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DETERMINATION OF THE BEARING POWER OF CLAY FOUNDATION

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SUMMARY AND CONCLUSIONS.

Assuming the clay soil as visco-elastic material, the author deduced the relation between the loading intensity and the amount of the sinking of the bearing plate on the soil. Using the data of the bearing tests due to Mr. Housel, the elastic and viscous constants are plotted against the amount of the sinking of the plate.

These curves explain the variations of the mechanical characteristics during the test, and these suggest the clear method of the determination of the bearing power of the clay foundation.

The method of the determination of the bearing power of the clay foundation was investigated fully by Mr. Housel in 1929 and in 1933. His method is based on the fact which he found, that the intensities of the pressure on the bearing plate of various size are related linearly with the periphery-area ratio, namely

$$p = mx + n \tag{1}$$

where

p: the intensity of the pressure,

x: the periphery-area ratio, the ratio of the periphery of the plate to the area of it.

The above relation exists at every stage of the bearing test. The m and n vary with the amount of the sinking of the plate, and from these values Mr. Housel derived so-called physical characteristic coefficient numbers K_1, K_2.

These coefficients K_1, K_2 are regarded to be related with the physical properties of soil under consideration, and by these facts the manner of variation of these numbers tells us the bearing power of the clay foundation. This is the opinion of Mr. Housel.

But his and Mr. Williamss theory about this is somewhat ambiguous, i.e. it contains some uncertain coefficients and the essential characteristic numbers of the soil are not included.

The author intended to clarify this point. The soil under consideration is assumed to have the visco-elastic properties, namely the soil is characteristed by the viscous and elastic coefficients by the following relation,

stress = (a constant) x strain + (a constant) x strain velocity.

Assuming the surface of the ground is the plane z = 0, the positive direction of z-axis to tend to the interior of the ground, and taking the cylindrical coordinates, the relations between stresses and strains may be as follows,

$$\left.\begin{aligned}\widehat{rr} &= -p+2k\frac{\partial u}{\partial r}, \quad \widehat{r\theta}=0\\ \widehat{\theta\theta} &= -p+2k\frac{u}{r}, \quad \widehat{\theta z}=0, \quad k=\mu+\nu\frac{\partial}{\partial t}\\ \widehat{zz} &= -p+2k\frac{\partial w}{\partial z}, \quad \widehat{rz}=k\left(\frac{\partial u}{\partial z}+\frac{\partial w}{\partial r}\right)\end{aligned}\right\} \tag{2}$$

, where u, w are displacements to the directions of the arises r and z and t is used to indicate time, p is the mean pressure, μ, ν are coefficients of elasticity and viscosity respectively.

To derive above relations the author assumed the soil is incompressible. This property is always varified by the plastic flow of the metal, but in this case the condensation of soil under the bearing plate is obviously observed and then this assumption is only approximate.

The constancy of the elastic- and viscous coefficients are also approximate, the elastic constant varies with the condensation and the viscous coefficient also varies with the moisture content remarkably as Mr. Ishimoto and Mr. Iida have shown.

Assuming above relations to be true, the fundamental equations of the visco-elastic soil are as follows,

$$\left.\begin{aligned}&\frac{\partial u}{\partial r}+\frac{u}{r}+\frac{\partial w}{\partial z}=0\\ &-\frac{\partial p}{\partial r}+k\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}-\frac{u}{r^2}+\frac{\partial^2 u}{\partial z^2}\right)=\rho\frac{\partial^2 u}{\partial t^2}\\ &-\frac{\partial p}{\partial z}+k\left(\frac{\partial^2 w}{\partial r^2}+\frac{1}{r}\frac{\partial w}{\partial r}+\frac{\partial^2 w}{\partial z^2}\right)=\rho\frac{\partial^2 w}{\partial t^2}\end{aligned}\right\} \tag{3}$$

When the loading velocity is small, right hand member of the last two equations may be neglected.

The intensity of the vertical stress under the bearing plate is rather uniform, when the soil is clayey. When the confinement of soil under the bearing plate at the periphery is complete the distribution of vertical pressure is increasing to the periphery of the plate but the soil is clayey the plastic flow of the soil at the boundary of the plate may lower the magnitude of this stress and the vertical stress under the plate may be considered to be uniform.

Consequently, the distribution of the vertical pressure under the plate is assumed to be uniform and to be increasing linearly with the time. The case when the plate holds its surface horizontal constantly is treated analogously, and these results are referred later.

Solving above fundamental equations under the boundary conditions,

when z = 0, $\widehat{zz} = -(P_0 + P_1 t)$, when r < a,
$= =$ when r > a
$\widehat{rz} = 0$

we obtain

$$(w)_{z=0} = \frac{a}{2\mu}\left(p_0 + p_1 t - \frac{\nu}{\mu}p_1\right)\int_0^\infty \frac{J_0(\alpha r)\,J(\alpha a)}{\alpha}\,d\alpha \tag{4}$$

, where

J_0. J_1 denote Bessel's functions of zero and 1 order respectively.

Under the boundary conditions,

when z = 0, $\widehat{zz} = 0$ when r > a,
$w = w_0 + w_1 t$ when r < a
$\widehat{rz} = 0$,

we obtain

$$(\widehat{zz})_{z=0} = -\frac{4\mu}{\pi}\left(w_0 + w_1 t + \frac{\nu}{\mu}\dot{w}_1\right)\frac{1}{\sqrt{a^2-r^2}} \qquad r<a$$
$$= 0 \qquad r>a \tag{5}$$

The condition that $\widehat{rz} = 0$ when $z = 0$, is checked by the fact that the influence of the shearing stress over the surface to the vertical deformation of the surface of soil does not exist.

In the above equation the numerical value of the term under the integral sign is a function of r/a, and equals to the value between 0.6369 and 1 then in the following discussions we take it as 1. From the equation (4), we get

$$p_0 + p_1 t = \frac{2\mu}{a}(w)_{z=0} + \frac{\nu}{\mu}p_1 \tag{6}$$

in the present case, the periphery-area ratio of the bearing plate is 2/a and then we obtain the formula

$$p = p_0 + p_1 t = mx + n$$

, where $m = \mu(w)_{z=0}$ $z = 0.$ $n = \nu/(\mu p_1)$ (7)

This is the same equation as Mr.Housel's.

The approximate character of the assumptions above mentioned, the results obtained are expected to be also approximate, but we intend to use these results to examine the characters of the clay foundation. By the help of the equations (7), and using the data obtained by Mr. Housel, we get the viscous and elastic coefficients of soil during the bearing tests.

The coefficient of viscocity thus obtained are related to the sinking of the bearing plate as shown in the figures 2, 4. The

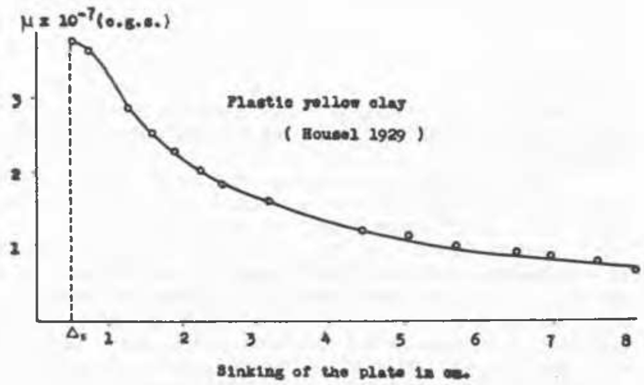

FIG. 1

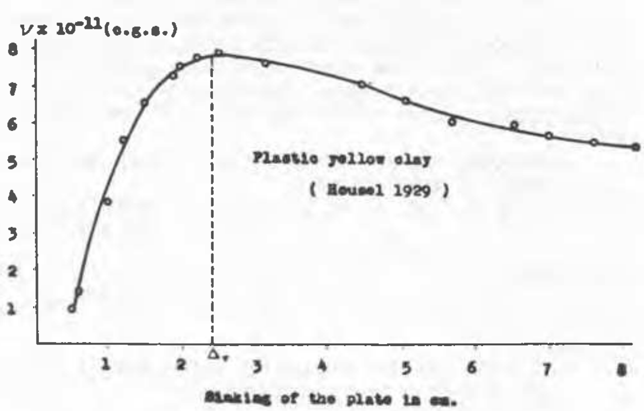

FIG. 2

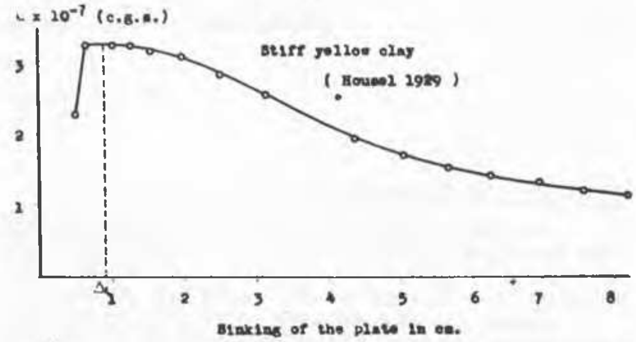

FIG. 3

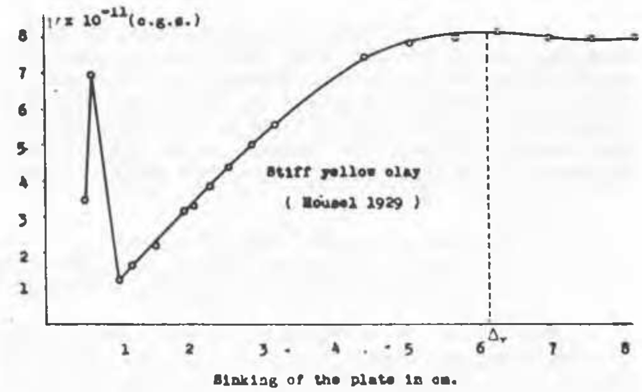

FIG. 4

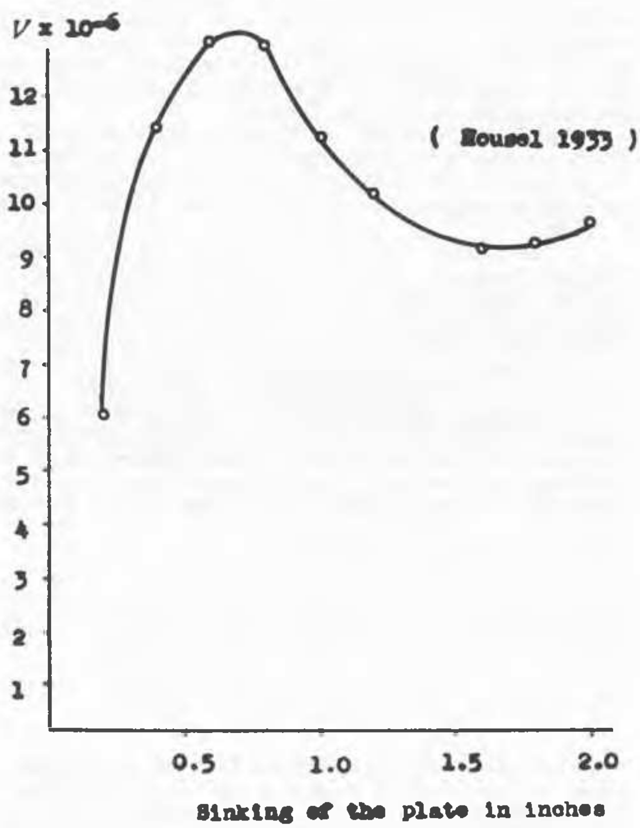

FIG. 5

coefficient of elasticity also varies with the amount of sinking as described in the figures 1, 3.

These curves show that the coefficients by which the mechanical properties of soil under considerations are characterised vary during the bearing test.

We assumed that these coefficients are constant, then these facts are to be interpreted as follows, i.c. the average values of these coefficients if these are constant from the beginning of the test to the instant under consideration, vary with the amount of the sinking of the plate.

From these curves we conclude that the resistance of the soil by elastic properties is considerable when the test was begun but shortly this resistance falls down when the sinking of the plate attained some value, say Δ_E and then it decreases slowly.

Contrary to this, the resistance of soil by viscous properties increases steadily and attains its maximum when the sinking of the plate is, say, Δ_V. When the soil is not stiff the viscous resistance decreases when the sinking of the plate passes Δ_V.

The stiffness of the soil is characterised by the value of Δ_E and Δ_V. The stiffer soil has larger Δ_E, Δ_V and $\Delta_V - \Delta_E$

From the above considerations we may take the value of Δ_E or Δ_V as the critical amount of sinking, as the case may be.

If we compare these values with that obtained by Mr. Housel from the consideration that the amount of sinking of the plate is critical when K2 attains its maximum, we conclude that his value is approximately equal to our Δ_V.

Figure 5 shows the results calculated from the data of the test performed by Mr. Housel in 1933, for the foundation consisting of two layers of soil. The curve shows that the resistance of the lower layer is growing after the upper layer loses its power of resisting.

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I e 3

THE STABILITY OF SLOPES ACTED UPON BY PARALLEL SEEPAGE

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I. INTRODUCTION.

The stability and conditions of inclination of natural and built-up slopes are influenced in a definite manner by the conditions of flow of the water in the pores. Then we have to deal either with steady percolating flow or with the known unsteady hydrodynamic tension phenomena of the pore water. In close connection with this stand above all the slipping phenomena taking place in different phases on hill-sides saturated with water.

Fig. 1 illustrates diagrammatically the two first phases of such a slipping phenomenon. It has been assumed that a source of water on the slope at A has first of all caused a slipping of the steep slope between A en B on a shell-shaped slidding surface. The material fallen down causes a sudden loading on the adjoining zone of slope downwards C - D. Under the aotion of this load a region of pore water subjected to compressive stressing starts in loam soil which has a very small permeability. A seepage, directed towards the free slope C - E on the valley side, exists in parallel with the gradual expanding of the pore water, and in the neighbourhood of the slope may be regarded approximately as parallel flow. The flow pressures thereby occurring cause a disturbance in the equilibrium of the lower parts of the slope in the form of a fresh sliding as second phase of a process which, by a repetition of such sliding phenomena, progresses downwards step by step and may attain formidable proportions, depending on morphological and geological conditions. The sequence of slip-ends in the form of wave peaks, and sagging zones as wave troughs, gives that greatly cut relief which is characteristic of hill-sides that are in danger of sliding and creeping.

In contrast to the above case of a movement of pore water directed outwards and disturbing the equilibrium, is the case of an percolating flow directed inwards, which acts not as a disturbing factor, but as a stabilising factor on the equilibrium of the slope in question. If, for instance in an earth dam as sketched in Fig. 2, the percolation gradient along a stream line is increased by the damming of the water, this causes an increased lift in the zone of the dam freshly submerged, and with this a relieving of the slope is connected.

Closer investigation of the conditions of stability of a slope with seepage shows that the problem may be regarded as purely static and without separation between solid and liquid phases. In the first place the question of the hydraulic gradient i along a normal to the surface of the slope comes into consider-