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As can be seen from the above table the agreement is also in this case surprisingly good.

Later Experience. Thanks to the working hypothesis it was possible to use the obtained test results for the calculation of the expected settlements. Very extensive calculations in this respect were made during the next year and thus the expected settlements and tiltings of the different parts of the works were estimated. The methods used and the very great difficulties encountered during these calculations will not be discussed in this paper. It may only be mentioned that in the original state of the ground the lateral stresses were assumed to be equal to the vertical stresses, and that the changes in the vertical stresses were calculated according to Boussinesq's equations, whereby also forces acting in the soil on account of changes in hydrostatic pressure were taken into account. The settlements at the surface of the ground were calculated as the change in length of laterally confined vertical columns reaching a depth of some 300 m.

In regard to relative size and direction of tilting the settlements observed later on agreed essentially with those estimated. The observed values were, however, only about one-third of the values estimated when using the coefficients for clayey Devon. This was in itself not surprising, since the deposit contained a very great percentage of sandy layers with very small compressibility. The factor of $1/3$ to be applied to the results calculated for entirely clayey Devon was found very soon during the early stage of the work when only the bottom course of concrete had been laid. For all subsequent works correctly estimated values of the settlement could be taken into account. The calculations made in regard to settlement and tilting have thus entirely fulfilled their purpose.

Although the working hypothesis has proved very useful in practice it was later on found that it does not entirely correspond with what actually happens. When continuing the test, the results of which are given in table 3, it proved that the values obtained deviated more and more from the calculated ones, which can be gathered from a comparison between Fig. 1 and 2. Full agreement could of course not be expected as the constants were calculated for another test. Also the fact that the test shows permanent deformation of the sample should not give too serious concern as the permanent deformation observed might be due to disturbances of the sample caused by its cutting out and its transport to the laboratory and does not indicate that, contrary to our assumptions, the actual ground is capable of permanent deformation.

It is more serious that the shape of the experimental unloading curve is not similar to the one determined by the working hypothesis. This indicates weakness in the latter. Further suspicion is aroused by the fact that, according to the working hypothesis, the unloading and the loading curve at partial unloading and subsequent loading should coincide as shown on Fig. 2. Tests carried out later on with sand indicate that at such change in loading the expansion and compression follows a hysteresis loop as shown in Fig. 1.

According to the working hypothesis the relation between the vertical and lateral pressures should be affected by hysteresis but the relation between the deformation and the sum of the principal stresses should be free from hysteresis. Later tests described in a paper by Mr. W. Kjellman have shown that for sand this latter relation is also affected by hysteresis and in all probability this holds true also for clay. Tests with sand have also shown that the lateral stresses by unloading vary in another manner than assumed in the working hypothesis and this is probably also true for clay.

It is thus clear that the working hypothesis described in the preceding paragraph gives only in a somewhat crude way the compressions and expansions in soil and that it should be considered as a first trial only by means of which the laboratory experience can be applied to calculations of settlements of actual structures.

No. D-4

IMPROVED METHODS OF CONSOLIDATION TEST
AND OF THE DETERMINATION OF CAPILLARY PRESSURE IN SOILS
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Summary. (1) It is to exclude the possibility of swelling of the cohesive soil samples during the consolidation test. Otherwise care should be taken that the water which is brought in contact with the sample should be of the same chemical composition as the ground water at the place from which the sample has been taken.

(2) The variations in soil moisture during the consolidation test are to be determined by means of measuring the deformations of the sample.

(3) The ratio of the increment of the lateral pressure to the increment of the vertical one is a constant and depends only on the structure and other natural properties of the soil, while the ratio of the lateral pressure to the vertical pressure may assume any value.

(4) The value of the capillary pressure of cohesive soil samples can be measured by means of one and the same apparatus by different methods, both direct or indirect.

The principles of soil research suggested by Professor Terzaghi in his remarkable work "Erdbaumechanik" are wide-spread in the USSR.

The consolidation test is used in the USSR only with the view of predicting settlements which are to be expected as a result of the compression of soil strata underlying structures with given loadings.

Accordingly, the ultimate pressure is at first determined which would not cause an appreciable settlement in a cohesive water-soaked soil.

This pressure is assumed to be equal to the capillary pressure existing in the soil at the moment the excavation has reached the base of the cut.

Secondly, the compressibility is determined which the undisturbed soil possesses under a loading exceeding the value of capillary pressure existing in it at the beginning of the excavation.

Considering that the portion of the compression consolidation curve corresponding to the practical pressures differs usually little from a straight line, the law of soil compression under foundation loads is assumed to be a linear one. Hence the coefficient of consolidation "a" is derived which serves for the determination of the soil deformation.

The consolidation test is performed according to the principle of Prof. Terzaghi, i.e. the sample is confined laterally, the friction between the sides of the apparatus and the soil is avoided and the surface of the sample is brought in contact with water in order to do away with the menisci which can hamper the outflow of the water from the sample during the test.

Experience has shown that a sample being placed in the apparatus of Terzaghi swells by contact with free water if no external pressure is applied or if the applied pressure is less than the capillary pressure existing in the sample during the test. This swelling is accompanied by suction of water into the sample. However the free water which is generally taken from water-supply line differs from the groundwater in that it contains other soluble salts than those found in the groundwater, and therefore hydrolysis, coagulation and other chemico-physical processes are caused, which influence the mechanical properties of the soil skeleton. To avoid these processes the following improvements have been developed in the USSR:

The undisturbed sample is confined in a rubber envelope "aa" Fig. 1, and placed in a rigid cylinder "dd". All the space between the envelope and the cylinder wall is filled with water and hermetically closed. The bottom of the sample rests on a porous plate "b" and the upper surface is confined by a cover "G" which is held in place by adjusting screws "ii" in order to prevent the swelling of the sample. The porous plate is brought in contact with free water "oc" in a vessel, in which the entire device is immersed. Hence, the possibility of the swelling of the sample is excluded, and its compression begins only when a pressure "p", which exceeds the capillary pressure in the sample, is applied to the cover "G".

The deformations of the sample are measured by means of a dial which records the sinking of the cover "G" as the pressure "p" is increased.

Experience has also disclosed the fact that the determination of soil moisture by drying the sample at 105°C does not give sound results, because it is impossible to evaporate all the water from the sample. This depends upon the fineness of the soil particles and on the sizes of the menisci covering the surfaces of the sample. The observation of this circumstance in laboratory tests often leads to the false conclusion that a part of the sample is filled with entrapped air or gas which in reality is not the case. Therefore the voids-ratio must be determined by means of a dial, as it was mentioned before.

Some devices, based on the principle as illustrated by the Fig. 1 have been constructed recently in various institutions of the USSR (Apparatus of Medkov, Laletin, Boulichev, Aerlich, as suggested by Prof. Davidenkov and Prof. Pokrovsky). In this apparatus the pressure in the hermetically closed water "h" can be measured. The following property of soils has been observed through experiments with this type of apparatus:

If the soil skeleton is compressed, being confined laterally, and the possibility of lateral friction eliminated, then the relation between the vertical pressure "p" and the horizontal pressure "q" acting on the sides of the sample, is as follows:

$$\frac{dq}{dp} = \xi \quad \text{where} \quad (1)$$

dp is the increment of the vertical pressure, dq—the increment of the horizontal pressure, ξ —a constant, depending on the properties of the given soil.

From equation (1) we have

$$q = \xi p + c \quad (2)$$

where c is the integration constant.

The value of c depends on the initial conditions of compression and on the way in which the sample is placed in the apparatus.

For instance, if we fill up with sand the apparatus of Professor Terzaghi, pouring the sand without packing, then at the initial moment we have $P_0 = 0$ and $C = 0$, hence eq. (2) becomes:

$$\frac{q}{p} = \xi$$

i.e. the relation between the vertical and horizontal pressures acting on the sample is represented by a straight line. Fig. 2 shows the results of tests on sand carried out by Boulichev. From these tests it was derived for sand:

$$\xi = 0.41$$

which coincides with the results indicated by Prof. Terzaghi in his "Erdbaumechanik". However if packing is employed when putting the sand in the apparatus, then we have at the initial moment $p = 0$ and $q_0 = q$; putting these values in eq. (2) we obtain $C = q_0$ and the eq. (2) becomes:

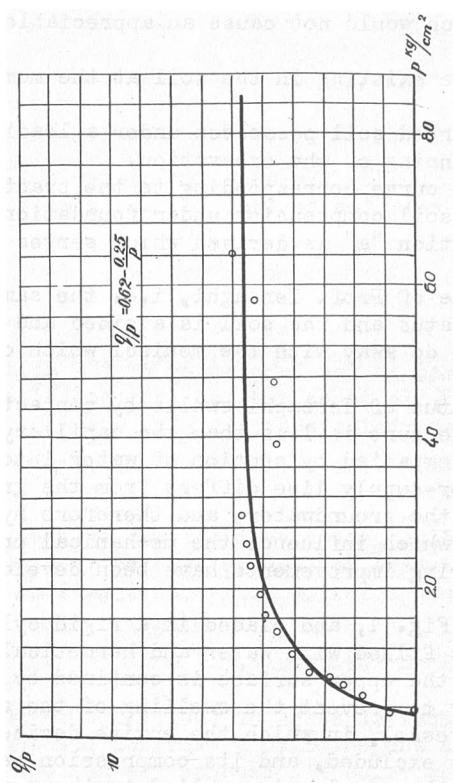


FIG. 4

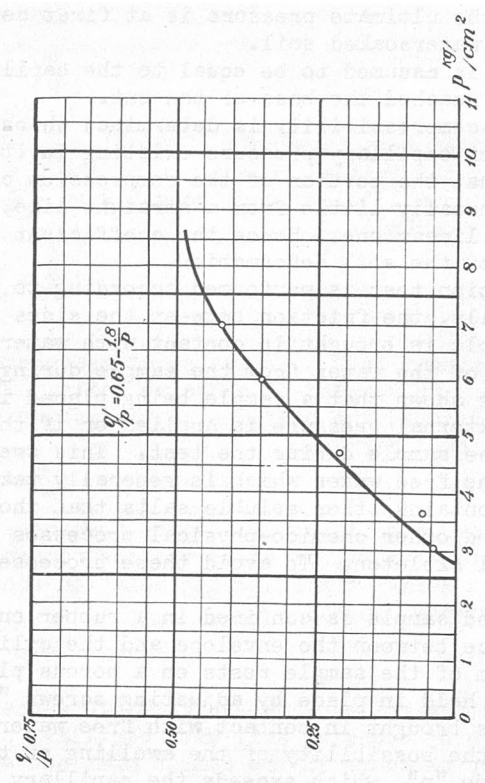


FIG. 5

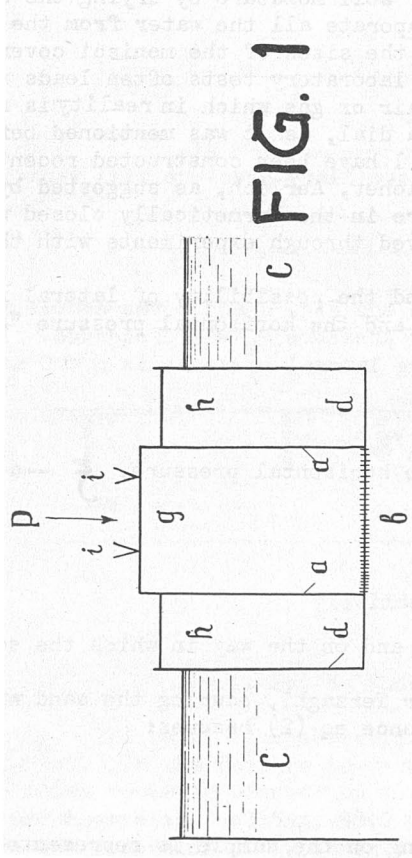


FIG. 1

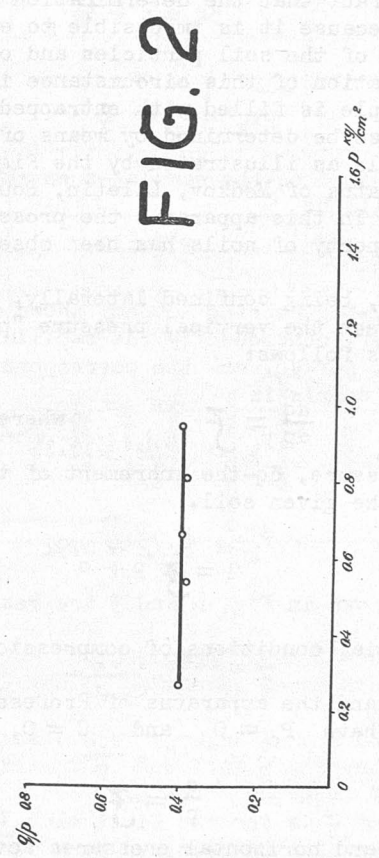


FIG. 2

The relation between the coefficient of lateral pressure and the applied load.

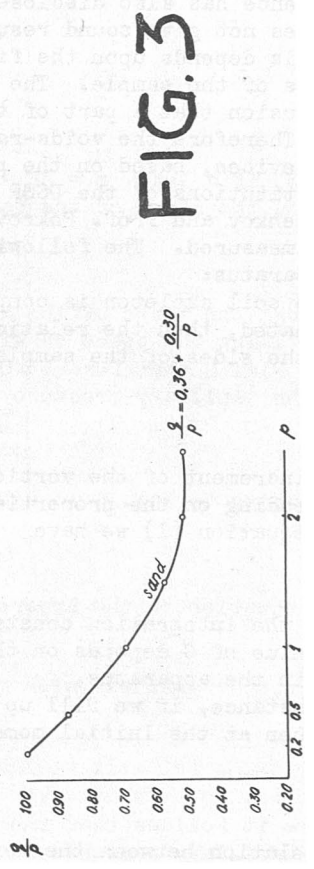


FIG. 3

$$\frac{q}{p} = \zeta + \frac{q_0}{p}$$

which is the equation to a hyperbola (Fig. 3)

If we have at the initial moment $p = p_0$ and $q = 0$, which is usually the case for clayey soils (as it shall be shown in detail below) then we get $C = -\zeta P_0$, and the relation between $\frac{q}{p}$ and P will be represented by the hyperbola:

$$\frac{q}{p} = \zeta - \zeta \frac{P_0}{p} \quad (3)$$

shown in Fig. 4 and 5.

These two diagrams show the results of measurements performed in the laboratory of the Military-Engineering Academy on samples of cohesive soils. The value of ζ is to be derived from eq.(3) assuming $p = \infty$. It represents the ordinate of the asymptote of the hyperbolae shown in Fig. 4 and 5.

It has been found for loam

$$\zeta = 0.62 \text{ and } 0.65$$

If the soil skeleton is compressed by pressures applied in different directions and if these pressures are within the limits, in which the coefficient of consolidation "a" can be assumed as a constant, then it is easy to derive the following relation from eq. (I) (Principles of soil-mass dynamics, by N. Gersevanoff, Moscow, 1933.)

$$\epsilon + \frac{a}{1 + 2\zeta} \cdot (R + S + U) = \text{const.} \quad (4)$$

where ϵ is the voids-ratio of the soil; R,S,U --the three principal stresses of the ellipsoid of stresses.

If a specimen of cohesive soil with capillary pressure P_k is subjected to a vertical pressure p , then the capillary pressure in the sample will be reduced to a new value P'_k because the menisci at the sides of the sample will be flattened. As the moisture of the soil does not change, it follows from eq.(4) that the sum of the principle stresses must remain constant. Before the load is applied this sum is equal to $3P_k$ and after applying the load it becomes $p + 3P'_k$

Therefore:

$$3P_k = p + 3P'_k$$

or

$$P'_k = P_k - \frac{p}{3}$$

Hence, if a load $p = 3P_k$ is applied to the sample, then the capillary pressure will vanish and the pressure on the sides of the sample will be

$$P'_k = 0$$

Through the tests of the Military-Engineering Academy shown on the Fig. 4 and 5 the sample was loaded without filling the space cc with water, consequently the capillary pressure P_k was preserved. It will be seen from Fig. 4 that for $p = 0$, 4 kg. per sq. cm. the corresponding lateral pressure is $q = 0$ and consequently the capillary pressure in the sample is

$$P_k = \frac{0.4}{3} = 0.13 \text{ kg/cm}^2.$$

According to Fig. 5 we have

$$P_k = \frac{2.75}{3} = 0.92$$

The equation to the hyperbolae shown in Fig. 4 and 5 are respectively

$$\frac{q}{p} = 0.62 - \frac{0.25}{p}$$

and

$$\frac{q}{p} = 0.65 - \frac{1.8}{p}$$

from where it follows that the value of ζ is for the first clay $\zeta = 0.62$ and for the second one $\zeta = 0.65$.

The capillary pressure in clay can also be measured by a direct method. In this case the sample is to be placed in the apparatus shown in Fig. 1 and brought in contact with water without the application of any load. After a while the pressure in the water will increase and the capillary pressure which existed in the sample at the moment it was placed in the apparatus can be determined.