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Deformational properties of anisotropic clay soils

Propriétés déformables des sols argileux anisotropes

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SYNOPSIS The results of a field loading test on soft saturated anisotropic soils are described. The pore pressure tends to concentrate in the edge zones of the embankment's foundation. The relationship between the coefficient of permeability and the rate of the change in the pore pressure is derived.

While estimating the deformations of soils in the foundation it is necessary to know their stress-strain state which depends on the processes of elasto-plastic deformation of the soil structure and the seepage of the pore water.

These processes considerably affect the development of the pressure in the pore water and the deformation of soils in the foundation during both the erection and exploitation of a building.

Soft clay soils are often characterised the filtrational anisotropy. Ward et al.(1955) pointed out that the filtrational anisotropy could result in the increase of the pore pressure in the foundation under the edge of the building during its erection and after its completion. Redestribution of the pressure in the pore water between the zones of the foundation under the centre and the edge of the building may take place. The increase of the pressure in the pore water without raising of stresses in the soil structure leads to formation of the zones of swelling, as well as to reduction of the bearing capacity of the foundation.

If the coefficient of permeability in the horizontal direction $k_x=k_y$ is considerably more than that in the vertical direction k_z, then the volume of the pore water having flowed in the horizontal direction into a certain elementary volume of soil may be more than the volume of the pore water, which has seeped out of it both in the vertical and the horizontal direction. This may lead to the increase of the pore-pressure and swelling of soil in the edge zones of the foundation.

To investigate the above-mentioned processes in anisotropic soils Gholly A.V. and Shulyatjev O.A.(1983) under prof. Dalmatov's scientific guidance carried out a field test. On the site composed of the saturated layered sandy clay underlaid in the depth 2.5-2.0m by the moraine sandy clay the experimental embankment was erected (Fig.1).

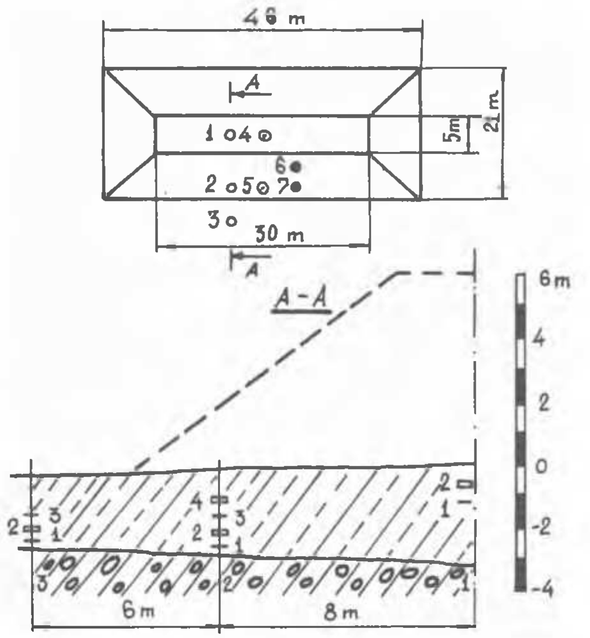

Fig.1 Plan and profile for the test site.

o- Holes with the stress cells;
•- Holes for obtaining the soil density and water content;
⊙- Holes with the ring marks;
▭- Horizontal stress cells;
—- Vertical stress cells.

The upper stratum of the sandy clay had the loyered structure, due to which it was characterized with the clearly expressed filtrational anisotropy. The value of the coefficient of permeability in the horizontal direction was $k_x = k_y = 9 \times 10^{-9}$ m/s, while in the vertical direction $k_z = 1,8 \times 10^{-9}$ m/s.

Some physico-mechanical properties of the soils in the test site are presented in the Table I.

Table I

Soil	γ kN/m^3	w	e	I_L	E mPa
Layered sandy clay	18,9	0,36	1,01	1,0	0,008
Moraine sandy clay	23,0	0,11	0,31	-0,22	0,040

Before the erection of the embankment seven holes were bored in its base. The stress cells were pushed into the walls of the 3 holes(No 1,2,3) in different depths for the determination of the stress state of the soil. The stress cells were driven into the ground practically without disturbing its natural structure by the means of the original device which was developed in LISI (Dalmatov B.I., Gholly A.V.,1970). The design of the stress cells made it possible to measure the vertical σ_z and horizontal σ_y stresses in the ground, as well as the pore pressure u_w, to the accuracy of 17%.

In two other holes /No 4,5/ ring marks were installed. The ring marks were intended to measure the vertical deformations of the successive soil layers to the accuracy of 0,1mm. Technique of the ring marks installation did not hinder the erection of the embankment and excluded the necessity for installation of the complicated bench mark system. The lowest ring marks were used as reference points. They were installed in such a depth, below which no deformation of the soil took place, namely, in the layer of the dense moraine clay.

The holes No 6 and 7 were used for determining the foundation's density and water content with the help of γ and neutron sourses of radioactivity, which allowed to investigate the state of the soil in the zone some 10-15cm around the walls of the holes. The wall casings were installed in the holes.

The embankment was built of sandy loam which was filled and levelled by a bulldozer. The sandy loam was placed in layers, each 1-2m thick. The dimensions of the test embankment after its completion are given in Fig.1.

Changes in the vertical and horizontal stresses in the soil and in the pore pressure according to readings of some stress cells are shown in Fig.2.

After completion of the test embankment (on the 25th day) the pressure in the pore water under the centre of the fill was about 80-90% of the applied pressure. Afterwards the pore

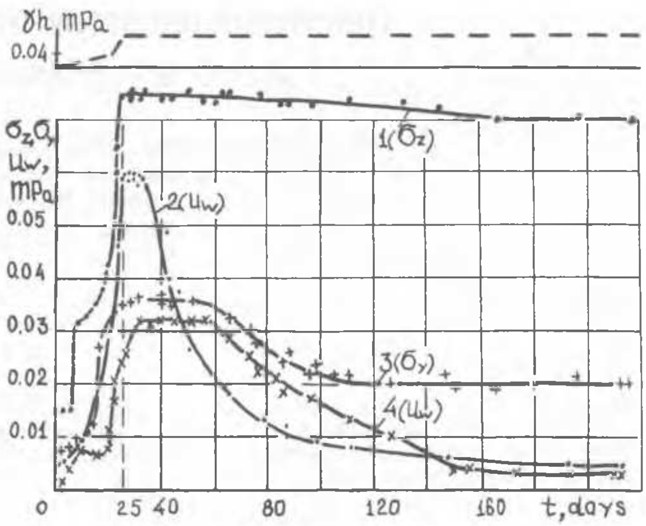

Fig.2 Change of stresses in the soil
1,2 - Hole No1, stress cell 1. Depth z= =-1,2m
3,4 - Hole No2, stress cell 2. Depth z=-2,0m

pressure under the centre of the embankment (hole No1) began to fall. At the same time the pressure in the pore water under the edge of the embankment (hole No2) tended to increase and raised from 0,024 MPa to 0,033 MPa. The stress σ_y has practically reached its maximum value in the last day of the construction period.

There were no changes in the readings of the stress cells, which were installed in the hole No3 in the distance of a little more than 3m from the edge of the test embankment.

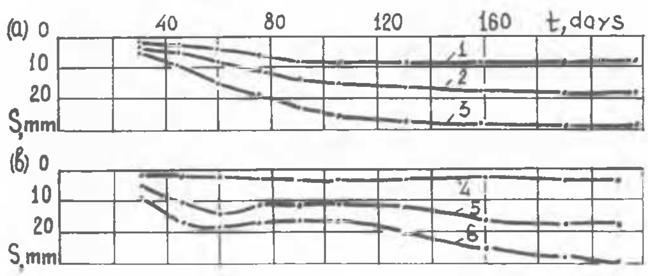

Fig.3 Displacements of the ring marks
1- Hole No4, Depth z = -22m; 2- Hole No4, z = -1.7m; 3- hole No4, z = -1.2m; 4- hole No5, z = -2.8m; 5- hole No5, z = -2.4m; 6- hole No5, z = -1.8m.

Fig.3 shows displacements of the ring marks installed in the holes No4 and 5 under the centre of the load (Fig.3a) and in the edge zone of the foundation (Fig.3b). The upper marks in the both holes installed in the foundation in the depth 0.4m below the ground level, experienced considerable displacements were probably caused by deformations of the lightly consolidated soil in the upper layer. Among other reasons of the deformations there may be the motion of the trucks and bulldozer on the surface of the embankment during its erection. Therefore the date on displacements of these marks are not discussed here. The marks installed under the centre of the embankment (hole No 4), experienced the whole settlement during approximately 160 days, whereas the marks under the edge of the embankment (hole No 5) during the first 60-75 days showed a certain upheaval, that is the soil was swelling. This can be explained by the above-mentioned and observed increase in the pore water pressure due to seepage of the water from the central zone of foundation to the edge zones because of the considerable filtrational anisotropy.

A certain swelling of the soil in the zone of the hole No 7 was confirmed by the decrease in the unit weight of soil according to the results of γ - radiation. In the zone of the hole No 7 the value of the unit weight of soil before the erection of the test fill was of 19.1 kN/m^3 (depth z = -2.2m, in 45 days after the beginning of the erection it was of 19.4 kN/m^3, and in 85 days - of 19,2 kN/m^3.

The rate of the change in the pore pressure of soil depends upon the coefficient of permeability. This fact may be used for determining the coefficient of permeability by the change in the pore water pressure with time. The experiments may be carried out with the help of the hydrocompression cell, which allows to apply the hydrostatic load to the surface of a sample of soil (Fig.4).

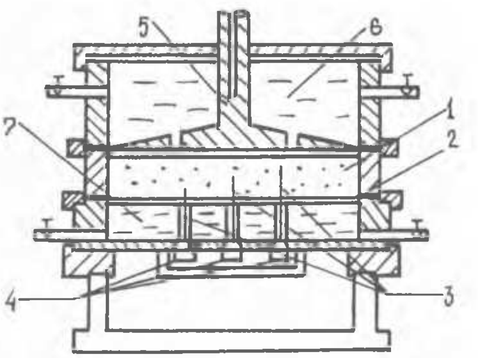

Fig.4 Hydrocompression cell

Under the action of the hydrostatic pressure the process of seepage through the sample begins and the pressure in the pore water increases. Hydrostatic pressure is applied to the soil sample, which is preconsolidated by such a vertical load that no changes in the porosity of the soil take place under the hydrostatic pressure.

The test procedure for determining the coefficient of permeability is following (Gholly A.V. et al.,1983a). Immediately after the placing the sample of the soil under investigation 1 to the ring part 2 of the hydrocompression cell the needles 3 of the pore pressure gauges 4 are pushed into the sample, and the compressive load is applied to the sample with the help of the loading ram 5. Under this load the sample 1 is allowed to consolidate wholly. After that the hydrostatic pressure is applied to the cell 6, which is prefilled with water. The hydrostatic pressure is chosen of such a value that there would be no consolidation of the sample, i.e. the value of the hydrostatic pressure must not exceed 0,01-0,03 MPa. Under this pressure the process of seepage of the pore water from the top cell 6 to the bottom cell 7 takes place. The pressure of the pore water in the sample 1 increases.

With the applying of the hydrostatic pressure to the soil sample the change in the volume of gas bubbles ΔV_g will be equal to the change in the volume of water in the sample ΔV_w, since the porosity of the soil does not alter

$$\Delta V_g = \Delta V_w \tag{1}$$

Assuming the seepage process to be isothermal and using the Mendeleev's law, let us write

$$V_{g1} = \frac{V_{g\,in}\cdot P_{g\,in}}{P_{g1}}; \quad V_{g2} = \frac{V_{g\,in}\cdot P_{g\,in}}{P_{g2}} \tag{2}$$

in which $V_{g\,in}$, V_{g1}, V_{g2} - initial and following volume of gas;

$P_{g\,in}$, P_{g1}, P_{g2} - initial and following pressure in gas.

In such a way, the change in the volume of gas ΔV_g during the time Δt will be

$$\Delta V_g = V_{g1} - V_{g2} = \frac{V_{g\,in}\cdot P_{g\,in}\,(P_{g2} - P_{g1})}{P_{g1}\cdot P_{g2}} \tag{3}$$

From the other hand, the volume of water V_{w1} which has flowed to the elementary layer of the soil sample where the pore pressure is measured, during the time Δt, according to the Darcy's law

$$V_{w1} = k\cdot\Delta t\cdot F\frac{\Delta P_1}{z_1\cdot\gamma_w} \tag{4}$$

in which k is the coefficient of permeability;
Δt is a time, during which seepage of the volume of water took place;
F is a cross-sectional area of the sample;
z_1 is a distance from the the top of the sample to the elementary layer under consideration;
$\Delta P_1 = \Delta P_{top} - \Delta P_m$;
ΔP_{top} is the applied hydrostatic pressure at the top of the sample:
ΔP_m is a mean value of the pore pressure in the layer during the time Δt;
γ_w is the unit weight of water.

Together with the seepage to the elementary layer there also will be the flowing of water out of it. The volume of water which is seeped out of this layer during the time t will be

$$V_{w2} = k\cdot\Delta t\cdot F\,\frac{P_2}{z_2\cdot\gamma_w} \tag{5}$$

where $\Delta P_2 = \Delta P_m - P_{bot}$; P_{bot} is the hydrostatic pressure at the bottom of the sample;
Z_2 is a distance from the bottom of the sample to the end hole of the pore pressure gauge needle, which is installed in the layer.

The change in the volume of water in the layer under consideration

$$\Delta V_w = V_{w1} - V_{w2} = k \cdot \Delta t \left(\frac{\Delta P_1}{z_1} - \frac{\Delta P_2}{z_2}\right) \quad (6)$$

Combining the equations (1), (3) and (5), we obtain

$$k = \frac{P_{g\,in} \cdot V_{g\,in}(P_{g2} - P_{g1}) \cdot \gamma_w}{P_{g1} \cdot P_{g2} \cdot \Delta t \cdot F\left(\frac{\Delta P_1}{z_1} - \frac{\Delta P_2}{z_2}\right)} \quad (7)$$

The initial pressure in a bubble of gas is calculated in accordance with the Laplace's equation

$$P_{g\,in} = P_{at} + U_w + 2\alpha/R_0 \quad (8)$$

where P_{at} is the atmospheric pressure;
U_w is the excess pore pressure;
$2\alpha/R_0$ is the surface stretch.

The values of pressure in the bubble of gas P_{g1}, P_{g2} may be expressed in the same way. The initial volume of gas is determined by the formula

$$V_{g\,in} = V'_{g\,in} \cdot V_s \quad (9)$$

in which $V'_{g\,in}$ is the relative initial volume of gas in the soil sample;
V_s is the volume of the sample and equal to $F \cdot h$.

Finally, the coefficient of permeability is

$$k = \frac{(P_{at} + 2\alpha/R_0)(2\alpha/R_1 + U_{w1} - 2\alpha/R_2 - U_{w2})}{(P_{at} + U_{w1} + 2\alpha/R_1)(P_{at} + U_{w2} + 2\alpha/R_2)} \times$$

$$\times \frac{V'_{g\,in} \cdot h}{\Delta t \cdot F\left(\frac{\Delta P_1}{z_1} - \frac{\Delta P_2}{z_2}\right)} \quad (10)$$

The determination of the coefficient of permeability by the rate of change in the pore pressure takes only some minutes (Gholly A.V. et al., 1983 b), whereas the direct method of determining k takes several hours or days. Besides the above procedure makes it possible to avoid the processes of silting in the soil sample.

If the filtrational anisotropy is relevant it is necessary to carry out the seepage tests across and along the layers of the soil texture.

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