Subgrade vertical deformations of a building in the zone of a deep pit influence

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Abstract. The existing methods for calculating the settlements of the foundations of buildings are developed for the soil mass without defects and damage. It is also assumed that the physical and mechanical characteristics of the soil are constant throughout the life cycle of buildings and structures. But the mechanical characteristics of subgrade soils are unstable and continuously change under the influence of various factors. The use of existing methods of calculation leads to inadequate settlement estimates of the subgrade and foundations of buildings and structures. In this regard, this work has developed a method for calculating the settlements of bases and foundations of buildings, taking into account the formation and development of defects and damages in the soil structure in the zone of influence of a deep pit. This method allows you to take into account the influence of factors such as dilation, formation and development of cracks, hardening, changes in the friction angle, shear modulus and deformation modulus. The developed method was tested on a real reconstructed object. Geotechnical monitoring for one year confirmed the results of calculations using the proposed methodology.

Key words: deep pit, long-term loading, soil rheology, creep, stress-strain state, calculation method, cracks, triaxial compression, deformation coefficients, finite element method, settlement.

1 Introduction

During constructing deep pit, located in a developed neighborhood, there is a need for geotechnical forecast (estimate) of impact of construction on the change in the stress-strain state of the soil massif, including foundations of surrounding buildings. Geotechnical forecast of influence should take into account the technological settlement of the foundations of the surrounding buildings, cause of diaphragm wall constructing [1-4]. This should take into account horizontal movement of pit shielding construction and unloading soils, loads from a new built structure or additional stress of the reconstructed buildings and other factors based on underground parts construction sequence, using analytical and numerical methods of calculation [5-6]. It also must consider nonlinear soil deformations under appropriate loading regime [7-11]. According to the standards of the Russian Federation, depending on the category of technical condition of existing buildings, the ultimate additional settlements of buildings are 20-50 mm. Calculation of bases on deformations for constructions of the surrounding building located in a zone of influence of new construction or reconstruction, according to standards of design of bases, is carried out from a condition:

\[ s_{ad} \leq s_{ad,u} \]  \hspace{1cm} (1)

Where \( s_{ad} \) is the additional settlement of surrounding building, \( s_{ad,u} \) is the ultimate additional settlement of surrounding building.

The existing deformation calculating methods are designed for the case of short-term loading with constant parameters for the whole period of exposure and exclude changes in the rheological
properties of the soil [12-15]. Actually mechanical and rheological properties of subgrade soils change at the constructing of pits near existing buildings [16-20]. In this regard, it is necessary to develop methods for calculating the settlement of bases, which take into account the features of deformation under such conditions.

2 Materials and methods

2.1 Description of the case

Building properties. The studied two-story building concrete frame made of precast concrete, with a basement floor plan has a rectangular shape, is located in the axis 9-15/A-G, has dimensions 36×36 m. The vertical bearing elements are the columns and the brick wall in the axes 9-15/G, 9/E-G, 15/F-G. The cross section of the building is shown in figure 1.

![Cross section of building](image)

**Figure 1.** Cross section of building.

Columns with dimensions of 300×450 mm are made of concrete of class B20. Floor slabs and coverings with a thickness of 220 mm are made of concrete of class B25. Crossbars with a height of 550 mm are made of concrete of class B25. The foundations of the building are columnar in the characteristic part with a size of 2.0×2.0 m. Depth of the foundation is 4.150 m and tape under the brick walls.

Soil properties. According to the geological survey, alluvial-deluvial Quaternary deposits and upper Permian eluvial deposits, overlapped by a technogenic deposits of Quaternary age, participate in the geological structure of the site (in the bored depth).

From the surface to the studied depth of 25.0 m, the geological and lithological structure is represented by the following engineering and geological elements (IGE): Bulk soil (IGE-1), Clay loam (IGE-2), Loam (IGE-3), Clay (IGE-4), Sandstone (IGE-5). The engineering and geological section is shown in figure 2, the physical and mechanical characteristics of the soil of the Foundation bases are shown in table 1.

**Table 1.** Physical and mechanical characteristics of soils.

| №  | ρ  | e   | E   | φ  | C  |
|----|----|-----|-----|----|----|
| 2  | 1.91 | 0.76 | 15.1 | 21 | 32 |
| 3  | 1.88 | 0.842 | 13.1 | 19 | 25 |
| 4  | 1.88 | 0.857 | 25.2 | 21 | 46 |
| 5  | 1.91 | 0.617 | 37.8 | 35 | 0  |
Where \( \rho \) is the density at natural humidity \((t/m^3)\), \( e \) is the porosity coefficient, \( E \) is the modulus of deformation \((MPa)\), \( \varphi \) is the friction angle \((deg)\), and \( C \) is the adhesion \((kPa)\).

2.2 Initial prerequisites

In the immediate nearness of the surveyed building from axes A, 9 and 15, pit is arranged. Depth of the pit, made with vertical walls without proper shoring, is 13 m. As a result of digging a deep pit, there were shear deformations of the base of the foundations, as well as excessive vertical deformations. Evidence for excessive deformation of the foundation soils was the appearance of cracks with a width up to 1.5 mm in the joints of the basement floor slabs, cracks in the beams of the basement floor with a width up to 1.5 mm.

The calculation of settlements was performed using LIRA-SAPR software complex, which uses the finite element method. In the structure of the program, a 3D model of the building was created. The columns were modeled with bar elements, slabs with shell elements, foundation elements with shell elements. This model was created to assess the effect of the stiffness of the above-ground part of the building on the foundation settlement, i.e., to calculate the settlements of the foundation base taking into account the joint deformation of the system "above-ground and underground parts of the building – foundation – subgrade". The deformation properties of the soil base in the model were taken into account by the foundation modulus \( C_1 \) and \( C_2 \) calculated by the modified Pasternak method, considering the continuous change in the three-dimensional stress state of the soil base in time.

As known, the Pasternak method describes the work of the soil using the compression coefficient \( C_1 \), which relates the intensity of the vertical rebound of the soil with its settlement, and the shear coefficient \( C_2 \), which characterizes the shear forces that occur due to adhesion and internal friction between its particles.

In this paper, the modified parsnip model was modified. For this purpose, when calculating the coefficients \( C_1 \) and \( C_2 \), the modules of base soil deformations are accepted for triaxial compression regime, taking into account changes in the volume stress-strain state of the soil, creep, unloading, long-term loading, formation and development of micro- and macro-cracks in the base soil mass in accordance with the recommendations [21].
The prediction of the subgrade geomechanical behavior must take into account main parameters changes that characterize the stress-strain state of the soil over time (figure 4).

**Figure 3.** a) Subgrade loading regime by design standards; b) Real subgrade loading regime during building erection.

**Figure 4.** a) Changes in soil strength during loading regime with the side pressure of 0.08 MPa; b) Changes in soil strength during loading regime with the side pressure of 0.16 MPa.
2.3 Calculation model

The soil ultimate stresses, taking into account the prehistory of loading, formation and development of cracks in subgrade, are determined based on the calculation model proposed in [22] (figure 5).

At the developing analytical equations of soil behavior under regime of alternating long-term static loads, the calculation model of soil under triaxial compression developed by I.T. Mirsayapov and I.V. Koroleva [23] is used as a basis for regime loads (figure 5).

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure5.png}
\caption{a) Stress-state of soil elementary volume at X, Y, Z at time \( t \) in pre-ultimate condition; b) In the space of main stresses at the stage of ultimate equilibrium; c) Cracks development scheme on the ultimate equilibrium surfaces in the soil elementary volume.}
\end{figure}

The accepted spatial model of dilating soil under conditions of loading regime taking into account changes in the main mechanical characteristics of clay soil in the process of the above loading regime and the development of micro- and macrocracks on the equilibrium surfaces. The results of experimental studies show that all the main parameters that characterize the stress-strain state of soils change under the considered modes [24-26]. This allows us to conclude that there are not constants characterizing the mechanical state of soil and necessary for creating computational models for practical calculations.

The maximum stress in the soil at triaxial loading is written as:

\[
R_{gr}(t, t_1, N) = \sigma_{tu}(t, t_1, N) = \frac{4A_{sh}}{A_1} \left[ \sigma_v(t, t_1, N) \cdot \cos \alpha_1(t, t_1, N) + \right] \\
\cdot \left[ +\sigma_v(t, t_1, N) \cdot \sin \alpha_1(t, t_1, N) \right],
\]

(2)

where \( A_{sh} = \frac{b^2}{4\cos \alpha_2(t, t_1, N)} \) – the area of the faces of the stress pyramid, and \( A = b^2 \) is the surface area of the cube.

Normal stress in the form of:

\[
\sigma_v(t, t_1, N) = \sigma_1 \cdot l(t, t_1, N) \cdot l'(t, t_1, N) + \sigma_2 \cdot m(t, t_1, N) \cdot m'(t, t_1, N) + \\
+\sigma_3 \cdot n(t, t_1, N) \cdot n'(t, t_1, N).
\]

(3)

Dilatant stresses in the form of:

\[
\sigma_d(t, t_1, N) = \frac{E \cdot \Delta \delta_d}{(1 + \nu) \cdot \nu}.
\]

(4)

Shear stresses at the limit equilibrium site:

\[
\tau_v(t, t_1, N) = S \cdot \tan \varphi_0(t, t_1, N) + c_0(t, t_1, N, \tau).
\]

(5)

Deformations of soil creep at time \( t_1 \) under long-term static loading are determined by the formula:

\[
\varepsilon_{pl}(t_1, t_0) = c_{\varepsilon}(t_1, \tau) \cdot \sigma(t_{1}, t_0) \cdot f(t_1, t_0),
\]

(6)

where \( f(t_1, t_0) = 1 - e^{-\gamma(t_1-t_0)} \) – creep strain growth function;

\( \gamma \) – the parameter of soil creep;

\( c_{\varepsilon}(t_1, \tau) = \frac{\varepsilon_{pl}(t_1, \tau)}{R_{gr, u}(t_1, \tau)} \) – limit measure of soil creep at time \( t \).
Figure 6. The stress state of the foundation base under long-term loading regime.

Average stress and stress intensity:
\[
\sigma = \frac{\sigma_x + \sigma_y + \sigma_z}{3},
\]
\[
\sigma_i = \frac{1}{\sqrt{2}} \sqrt{\left(\sigma_x - \sigma_y\right)^2 + \left(\sigma_y - \sigma_z\right)^2 + \left(\sigma_z - \sigma_x\right)^2 + 6 \left(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2\right)}.
\]

When the axes of the main stresses and deformations coincide with the central axis of the foundation:
\[
\epsilon_i = \epsilon_1 + \epsilon_2 + \epsilon_3, \quad \epsilon_i = \frac{2}{3}(\epsilon_1 - \epsilon_3)
\]

In other cases we use the condition of alignment of stress tensors and strain increments:
\[
\frac{\Delta(\epsilon_x - \epsilon_y)}{\Delta(\sigma_x - \sigma_y)} = \frac{\Delta(\epsilon_y - \epsilon_x)}{\Delta(\sigma_y - \sigma_x)} = \frac{\Delta(\epsilon_z - \epsilon_x)}{\Delta(\sigma_z - \sigma_x)} = \frac{\Delta\epsilon}{\Delta\sigma_i} = \chi.
\]

Conditional modules that characterize the transition from the natural state of the base to the state when an additional vertical load is applied:
\[
K_V = \frac{\Delta\sigma}{\Delta\epsilon_V},
\]
\[
G_V = \frac{\Delta\sigma_i}{3\Delta\epsilon_i}.
\]

Conditional modules that characterize the transition from the natural state of the base to the state after applying a local load:
\[
K_V(t) = \frac{\Delta\sigma}{\Delta\epsilon_V + \Delta\epsilon_V(t)},
\]
\[
G_V(t) = \frac{\Delta\sigma_i}{3(\Delta\epsilon_i + \Delta\epsilon_i(t))},
\]
\[
\Delta\epsilon_i = \sigma_i \cdot K_v(t, \tau).
\]

The increment of axial deformation:
\[
\Delta\epsilon_z = \frac{\Delta\sigma_z}{G_V} - \Delta\sigma \cdot \frac{3K_V - G_V}{3K_V \cdot G_V},
\]
\[
\Delta\epsilon_{z,i}(t) = \frac{\Delta\sigma_z}{G_V(t)} - \Delta\sigma \cdot \frac{3K_V(t) - G_V(t)}{3K_V(t) \cdot G_V(t)}.
\]

The base sediment is divided into equal layers up to the conditional depth of the compressible thickness:
\[
S = \sum_{i=1}^{n} \left(\epsilon_{z,i} + \epsilon_{z,i}(t)\right) \cdot h_i.
\]
3 Results

Based on the above methodology, numerical studies of the stress-strain state of the soil base of the foundations of the building in question were carried out in the LIRA-CAD software complex, which implements the finite element method, taking into account the influence of a deep pit using a modified Pasternak calculation model based on an analytical calculation model of the soil under triaxial loading regime. The obtained calculation results are shown in figures 7, 8, which confirm the general picture of deformation of the building under study.

![Figure 7](image1.png)

Figure 7. Vertical movement (mm) of building foundations.

![Figure 8](image2.png)

Figure 8. Vertical movement of the soil mass.

4 Discussions

To get the results of calculating the settlements of the foundation, which have a slight deviation from the real one, it is necessary to take into account the rigidity of the overlying structures, as well as the influence of the pit wall. Calculations can be performed using software complexes that use the finite element method, allowing you to build a three-dimensional model of a building. The model can
include the entire building or only the underground part. In the calculation model, the foundation can be considered on an elastic base. To account for the plastic properties of soil, changes in the volumetric stress-strain state of soil, creep, unloading, long load, formation and the development of micro and macro cracks detection in soil base, it is recommended to use the modified Pasternak model, taking into account changes in volume and shear modulus of deformation analysis model of soil under triaxial loading regime.

Numerical studies have shown that both vertical and horizontal deformations occur under the building foundations. More than half of the building is in the zone of horizontal deformations greater than 5 mm. Deformations of the base lead to deformation of the entire frame of the building and additional non-design stresses in the structures. The maximum horizontal and vertical movements of the foundations were 40 and 70 mm respectively.

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