PORE PRESSURES DURING THE CONSOLIDATION OF HIGHLY COMPRESSIBLE SOILS

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SUMMARY

Pore-water pressure measurements made during consolidation tests on saturated, highly compressible soils, have demonstrated that the pore pressure ratio decreases with increasing load and with increasing preconsolidation time. Consolidation tests, in which the preceding load had acted for a period of 10 days, showed pore pressure ratios that were appreciably less for the following load increment than did corresponding tests with only one day of preconsolidation time. The time required for pore pressure mobilization increases with rising load. A phenomenon similar to the Mandel-Cryer effect is thus observed, manifested as an increase in pore pressure during the early stage of consolidation.

Pore pressure measurements made during simultaneous consolidation on two and three different soil layers have indicated that the rate of pore pressure dissipation is largely dependent upon the properties of the soil layer adjacent to the drainage boundary. The progress of consolidation is accordingly dominated by the permeabili-

ty and the coefficient of volume compressibility of the soil material close to the drainage layer; moreover, the time-dependence of the coefficient of consolidation greatly influences the rate of consolidation in highly compressible soils.

INTRODUCTION

The pore-water pressure increment Δu, resulting from a change in the loading conditions of a saturated fine-grained soil, can be expressed by the formula (1) (Zur & Wiseman 1969)

$$\Delta u = \beta \Delta\sigma_{oct} + \alpha \Delta\tau_{oct} \qquad (1)$$

where α and β are pore pressure coefficients dependent upon the properties of the soil, and $\Delta\sigma_{oct}$ and $\Delta\tau_{oct}$ are the octahedral normal stress and shear stress increments.

If the soil is only partly saturated with water, the increase in pore-water pressure will be less than that in a fully saturated soil. Below the preconsolidation load, the pore-water pressure is generally small, and similar effects may be induced by quasi-preconsolidation resulting, for example, from secondary settlements or cementing bonds between soil particles. In overconsolidated soils, consequently, an increase in load generally results in no more than small changes in the pore pressure.

In a triaxial test with axial symmetry, the principal stress increment $\Delta\sigma_2 = \Delta\sigma_3$, and

$$\Delta u = \beta \frac{\Delta\sigma_1 + 2\Delta\sigma_3}{3} + \sqrt{2}\,\alpha(\Delta\sigma_1 - \Delta\sigma_3) \qquad (2)$$

In anisotropic soils, the pore pressure is further dependent upon the compressibilities in the direction of the three principal stresses, and upon the dilatation coefficient of the soil. As the stress state during sedimentation and consolidation in nature is generally anisotropic, the pore-water pressure is often dependent upon the direction of the stress applied. Aelotropic behaviour may also be a result of the occurrence of horizontal or inclined organic fibres.

CONSOLIDATION PORE PRESSURE

In the consolidation theory developed by Terzaghi and Fröhlich (1936), it is assumed that the initial pore-water pressure is equal to the applied total normal stress increment, and the soil is assumed to be normally consolidated. A numerical method suitable for the calculation of one-dimensional consolidation of slightly overconsolidated soils has been developed by Raymond (1966), on the assumption of different consolidation characteristics in the overconsolidated and the normally consolidated range.

According to the theory by Terzaghi and Fröhlich (1936), the pore pressure (u) in a consolidating soil layer with constant initial pore pressure varies with depth (z) and time (t), as expressed by formula (3)

$$u(z,t) = \sum_{m=1,2,\ldots}^{\infty} \frac{2\Delta p}{m\pi}(1 - \cos m\pi)\sin\left(\frac{m\pi z}{2h}\right) e^{-m^2 Mt} \qquad (3)$$

where $Mt = \pi^2 c_v t/(4h^2) = \pi^2 T_v/4$, h = thickness of a half-closed soil layer (drainage through one boundary layer), c_v = coefficient of consolidation, T_v = time factor and Δp = total stress increment. The pore-water pressure in the middle of an open soil layer of thickness 2h, and with drainage through both top and bottom, can be determinated by setting z = h. This pressure is equal to the pore pressure at the bottom of a half-closed soil layer of thickness h, and with drainage through its top alone.

However, as a rule the pore pressure ratio $\Delta u/\Delta p$ is less than 1.0 even at the beginning of the consolidation period. This is illustrated by Fig. 1, which indicates the variations in pore pressure at the bottom of a half-closed soil specimen in laboratory consolidation

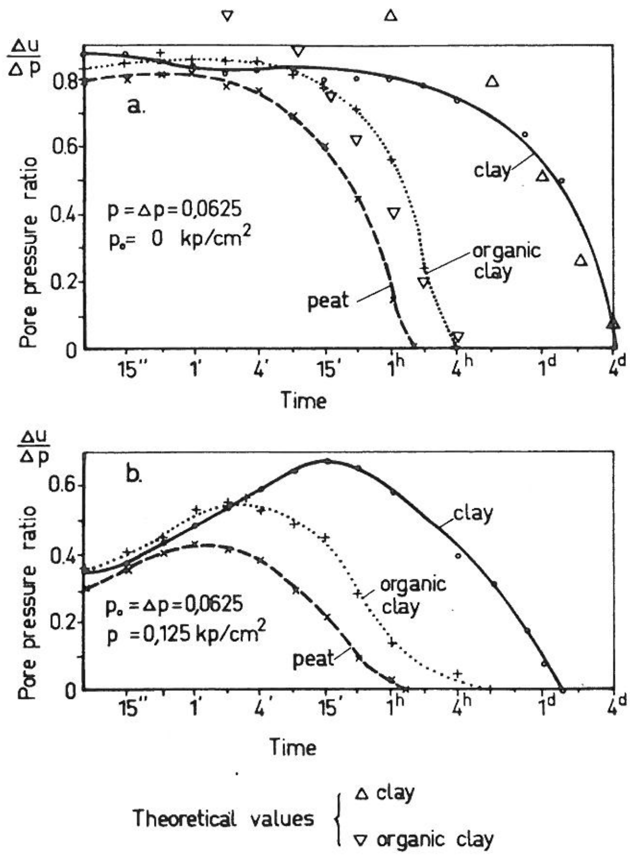

Fig. 1. Pore pressure dissipation at low pressure intensities in consolidation tests on highly compressible remoulded soils.

tests with three different soils. The cross section of the specimen was 20 cm^2, and its thickness was 20 mm (in tests with 2- and 3-layer systems each soil layer was 10 mm thick, and the total thickness of a 3-layer system was thus 30 mm). The pore-water pressure was measured partly by means of a manually operated, Geonor-pore pressure measuring divice, and partly by a self-recording automatic pore-pressure transducer.

The soil material was placed in a wholly remoulded state, with water content close to the liquid limit. The clay material had a liquid limit of w_L = 92.9 and a plasticity index of I_p = 63.2. The corresponding values for the organic clay were w_L = 163.4 and I_p = 108.3, and for peat matrix w_L = 972 and I_p = 300 (the plant fibres were separated from the peat matrix). The first load increment was Δp = 0.0625 kp/cm^2; following this, the load was doubled ($\Delta p/p$ = 1) after a consolidation period of 1...4 days. Special long-duration tests were also made with a consolidation time of 10 days for each load increment.

For the first load increment, the initial pore pressure ratio was approximately $\Delta u/\Delta p$ = 0.8···0.9 (Fig. 1a). However, the time required for pore pressure mobilization generally increased for the following load increments with increasing load. This trend was most evident in peat and organic clay; simultaneously, a diminution occurs in the maximum pore pressure ratio (Fig. 2). This effect is partly attributable to the slowness of the measuring system, particularly when the manually operated pore pressure device was used, as ratio V_L/V_0 between the water volume of the measuring system and the volume of the sample was about 0.07 in the automatic transducer device, and about 0.20 in the manually-operated measuring device - as

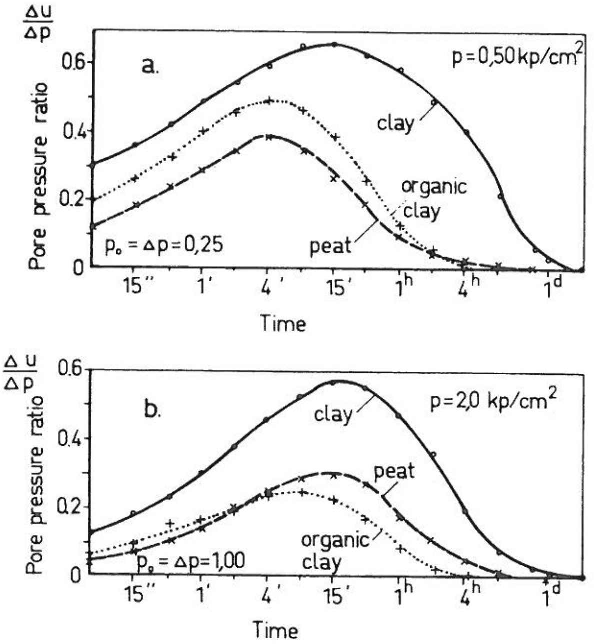

Fig. 2. Pore pressure curves at higher pressure intensities in highly compressible soils.

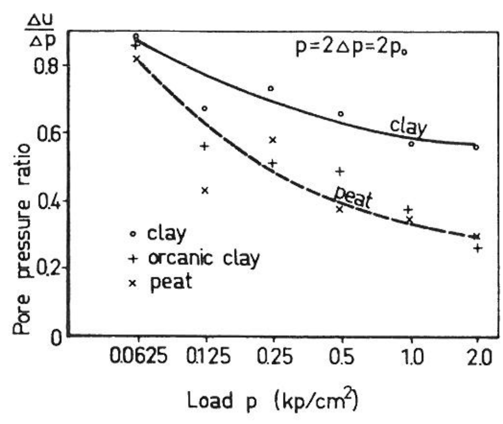

Fig. 3. Pore pressure ratio corresponding to maximum observed pore-water pressure as a function of the applied load in consolidation tests on highly compressible soils.

increase in pressure mobilization time is probably attributable in part to the increasing stiffness of the soil specimen on increase in load. The phenomenom of rising pore pressure during the early stage of consolidation results in a shape of the pore pressure-time curves similar to that of the Mandel-Cryer effect (Hwang et al. 1971). This effect is caused by the state of total stress at a point in a porous body being dependent upon the excess pore pressure at that point, and the mechanical parameters of the soil skeleton.

TESTS ON 2- AND 3-LAYER SYSTEMS

Frequently, natural soil deposits consist of two or three different soil layers, such as peat, organic clay and clay or organic clay, clay and silt, consolidating at the same time. The simultaneous consolidation of soil layer systems of these types may be studied by means of soil specimens built up of different soils, or by consolidation tests, with oedometers connected in series. At theoretical study of the consolidation of a 2-layer system has been made by Kunz (1942), who deduced formulae (4) and (5) for pore pressures u_1 and u_2 in the first and second consolidating soil layer, on the assumption of a constant initial pore pressure (u_0):

$$u_1(z,t) = \sum_{m=1}^{\infty} e^{-\nu_m^2 t} a_m \frac{\sin\left(\frac{\nu_m z}{\sqrt{c_{v1}}}\right)}{\sin\left(\frac{\nu_m h_1}{\sqrt{c_{v1}}}\right)} \tag{4}$$

$$u_2(z,t) = \sum_{m=1}^{\infty} e^{-\nu_m^2 t} a_m \frac{\cos\left[\frac{\nu_m (h - z)}{\sqrt{c_{v2}}}\right]}{\cos\left(\frac{\nu_m h_2}{\sqrt{c_{v2}}}\right)} \tag{5}$$

where h_1 = thickness of the first soil layer, and $h = h_1 + h_2$ = the total thickness of both layers; c_{v1} and c_{v2} are the consolidation coefficients of the different layers. The ν_m-values can be calculated from equation (6)

$$\tan\left(\frac{\nu_m h_1}{\sqrt{c_{v1}}}\right) \tan\left(\frac{\nu_m h_2}{\sqrt{c_{v2}}}\right) = \frac{k_1}{k_2} \sqrt{\frac{c_{v2}}{c_{v1}}} \qquad (6)$$

and the a_m factor is determined by equation (7)

$$a_m = \frac{\dfrac{2u_0 \sqrt{c_{v1}}}{h_1 \nu_m} \sin\left(\dfrac{h_1 \nu_m}{\sqrt{c_{v1}}}\right)}{1 + \dfrac{h_2}{h_1}\left[\dfrac{k_1}{k_2} + \left(\dfrac{m_{v2}}{m_{v1}} - \dfrac{k_1}{k_2}\right) \sin^2\left(\dfrac{\nu_m h_1}{\sqrt{c_{v1}}}\right)\right]} \qquad (7)$$

where k_1 and k_2 are the coefficients of permeability, and m_{v1} and m_{v2} the coefficients of volume compressibility of the different soil layers. A computer program for a consolidating 2-layer system has been worked out by Tammivuori (1971). An analytical solution to the multi-layer consolidation problem for a general set of boundary conditions, and for arbitary load history, has been presented by Schiffman and Stein (1970). The accuracy of a number of approximate solutions for the consolidation of layered clays has been investigated by Barden and Younan (1969); they stressed the importance of the interface-flow condition, but concluded that in practical problems an approximate layer thickness transformation method is generally adequate.

Fig. 5 draws a comparison between theoretical and experimental pore pressures at the bottom of a half-closed, 2-layer system of peat and clay (Tammivuori 1971). The marked discrepancy between the curves is attributable in part to the initial pore pressure ratio

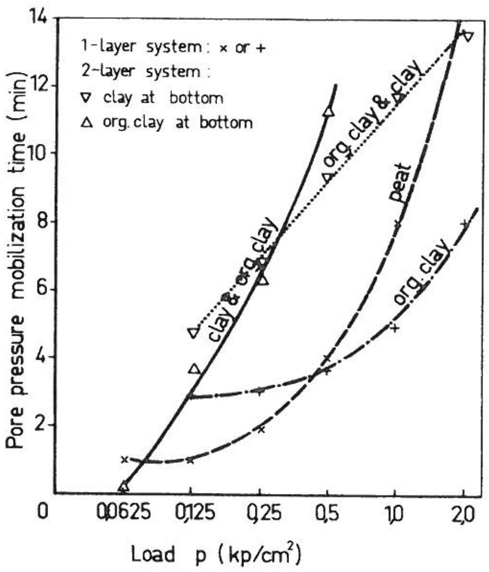

Fig. 4. Time necessary for pore pressure mobilization as a function of the applied load.

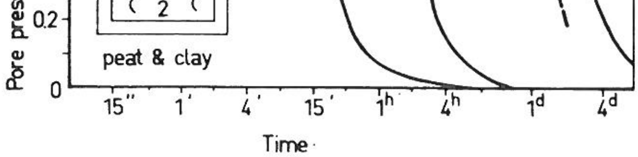

Fig. 5. Comparison of theoretical and experimental pore pressure curves in 2-layer systems of peat and clay.

being less than 1.0, and to the fact that the coefficient of consolidation is not constant (as assumed in the theoretical solution), but decreases with increasing load, and particularly in peat. By reason of the rapid dissipation of pore water adjacent to the drainage boundary, the top layer is compressed even at an early stage of the consolidation period, and to a large extent the progress of consolidation is then dominated by the consolidation coefficient of the top layer.

The influence of the duration of a load upon the pore-water pressure during the following loading period has also been observed in tests with 2- and 3-layer systems. As is demonstrated by Fig. 6, a clear difference exists between pore pressure ratios observed after a one-day preconsolidation period, and the corresponding pore pressure ratios after a period of 10 days of the previous load. The water content of the fine-grained and organic soils tested is very high, and it is possible that the effect is particularly marked in such plastic and organic soils. Similar effects have been reported by Narain et.al. (1969), according to whom the quasi-preconsolidation pressure increased with decreasing load increment ratio, the increasing duration of sustained loading, and rising consolidation pressure.

Fig. 7 illustrates the variations in pore pressure mobilization time in tests on 2- and 3-layer systems. No regard has been paid to the possible influence of which soil layer is situated adjacent to the drainage boundary. The time required for pore pressure mobilization clearly increases with rising load and also with increasing soil stiffness. As the time behaviour is dependent upon the orientation of the consolidating layer system and particularly upon the consolidation coefficient of the soil layer adjacent to the drain-

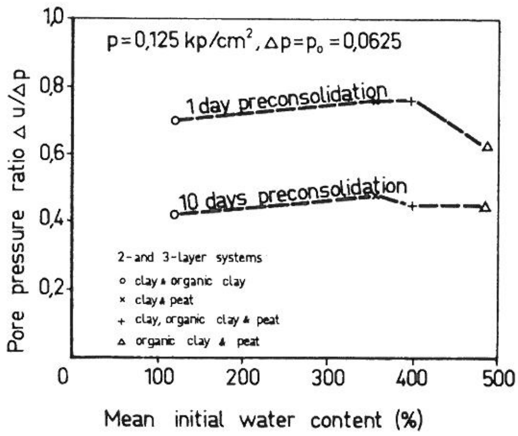

Fig. 6. Influence of duration of the preceding load on the pore pressure ratio in consolidation tests on highly compressible soils.

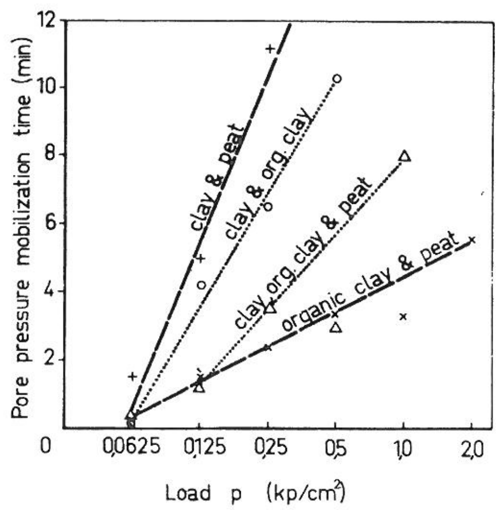

Fig. 7. Pore pressure mobilization time observed in consolidation tests on 2- and 3-layer systems of clay, organic clay and peat.

age boundary it can be expected that the behaviour of, for instance, peat-clay and clay-peat systems will differ. According to Fig. 4, this also seems to be the case in the clay-organic clay system.

DISCUSSION OF TEST RESULTS

One of the basic assumptions of the "classical" consolidation theory is that the soil substance is incompressible, and that the change in volume is solely attributable to expelled pore water. This assumption is far from correct in organic soils, as the organic soil matter may be highly compressible. Partly-decomposed plant fibres, even at great depths under the water table, may contain small amounts of gas, which are not completely removable during saturation with air-free water. Under such conditions, pore pressure ratios considerably less than 1.0 must be expected. The humus content of the organic clay was found to be 11.2 %, and that of the clay material 0.79 % consequently, the compressibility of the soil matter noticeably influences the consolidation behaviour in these soils also.

In a highly compressible (for example organic) soil, it can be expacted that the maximum intergranular stress that acts between the soil particles will be equal to the yield stress of the soil skeleton. The pore pressure is proportional to the difference between total and effective (intergranular) stress, but is further dependent upon the area of contact between the soil particles (Skempton 1960). As the contact area increases with rising load, it can be expected that the pore pressure ratio will diminish with increasing load in drained tests, both as a result of the increase in yield stress of the soil skeleton, and by reason of increasing contact area.

The initial water content of the highly compressible soils tested was very high; as the soils were in a remoulded state, the "stiffness" of the soil structure was very low. The main part of the pore-water can then be regarded as free pore-water with a high pore pressure response. However, when the load is increased, the water content diminishes rapidly, and the part of the pore-water which must be considered to be bound to some extent to the soil particles increases successively. At the same time, the pore pressure response becomes slower, and a decrease occurs in the maximum pore pressure ratio observed.

In undisturbed soil, the grain-skeleton structure induces additional resistance to compression; this resistance reduces the pore-water pressure at an early stage of the consolidation period. When the soil structure is compressed, a part of the load may gradually be carried over to the pore-water, with consequent increase in the pore-water pressure. Observations of pore pressure have shown that the development of pore-water pressure is irregular, and that the pore pressure ratio is smaller at load intensities below the preconsolidation pressure. It can be expected that similar effects will arise in fibrous peat and other soils of pronounced soil structure at load intensities that exceed the preconsolidation pressure.

Negative pore pressures may occur during thixotropic strength regain in remoulded soils. This effect can reduce the observed pore pressure ratio within a certain period after remoulding, but in the main its influence would be limited to the first loading period. However, the additional strength built up by quasi-preconsolidation pressure during sustained loading may be broken down in part by a following load increment. In very sensitive or highly compressible soils with an unstable structure, a load increment that exceeds the critical quasi-preconsolidation pressure can thus induce a new disturbance of the soil structure, which will again affect the pore-water pressure.

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