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Stress-Deformed State Of Partially And Fully Water-Saturated Soil Base

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Abstract – The article discusses the critical load determining issues on the subgrade, taking into account the water saturation degree. The stress-strain state of the soil medium at the loaded area base is predicted taking into account the pores partial saturation with water and air. The stress-strain state assessment was carried out from Tertsagi effective stresses standpoint, i.e. taking into account the pore pressure occurrence and pore fluid compressibility [1,2,3,5].

Keywords – Water Saturation, Pore Water, Volumetric Compressibility, Compaction Phase, Bearing Capacity, Quasi-Homogeneous Medium, Reduced Parameter, Porosity, Effective Stress.

I. Introduction

It is known that the majority of clay soils in natural bedding are in a state close to full water saturation [7]. In this case, the stress-strain state formation of the soil base under the external load influence is complex due to the pore pressure occurrence, and its subsequent dispersion. In the compaction phase, the pore pressure reaches its maximum and largely determines the bearing subgrade capacity and deformability during its active loading period.

Let us consider the stress-strain state of the soil foundation under the local load influence, assuming that the soil skeleton is characterized by the parameters of G_o, K_o linear deformation and $c(\rho), \varphi$ fracture and the pore gas-containing water is characterized by the modulus of K_{gw} volumetric compressibility.

In the compaction phase in a clay soil base, the stress-strain state formation is accompanied by volumetric and shear deformations of the skeleton and an increase in pore pressure, but without changing the solid and liquid components ratio per unit volume, since the filtration process is practically absent. In this case, the soil medium can be considered as a quasi-homogeneous continuous deformable medium, characterized by the reduced K_{np}, μ_{np} deformability parameters and c_{np}, φ_{np} strength, which are determined by the known [6, 8] relations:

$$K_{np} = K_o + K_{gw} / n \quad (1)$$

$$\mu_{np} = \frac{K_{np} - G_o}{2\left(K_{np} + G_o\right)} \quad (2)$$

$$G_o = G_{np} = G \quad (3)$$

$$1/K_{gw} = \frac{1}{\Delta u_w}\left(1 - \frac{S'_r}{S''_r}\right) \quad (4)$$

where n– is a porosity;

S'_r – is an initial value of the water saturation degree;

S''_r – is the current value of the water saturation degree corresponding to the given increment in pore pressure.

Moreover, the water saturation degree S''_r is determined depending on S'_r according to the formula proposed by prof. Z.G. Ter-Martirosyan. [4,6,8].

$$S''_r = \frac{u''_w / u'_w}{\alpha + u''_w / u'_w(1-\alpha) + \left(1 - S'_r\right)/S'_r} \quad (5)$$

where u'_w – is an initial pressure in pore water, equal to the sum of hydrostatic pressure and atmospheric pressure, i.e.

$$u'_w = \mathrm{P_a} + \gamma_w \cdot z \quad (6)$$

after some transformation we get

$$S''_w = \frac{1 + \Delta u_w / u'_w}{\alpha + \left(1 + \Delta u_w / u'_w\right)\left(1-\alpha\right) + \left(1 - S'_r\right)/S'_r} \quad (7)$$

where Δu_w – is an excess pore pressure in the soil relative to the initial pressure in the u'_w pores.

II. Methods and materials.

Thus, the soil medium under consideration is multiphase, consisting of mineral particles, pore water and air, which we presented as quasi-homogeneous, characterized by the reduced parameters of deformation K_{np},μ_{np} and destruction φ_{np}, c_{np}[6,8].

Moreover, the K_{np},μ_{np}, $\varphi_{пр}$, c_{np} parameters are determined from the results of unconsolidated – undrained tests, and the equation of the limiting equilibrium of such a medium can be written in the form:

$$\tau^{т} = \sigma^{т}\, tg\varphi_{пр} + c_{пр} \quad (8)$$

where $\tau^{т}$ is a limit value of shear resistance corresponding to a given $\sigma^{т}$ stress.

In this case, the total stresses components in the soil base under the local load action can be determined by the formulas [1,6,8], and the volumetric deformation ε_v, the surface subsidence $S_o\,(z)$ will be determined as follows:

$$\varepsilon_v = 3\varepsilon(x,z) = \frac{2\left(1+\mu_{np}\right)}{3\pi K_{np}}\, q\left(\alpha_1 - \alpha_2\right) \quad (9)$$

$$S_o(z) = q \cdot \frac{1-\mu_{np}}{\pi G_o}\left[(x+b)\ell n(x+b)^2 - (x-b)\ell n(x-b)^2\right] \quad (10)$$

III. RESULTS.

From these solutions, as special cases, it is possible to obtain solutions for soil with varying degrees of water saturation.

We now turn to the stress state assessment of the subgrade at the end of the compaction phase and at the shear phase beginning. In this case, the stress state assessment can be made from Tertsagi standpoint effective stresses. Then it becomes necessary to determine the pore pressure.

Let us first determine the pore pressure in the subsoil according to the well-known [6,8] formula

$$u_w(x,z) = \sigma(x,z)\cdot \beta_w = \sigma(x,z)\frac{K_{gw}}{K_{gw}+nK_o} \quad (11)$$

где $\sigma(x,z) = \dfrac{2(1+\mu_{np})}{3\pi} q(\alpha_1 - \alpha_2)$ (12)

Then, based on the principle of effective stresses K. Tertsagi

$$\sigma = \sigma' + u_w \quad (13)$$

we obtain the following expressions for determining the main effective stresses taking into account

$$\sigma_{1,2} = \frac{q}{\pi}(\alpha \pm \sin\alpha) \quad (14)$$

i.e. we have $\sigma_1' = \dfrac{q}{\pi}\left\{\alpha\left[1-2\beta_w/3(1+\mu_{np})\right]+\sin\alpha\right\}$

$$\sigma_2' = \frac{q}{\pi}\left\{\alpha\left[1-2\beta_w/3(1+\mu_{np})\right]-\sin\alpha\right\} \quad (15)$$

In the absence of pore pressure $(\beta_w = o)$, this expression coincides with the formula, i.e. with a one-component medium case. To determine the critical load, we substitute equations (15) into the limit equilibrium condition, assuming that the adhesion changes insignificantly due to small volumetric deformations.

We get:

$$z = \frac{q-\gamma h}{\pi\gamma}\left(\frac{\sin\alpha}{\sin\alpha} - B\alpha\right) - \frac{c_o}{\gamma}ctg\varphi - h \quad (16)$$

where $B = 1 - 2\beta_w(1+\mu_{np})/3$ (17)

Expression (16) is the equation of the equilibrium limiting region boundary in the function form $z(\alpha).(z_{\max})$ determined from the extremum condition $\dfrac{dz}{d\alpha} = o$, whence we get

$$\cos\alpha^* = B\sin\varphi$$

$$\alpha^{*} = \arccos(B\sin\varphi) \quad (18)$$

Substituting the obtained α^{*} value into the original equation (16), we determine the value of the critical load q^{*}, which induces plastic flow in the subsoil at a given maximum depth $z_{\max}$, i.e. we get

$$q^{*} = \pi \cdot \frac{\sin\varphi}{\sin\alpha^{*} - \alpha^{*}\cos\alpha^{*}}\left[\gamma(z_{\max} + h) + c \cdot ctg\varphi\right] + \gamma h \quad (19)$$

This is an expression in the special case when $\beta_w = o,\ B = 1$ coincides with the known solution [3,7,8], that is, when the pore pressure occurrence in clayey soil is not taken into account:

$$q^{*} = \pi \frac{1}{ctg\varphi + \varphi - \pi/2}\left[\gamma(z_{\max} + h) + c \cdot ctg\varphi\right] + \gamma h \quad (20)$$

In the case of complete water saturation of the base soils, i.e. when $S_r = 1$ we get that $\beta_w = 1;\quad \mu_{np} = 0{,}5;$ $B = 0$

$$q^{*} = \pi\left[\gamma(z_{\max} + h) + c \cdot ctg\varphi\right] \cdot \sin\varphi + \gamma h \quad (21)$$

IV. CONCLUSION.

If you do not allow the plastic flow occurrence of the soil skeleton at any base point, i.e. when $z_{\max} = o$ we get

$$q^{*} = \pi\left[\gamma h + c \cdot ctg\varphi\right]\sin\varphi + \gamma h \quad (22)$$

Comparing expressions (19) and (21), it is possible to estimate the water saturation degree influence on the critical load value on the soil.

According to existing standards, all soils with a water saturation degree more than 0.8 are considered saturated with water. Therefore, it is advisable to consider an example for cases where the water saturation degree is 0.70; 0.80; 0.85; and 1.0, respectively:

B = 0,948; 0, 734; 0,672; 0,00

μ_{np}= 0,27; 0,39; 0,446; 0,5

$\alpha^{*} = 1{,}012;$ 1,137; 1,152; 1,2566

So we have:

$q^{*} = 1{,}5\text{MPA}$ at $S_r = 0{,}7;$

$q^{*} = 0{,}974\text{MPA}$ at $S_r = 0{,}8;$

$q^{*} = 0{,}910\text{MPA}$ at $S_r = 0{,}85;$

$q^{*} = 0{,}550\text{MPA}$ at $S_r = 1{,}0.$

It can be seen that the bearing capacity of the foundation soils essentially depends on the water saturation degree, which must be taken into account when calculating the structures foundations composed of clay soils.

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