International Journal for Innovative Engineering and Management Research
A Peer Revieved Open Access International Journal
www.ijiemr.org

COPY RIGHT

ELSEVIER
SSRN

IJIEMR Transactions, online available on 25th Nov 2020. Link :http://www.ijiemr.org/downloads.php?vol=Volume-09&issue=ISSUE-12

DOI: 10.48047/IJIEMR/V09/I12/77

Title: **STRESS-DEFORMED STATE OF UNSATURATED SOIL BASE**

Volume 09, Issue 12, Pages: 401-405

Paper Authors

Yakubov M. M., Fayzulla A. I.

International Journal for Innovative Engineering and Management Research
A Peer Revieved Open Access International Journal
www.ijiemr.org

ISSN: 624.046

STRESS-DEFORMED STATE OF UNSATURATED SOIL BASE

Yakubov M. M.- cand. of technical sciences, associate professor, Samarkand state architecture and civil engineering institute, Samarkand. muqumjon.yakubov@mail.ru, Tel: +99893-331-78-33

Fayzulla A. I. - cand. of technical sciences, t.e. prof., Samarkand state architecture and civil engineering institute, Samarkand. incpekciya@yandex.ru, Tel: +99898-273-14-12

Abstract: This article discusses the critical load determining issues, taking into account hardening. The results of evaluating volumetric deformations and changes in density at various subgrade points are presented. It also provides formulas for determining the critical load taking into account the hardening parameter for non-water-saturated clay soils and heavy clays.
Key words: critical load, stress-strain state, compaction phase, volumetric deformation, soil density, plastic flow, adhesion.

Introduction. Let a strip uniformly distributed load (q) act on the subgrade surface on the width (2b) (Fig. 1). It is necessary to determine the critical load magnitude, which causes plastic deformation to a given depth (z_{max}) below the loaded area edges.

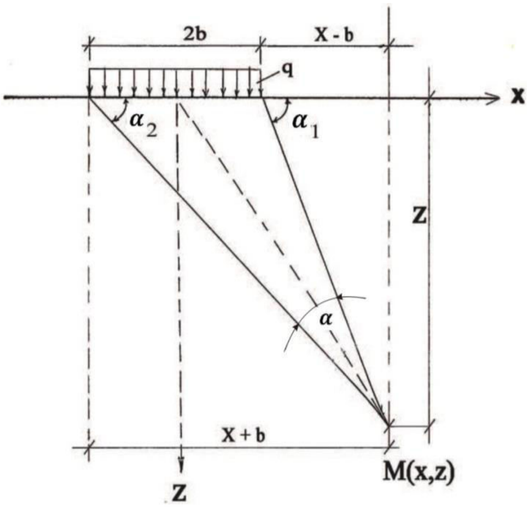

Figure:1. An uniformly distributed load action scheme in a plane problem.

Methods and materials. Let us analyze the formation and development process a stress-strain state in the compaction phase before the significant shears start. It is known [1,5,6] that the compaction phase is characterized by a significant development of volumetric deformations of the soil skeleton and its hardening.

In this regard, the volumetric deformations assessment in the subgrade at various points is of great importance. Such an assessment can be made on the formula basis (1) taking into account (2):

$$\varepsilon = \frac{\varepsilon_x + \varepsilon_y + \varepsilon_z}{3} = \frac{\varepsilon_x + \varepsilon_z}{3} = \frac{\sigma}{3K_o}, \tag{1}$$

where $K_o = \dfrac{E_o}{3(1-2\mu_o)}$;

$$G_o = \frac{E_o}{2(1+\mu_o)}.$$

$$\sigma = \frac{(1+\mu_o)}{3}(\sigma_x + \sigma_z) = \frac{2(1+\mu_o)}{3\pi} q(\alpha_1 - \alpha_2) \tag{2}$$

International Journal for Innovative Engineering and Management Research
A Peer Revieved Open Access International Journal
www.ijiemr.org

i.e. we have:

$$\varepsilon(x,z)=\frac{2(1+\mu_o)}{9\pi K_o}q(\alpha_1-\alpha_2)\quad(3)$$

where $\alpha_1 = arctg\dfrac{z}{x-b}$;

$$\alpha_2 = arctg\frac{z}{x+b} \text{ at } x>b$$

$$\alpha_1=\frac{\pi}{2}+arctg\frac{b-x}{z};$$

$$\alpha_2 = arctg\frac{z}{x+b} \text{ at } x<b.$$

The magnitude of the volumetric deformations of the soil can also determine the increment in the skeleton density by the formula

$$\Delta\rho \cong \rho_o \cdot \varepsilon_v \quad (4)$$

$$\varepsilon_v = 3 \cdot \varepsilon \quad (5)$$

Thus, dependence has been obtained that determines the regularity of changes in soil density at various points of the loaded foundation in the compaction phase [8]. The calculation results performed on the basis of formulas (3), (4), and (5) are shown in Fig. 2 with the following soil parameters:

$$\rho_o = 1{,}4\,g/cm^3;\ K_o = 5\ MPa;\ \mu_o = 0{,}3$$

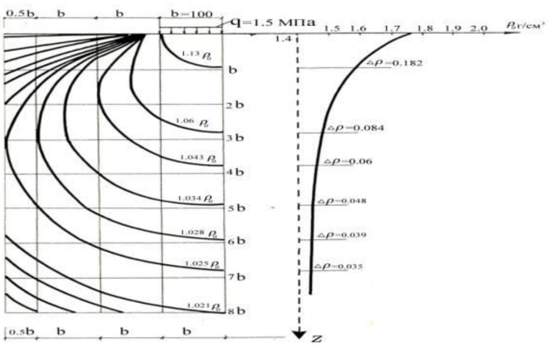

Fig. 2. Density contours at the structures base/on the left/and the change in density along the depth *z* at *x* = *o*/on the right/.

Figure 2 shows that an inhomogeneous state of compaction occurs at the base with a central core under the loaded area at a certain depth from the surface. Obviously, this circumstance should be reflected in the value of the critical load, which is not taken into account in traditional methods of calculating this value [3,6,7]. Let us consider the stress state of an unsaturated subsoil at the final stage of the compaction phase and at the beginning of the shear phase, when a zone of plastic flow arises under the edges of the loaded soil section, and when the compaction process is almost complete.

If we solve these problems by the traditional method [3,4,7], i.e. disregarding soil compaction in the compaction phase, we obtain the following expression for determining the critical load, corresponding to a given plastic flow depth

$$q^* = \frac{\pi}{ctg\varphi+\varphi-\dfrac{\pi}{2}}\left[\gamma(z_{\max}+h)+c\cdot ctg\varphi\right]+\gamma h \quad (6)$$

where h – is a load application depth;

γ – is the specific gravity of the soil is higher than the depth of the load application;

φ, c – is s strength parameters of soil for a given density - moisture.

Obviously, this formula does not take into account the compaction phase effect on the strength properties, i.e. the soil strengthening due to its compaction. This circumstance is not taken into account in the existing methods for calculating the critical load. Meanwhile, it is obvious that it should influence the plastic flow zones development nature in an inhomogeneously compacted base and, therefore, should affect the value of the critical load.

International Journal for Innovative Engineering and Management Research
A Peer Revieved Open Access International Journal
www.ijiemr.org

If we assume that at constant humidity the strength parameters depend on density, and the adhesion is significant, the angle of internal friction is insignificant, then the problem of determining the critical load can be solved as follows:

Let us assume, on the basis of experiments, that $\varphi(\rho)=\varphi=const$

$c(\rho)=c_o(\rho_o)+c_\rho\Delta\rho$ (7) where

c_ρ – is a hardening parameter;

c_o– is an initial cohesion corresponding to the initial soil density.

Then taking into account (3), (4) and (5) we get:

$$\Delta\rho=\rho_o\frac{2(1+\mu_o)}{3\pi K_o}q(\alpha_1-\alpha_2) \quad (8)$$

Hence, taking into account (7), we have

$$c(\rho)=c_o(\rho_o)+c_\rho\rho_o\frac{2(1+\mu_o)}{3K_o}\cdot\frac{q(\alpha_1-\alpha_2)}{\pi}$$

(9)

or $c(\rho)=c_o(\rho_o)+K_\rho\dfrac{q(\alpha_1-\alpha_2)}{\pi}$

(10)

where

$$K_\rho=\frac{2(1+\mu_o)}{3K_o}\cdot c_\rho\rho_o$$

The limiting equilibrium equations for the plane deformation conditions according to Mohr–Coulomb strength theory can be written in the form:

$$\sigma_1-\sigma_2=[\sigma_1+\sigma_2+2c(\rho)ctg\varphi]\sin\varphi$$

(11)

where $c(\rho)$– is determined by formula (9).

In this case, the critical load should be determined on the basis of a joint consideration of equations (11) taking into account (8) and (9), and also assuming that a hydrostatic distribution of stresses arises from the action of the surcharge and its own weight, i.e. we have

$$\sigma_1=\sigma_2=\gamma(z+h) \quad (12)$$

Then we get

$$z=\frac{q-\gamma h}{\pi\gamma}\left[\frac{\sin\alpha}{\sin\varphi}-\alpha(1+K_\rho ctg\varphi)\right]-\frac{c_o}{\gamma}ctg\varphi-h$$

(13)

Obviously, at $K_\rho=o$ this expression coincides with the known [3,7] expressions for determining the plastic flow zone ordinates of soil without hardening..

Expression (13) is the of the boundary equation of the limiting equilibrium region in the function $z(\alpha)$ form, since it satisfies the limiting equilibrium equation (11).

To determine the maximum ordinate of this region, we take the derivative with respect to (α) expression (13) and equate to zero.

$$\frac{dz}{d\alpha}=\frac{q-\gamma h}{\pi\gamma}\left[\frac{\cos\alpha}{\sin\varphi}-(1+K_\rho ctg\varphi)\right]=o$$

(14)

Hence we have that

$$\cos\alpha^*=\sin\varphi(1+K_\rho ctg\varphi) \quad (15)$$

$$\alpha^*=\arccos[\sin\varphi(1+K_\rho ctg\varphi)]$$

(16)

Substituting the obtained α^* value into the original equation (13), we can determine the maximum ordinate value of the plastic flow region

$$z_{\max} = \frac{q - \gamma h}{\pi \gamma}\left[\frac{\sin \alpha^* - \alpha^* \cos \alpha^*}{\sin \varphi}\right) - \frac{c_o}{\gamma} ctg\varphi - h$$

(17)

Hence, it is easy to determine the value of the load that causes plastic flow under the edges of the loaded area to a given $z_{\max}$ depth, i.e. we have

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

(18)

In the particular case, when there is no strengthening, i.e. when $K_\rho = o$ we come to the well-known expression (6), т.е.

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

because $\alpha^* = \frac{\pi}{2} - \varphi$.

Results. Thus, a solution to the problem of determining the critical load on the subsoil corresponding to a given depth of the zone of plastic flow $z_{\max}$ has been obtained.

For heavy clay soils, for which $\varphi \approx o$ expression (18) can be simplified assuming that the equation of limiting equilibrium has the form

$$\sigma_1 - \sigma_2 = 2c(\rho) \quad (19)$$

Substituting here the values σ_1, σ_2 and $c(\rho)$ out of (9), taking into account (10), we get:

$$\frac{q - \gamma h}{\pi}(\sin \alpha - K_\rho \alpha) = c_o$$

(20)

When this expression reaches its maximum, then the state of limiting equilibrium begins to emerge under the edges of the loaded area, i.e. (20) determines the equation of the boundary of the region where the condition of limiting equilibrium is satisfied. Expression (20) will have a maximum at

$$\frac{dq}{d\alpha} = -\pi c_o \frac{\cos \alpha - K_\rho}{(\sin \alpha - K_\rho \alpha)^2} \quad (21)$$

Hence we have that

$$cos \alpha^* = K_\rho \quad (22)$$

$$\alpha^* = \arccos K_\rho$$

The critical load corresponding to a given angle α^* can be determined based on (20) as follows:

$$q^* = \frac{\pi c_o}{\sin \alpha^* - K_\rho \alpha^*} + \gamma h \quad (23)$$

Thus, we have obtained a simple expression for determining the initial critical load q^* on a non-water-saturated clay soil base, which is strengthened in the compaction phase [8].

It is obvious that in the absence of hardening, i.e. when $K_\rho = o$ value of the critical load decreases, and coincides with the known [3,7], values of the critical load for determining the initial critical load without taking into account the hardening, i.e. get

International Journal for Innovative Engineering and Management Research
A Peer Revieved Open Access International Journal
www.ijiemr.org

$$q^* = \pi c_o + \gamma h \quad (24)$$

Conclusion. Taking into account the strengthening of clay soil in the compaction phase when determining the initial practical load is necessary, because this can lead to a significant increase in the bearing capacity of the foundation soils, to a decrease in the size of the foundations and therefore to economic efficiency.

References

1. Bezukhov N.I. Fundamentals of the elasticity, plasticity and creep theory. M., "High School", 1958 , p 512.
2. Bishop A.V., Henkel D.I. Determination of soil properties in triaxial tests. M., Gostroyizdat,1961, p. 231.
3. Gersevanov N.M. Collected works (Vol. I and II). M., 1958.
4. Maslov N.N. Shear resistance of clay soils in an unconsolidated state and creep issues.– In the book: Proceedings of the meetings "Construction on soft soils". – Tallinn, 1975. pp.76-79.
5. Kh. Z. Rasulov, S.M. Norboev. Rheological properties of loess. T., Science and technology publishing, 2012, p.158. (pp.28-31).
6. Ter - Martirosyan Z.G. Forecast of mechanical processes in massifs of multiphase soils. -M., "Nedra",1986, pp 68-78.
7. Tsitovich N.A. Soil mechanics. M., "High School",1979, p. 268.
8. Yakubov M.M. Deformability and strength of multiphase clay soil at different density – moisture. The research abstract for the Cand. tech. sciences degree, M., 1984, p. 25.