Bearing capacity of foundations under regime cyclic loading

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ABSTRACT

The method of calculation of bearing capacity of the foundation base with long-term non-linear deformation of clay soils under the regime of cyclic prolonged static loading. In the calculation of the bearing capacity of the base is considered a solid phase of development zones limit equilibrium, which is achieved for the foundation bases at the end of the formation of a hard core, deforming the ground.

Keywords: clay soil, bearing capacity, limit equilibrium, limiting the critical pressure, regime of cyclic loading

1. INTRODUCTION

A solution of problem of quality of quantitative prediction of the behavior of soil masses of foundations, primarily requires proper determination of strength and deformation parameters of soil base, taking into account their initial stress-strain state, the change of rheological properties and stress-strain state during the operation of the building.

One of the specific objectives in the design of structures in service to consider the effect of cyclic loading, is to assess the carrying capacity and forecast sediment of foundation bases for the changes in the physical and mechanical properties of soils in the joint action of long-term static and cyclic loads. These issues are particularly relevant to the grounds composed of clayey soils (Mirsayapov & Koroleva 2013).

2 LABORATORY TESTING OF SOIL

To determine the effect of loading mode and the type of stress state on the strength and deformability of clay soil were studied experimentally under cyclic loading regime (Fig. 1).

During the experimental studies established that the strength of the soil samples under regime's combined long-term static and cyclic loading from the strength of soil samples with short-term cyclic loading and the strength of the soil samples during prolonged static loading is different (Fig. 2).

Characteristic features of the destruction of the samples were obtained. The soil sample zones of different density are different from the original values were observed (Fig. 3). This fact shows the difference

in the degree of damage soil's these zones.

Fig. 1. The regime's combined long-term static and cyclic loading.

To determine the influence of the spatial state of stress on physical and mechanical characteristics of the soil after the destruction of the samples of the typical areas of soil samples were taken and main changes the physical and mechanical characteristics were determined.

The integrated volume during the long loading there is a internal friction angle decrease by 16%, the specific adhesion of soil decreases by 6%. At the same time, the values φ and c obtained by monoplanar soil's cut taken from characteristic areas of these samples are significantly different from these values. The angle of

internal friction φ of soil in vertical pyramids in short term tests was 153-161% of the initial value, during the long tests it increased on average by 88%, in zone 4 (Fig. 3) φ decreased by 32-35% and by 41-45% respectively. Specific cohesion c in dense soil zone increased in 1.88 and 2.38 times, and in loosened zone decreased by 41-43% and 61-67% of the initial value depending on the duration of the test. Information about the angle of internal friction φ and specific cohesion c change characteristics of the sample areas and integrated volume with short-term and long-term triaxial compression are presented in diagrams (Fig. 4 and Fig. 5).

Fig. 2. Graph changes in soil strength under different conditions of triaxial loading.

Fig. 3. Location scheme of local zones of different density of a specimen under triaxial tests: 1-vertical consolidated pyramids; 2 – consolidated pyramids at specimen's sides; 3 – uniformed deflected state area; 4- area of dilatancy.

Fig. 4. Graph of changing of the specific cohesion c in

characteristic areas of the sample based on a series of tests under the triaxial compression.

Fig. 5. Graph of changing of the angle of internal friction φ in characteristic areas of the sample based on a series of tests under the triaxial compression.

This shows that the deterioration of physical and mechanical properties of the soil under the triaxial compression largely localized in the zone 4 (Fig. 3).

According to test results (Mirsayapov & Koroleva 2011a) the schedule of dependence between the average stress σ_m and volumetric strain ϵ_v , between the intensity shear stresses τ_i and the intensity of shear strain γ_i , between the average stress σ_m and the intensity of shear strain γ_i .

The joint analysis of these graphs shows that an increase in medium voltage and time of load change is the development of deformation shape (positive dilatancy) during compaction of soil within the tested volume.

The results were used in developing the methodology of calculation of the bearing capacity of soils foundation bases under regime cyclic loading.

3 BEARING CAPACITY OF SOILS FOUNDATION BASES

Evaluation of the bearing capacity of the base is done in order to ensure the strength and stability reasons. According to SP 22.13330.2011 "SNIP 2.02.01-83 * Foundations of buildings and structures" bearing capacity of foundation is considered secured when the condition

$$F \leq \gamma_c \cdot F_n / \gamma_n, \quad (1)$$

where F – design load;

F_n – limit bearing capacity of the foundation;

γ_c – coefficient working conditions;

γ_n – reliability coefficient.

In assessing the bearing capacity of the base is used limit the critical stress corresponding to complete exhaustion of soil bearing capacity and the continuous development of zones of limit equilibrium, which is

achieved for the foundation bases at the end of the formation of a hard core, deforming the base and expander soil in sides (Fig. 6).

Fig. 6. Design scheme to the definition of the limit the bearing capacity of the foundation base under long-term static and cyclic loading.

In accordance with the applicable design standards of foundations in the evaluation of the bearing capacity of foundation design resistance under the foundation is determined by the formula

$$R_n = N_\gamma \cdot \xi_\gamma \cdot b \cdot \gamma_1 + N_q \cdot \xi_q \cdot \gamma_1' \cdot d + N_c \cdot \xi_c \cdot c_1, \quad (2)$$

where N_γ , N_q , N_c – dimensionless coefficients of bearing capacity of the soil under the foundation, depending on the angle of internal friction φ_i ,

$$\begin{aligned} N_\gamma &= (N_q - 1)(0.29 + 0.47 \operatorname{tg} 1.4\varphi); \\ N_q &= \operatorname{tg}^2 \left(45^\circ + \frac{\varphi_1}{2} \right) \cdot e^{\pi \operatorname{tg} \varphi_1}; \\ N_c &= \frac{N_q - 1}{\operatorname{tg} \varphi_1}. \end{aligned} \quad (3)$$

ξ_γ , ξ_q , ξ_c – coefficients depending on the ratio of the length and width of the foundation soles $\eta = \frac{l}{b}$,

$$\xi_\gamma = 1 - 0.25/\eta; \xi_q = 1 + 1.5/\eta; \xi_c = 1 + 0.3/\eta. \quad (4)$$

γ_1 and γ_1' – specific gravity ground under the foundation, which are possible within the prism bulging respectively above and below the sole foundation.

Calculated parameters φ_I , c_I according to GOST 12248-2010 "Soils. Laboratory methods for determining the strength and deformability"

Thus, the limit values of the design resistance R_n and ultimate load P_n depends on the angle of internal friction φ_I and specific adhesion c_I .

For soils that have rheological properties, angle of internal friction $\varphi_I(t)$ and specific adhesion $c_I(t)$ is not constant. They depend on the combination of exposure time long-term static loading and cyclic loading.

In the calculation input maximum long-term value $\varphi_I(t)$ and $c_I(t)$, corresponding to the time $t = t_{exp}$ or

$t = t_n$, where t_{exp} – time – period of operation structures and t_n – observation time, which determines the ultimate bearing capacity of the foundation base.

The values of the angle of internal friction $\varphi_I(t)$ and cohesion specific $c_I(t)$ determined according to the orientation of potential sites limit state, which is generally not constant, but changes in the process of inelastic deformation of the ground $\alpha = \arccos(l)$ (Mirsayapov & Koroleva 2011a).

In accordance with the results of the research value of the specific coupling under regime long-term static and cyclic loading can be represented as

$$\begin{aligned} c_0(t, N, \tau_1) &= c_0(\tau_1) \cdot m(t, N, \tau_1) \cdot \lambda(t, N, \tau_1) \times \\ &\times \sqrt{\frac{K(\tau_1)}{K(t, N)}} \cdot \frac{1}{1 + K(\tau_1) \cdot C(t, N, \tau_1)} \end{aligned} \quad (5)$$

where $C(t, N, \tau_1)$ – the soil creep volume rate;

$c_0(\tau_1)$ – the initial value of the soil specific adhesion under short-time loading;

$m(t, N, \tau_1)$ – soil hardening function due to water-colloidal links recovery;

$\lambda(t, N, \tau_1)$ – hardening function by restoring the structural relationships of soil based on a combination of various blocks under the regime loading;

$K(\tau_1)$ – initial value of the bulk modulus of deformation of soil;

$K(t, N)$ – current value of the bulk modulus of deformation of soil.

Changing the angle of internal friction of soil is determined from the condition:

$$\alpha = 45^\circ \pm \frac{\varphi}{2}. \quad (6)$$

The change in the time value of the angle of internal friction is determined by the expression:

$$\varphi(t) = \frac{\pi}{2} - 2\alpha(t), \quad (7)$$

where $\alpha(t)$ – the time-varying inclination angle of sliding pads to a plane perpendicular to σ_1 .

Time-varying inclination angle sliding pad can be represented as:

$$\alpha(t) = \arccos(l), \quad (8)$$

$$\text{where } (l)^2 = \frac{1}{3} \cdot \frac{3d\varepsilon_2 \cdot d\varepsilon_3 - I_2 + \sqrt{I_2^2 - 3I_1 I_3}}{(d\varepsilon_1 - d\varepsilon_2)(d\varepsilon_1 - d\varepsilon_3)};$$

$d\varepsilon_1, d\varepsilon_2, d\varepsilon_3$ – main deformations increments under the regime loading;

$$I_1 = d\varepsilon_1 + d\varepsilon_2 + d\varepsilon_3,$$

$$I_2 = d\varepsilon_1 \cdot d\varepsilon_2 + d\varepsilon_2 \cdot d\varepsilon_3 + d\varepsilon_3 \cdot d\varepsilon_1,$$

$I_3 = d\varepsilon_1 \cdot d\varepsilon_2 \cdot d\varepsilon_3$ – first, second and third deformation increments invariants.

Based on the fact that at regime long-term static and

cyclic loading strain increment is mainly due to the creep strain, the expression for the calculation of strain increments take the form:

$$\begin{aligned} d\varepsilon_1(t) &= \frac{1}{E(t)} \int_{\tau}^t K_0(t, N, \tau) \cdot \sigma_1(N, \tau) \cdot d\tau; \\ d\varepsilon_2(t) &= \frac{1}{E(t)} \int_{\tau}^t K_0(t, N, \tau) \cdot \sigma_2(N, \tau) \cdot d\tau; \\ d\varepsilon_3(t) &= \frac{1}{E(t)} \int_{\tau}^t K_0(t, N, \tau) \cdot \sigma_3(N, \tau) \cdot d\tau, \end{aligned} \quad (9)$$

where $K_0(t, N, \tau)$ – the core of the volume creep;

$E(t)$ – deformation modulus;

t – total duration of the cyclic loading and prolonged static load.

Consequently, the orientation of potential sites limit state (shift) during the inelastic deformation varies in accordance with the formula

$$\begin{aligned} \alpha(t, N, \tau) &= \\ &= \arccos \sqrt{\frac{1}{3} \cdot \frac{3d\varepsilon_2(t) \cdot d\varepsilon_3(t) - I_2 + \sqrt{I_2^2 - 3I_1I_3}}{(d\varepsilon_1(t) - d\varepsilon_2(t))(d\varepsilon_1(t) - d\varepsilon_3(t))}} \end{aligned} \quad (10)$$

Then the dimensionless coefficients of bearing capacity during long-term inelastic deformation takes the form

$$\begin{aligned} N_{\gamma} &= (N_q - 1) \times \\ &\times \left(0,29 + 0,47 \cdot \operatorname{tg} 1,4 \left(\frac{\pi}{2} - 2\alpha(t, N, \tau) \right) \right); \\ N_q &= \operatorname{tg}^2 \left[\frac{\pi}{4} + \frac{\pi}{4} - \alpha(t, N, \tau) \right] \cdot e^{\pi \operatorname{tg} \left[\frac{\pi}{2} - 2\alpha(t, N, \tau) \right]}; \\ N_c &= (N_q - 1) / \operatorname{tg} \left[\frac{\pi}{2} - 2\alpha(t, \tau) \right]. \end{aligned} \quad (11)$$

Expressions (11) allow us to estimate the ultimate bearing capacity of the soil foundation under long-term cyclic loading regime with the strained state of the soil and changes in the rheological properties of soils.

When calculating the bearing capacity of the foundation base proposed method was tested (Mirsayapov & Koroleva 2011b). The results shows of comparison in Figure 7.

Fig. 7. Schedule changes of soil strength in time.

4 CONCLUSIONS

An improved method for calculating the bearing capacity of the foundation is designed. The proposed method allows the calculation of the bearing capacity of foundation bases, depending on the strain state of change and processes of self-strengthening clay soils, duration and mode of loading and reliably evaluates the bearing capacity of the grounds at all stages of loading.

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