the widespread idea concerning the effects of pore pressure on the strength of clay soils. The values of the pore pressure must be considered (taking into account the above-mentioned coefficients α and β) to be indicators of the degree of the discrepancy between the density and strength of clay soils, on one hand, and the values of pressures applied to them, on the other. Such an approach to the problem is justified in particular by the results of the tests (Lo, 1961) which proved the dependence of the pore pressure on the degree of straining the soils.

The effects of the cementation bonds on the process of the formation of the properties of clays, including their underconsolidation, is illustrated in Fig. 3. Curve 1 corresponds to a clay sediment having no cementation bonds. This curve has a conventional shape characteristic of the process of normal consolidation. Curve 2 illustrates natural consolidation of a clay sediment in which considerable cementation bonds (including thixotropic ones) sprang up at the earliest stage of its formation when the void ratio e' greatly exceeded the void ratio e_L, at the liquid limit. By the completion of consolidation under natural pressure σ_c, the void ratio e_0 is again greater than e_L. The strength of the clay soil in such a condition either corresponds to the pressure σ_c or exceeds the strength corresponding to the latter pressure (due to the cementation bonds). After the destruction of these bonds, the strength of the soil will, in accordance with Curve 1, correspond to a substantially lower pressure σ_{01}, and may be manifested in liquefaction of the soil. The ratio σ_c/σ_{01} may be considered one of the criteria of the sensitivity of clay soils.

When the cementation bonds form after the soil has consolidated to the state corresponding to the void ratio e'' (e'' is much smaller than e_L) natural consolidation under pressure rising to the value of σ_c will be illustrated by Curve 3. If the natural structure of clay is destroyed under such conditions, only a comparatively insignificant decrease of strength will result. After this the strength will correspond to the pressure σ_{02} which is many times greater than σ_{01}. Any possibility of liquefaction of soils under such conditions is out of the question.

For cases in which the cementation bonds sprang up at early stages of the formation of deposits and are of relatively low strength, natural consolidation is illustrated by Curve 4, which gradually approaches Curve 1.

In accordance with Fig. 3 sensitivity of clays grows abruptly with the diminishing of the natural consolidation coefficient K_c, i.e., with the increase of the degree of their underconsolidation. Taking account of the above discussion including relation (7), the Skempton's formula for computation of the pore-pressure parameter, A_f, by measured values of the increment of the pore pressure

$$A_f = \Delta u/(\sigma_1 - \sigma_3) \quad (8)$$

may be represented as follows:

$$A_f = (\sigma_1 - \sigma_0)/(\sigma_1 - \sigma_3), \quad (9)$$

where σ_1 and σ_3 are respectively maximum and minimum principal stresses when a soil is being tested, σ_0 is the pressure corresponding to the density of a soil in accordance with the curve for normal consolidation (of the type of Curve 1, Fig. 3).

Eq (8) indicates that, assuming $\alpha = 1$ for undrained conditions, and $\beta = 1$ for complete peptization of a soil, the value of A_f depends on the relation between σ_0 and the two principal stresses σ_1 and σ_3, i.e., it indicates to what degree the density and the strength of a soil correspond with these principal stresses.

Consider first the significance of the relation between σ_0 and the minimum principal stress σ_3. Their equality ($\sigma_0 = \sigma_3$) is characteristic of the normal density of a soil (with respect to the pressure σ_3), A_f being equal to unity for this condition. When $\sigma_0 < \sigma_3$, soils are underconsolidated, and $A_f > 1$. When $\sigma_0 > \sigma_3$, soils are overconsolidated (with respect to the pressure σ_3), and $A_f < 1$.

Soils overconsolidated with respect to the pressure σ_3 may be either underconsolidated or normally consolidated, or overconsolidated with respect to the maximum principal stress σ_1. When they are underconsolidated, $\sigma_0 < \sigma_1$, and A_f may be of any value within the interval of 0 to 1. When they are normally consolidated, $\sigma_0 = \sigma_1$, and $A_f = 0$ (the state of critical density). When they are overconsolidated, $\sigma_0 > \sigma_1$, and $A_f < 0$.

Thus, one and the same soil may have different values of A_f (negative for relatively low values of σ_3, positive for relatively high values of σ_3), depending on the stress state when a soil is being sheared, that is, depending on the value of the minimum principal stress σ_3.

The above discussion justifies the expediency of taking into account, when appreciating soils, not only the history of their stressed state (as is usually the practice when classifying soils into normally consolidated and overconsolidated ones), but also the degree of the efficiency of the effects of stresses upon the process of consolidation. It is reasonable to consider clay soils, consolidation of which under natural as well as laboratory conditions proceeds efficiently due to the absence of the effects of the cementation bonds, to be normally consolidated soils. The efficiency of pressure under such a condition may be taken as equal to unity. The stress-strain curves for such soils are of the conventional gentle-sloping character and have no sharp bends. The destruction of the natural structure of such soils under a constant stress state cannot be accompanied by any diminution in their volume. The pore-pressure parameter, A_f, for these soils is equal to unity.

Underconsolidation is characteristic of soils, the process of consolidation of which proceeded under the effects of the cementation bonds which reduce the efficiency of consolidation as well as the efficiency of pressure. The stress-strain curves for such soils are of another character; their slopes become steeper with the increase of pressure, their initial portions being often horizontal. Underconsolidated soils are capable of decreasing their volume after the destruction of their natural structure under constant stress state. The pore-pressure parameter, A_f, for such soils is greater than unity.

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