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SIMULATION OF THE SOIL BASE OF A BUILDING FOUNDATION NEAR A DEEP PIT

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Abstract: The densification of urban development necessitates the construction of buildings with a developed underground volume. In this process, existing building foundations are often located at the edge of deep excavations. The task of assessing the impact of constructing deep foundations on surrounding structures is important and relevant. An analysis has been conducted based on data from foundation settlements near the edges of deep excavations using numerical modeling results and geotechnical monitoring studies by Russian and foreign authors. To evaluate additional settlement of building foundations near deep excavations, research was carried out to study the stress-strain state of the ground near a deep excavation in a laboratory model created in a flat tank with transparent walls. Displacement values were recorded using video recording and electronic sensors, then processed. The experimental results correlate well with those obtained by other researchers. A pattern for distribution of horizontal stresses and displacements beyond the boundaries of the deep excavation model has been established. Based on these findings, a method for calculating foundation settlements near deep excavations considering changes in the stress-strain state of the surrounding soil mass has been proposed. Based on the research results, it has been found that there is an uneven change in the stress-strain state of soils under building foundations along the edges of excavations. Deformation of the soil mass beneath the foundations of buildings located within the collapse prism occurs non-linearly and unevenly. Construction of deep excavations leads to changes in the stress-strain state of the soil mass supporting foundations situated at the edge of the excavation. This causes changes in the ratio between vertical and horizontal stresses, resulting in altered deformation characteristics of the soil and consequently increased settlement.

Keywords: Numerical simulation, Geotechnical monitoring, Soil mechanics, Deep pit, Clay soil.

1. INTRODUCTION

Urban densification creates a need for constructing buildings with extensive underground volumes. Existing building foundations are frequently located adjacent to the edges of deep excavations. Assessing the influence of deep foundation construction on surrounding developments remains both significant and timely.

The stress-strain condition of soil foundation beneath building foundations close to excavation pits is highly complex. Deformations occur due to influences such as deformation of retaining structures, defects or damage caused during installation of retaining elements, and soil excavation activities (Mirsayapov & Aysin, 2021). Other factors include distance from the building's foundation to the retaining structure, type and rigidity of the foundation itself, overall structural stiffness, and total load exerted by the building onto its subsoil base.

Research by Jiang et al. (Jiang, Lu, Chen, Liu, & Li, 2021) indicates that when excavating a trench 18 meters deep, maximum surface soil settlements occurred at distances of 8 meters and reached 9.5 mm, corresponding to a maximum displacement of the retaining wall by 13.9 mm. Here, the coefficient characterizing maximum subsidence outside the trench f_1= s/H equals 0.05%. Studies by Dong et al. (Dong, Burd, & Houlsby, 2018) show that for a 12-meter-deep excavation, maximum settlements measured 7.5 mm at points 10 m away from the edge, while horizontal movements amounted to 5.6 mm with a maximum retaining wall shift of 14 mm, yielding f_1 = 0.075%.

Monitoring results presented by Nikiforova N.S. (Nikiforova, 2010) demonstrate that at the “Okhotny Ryad” site in Moscow, maximum settlements approached 13 mm given a depth of 16 meters, accompanied by horizontal

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shifts measuring 10 mm, thus leading to f_1= 0.2%. According to Ter-Martirosyan Z.G. et al.'s investigations (Ter-Martirosyan, Ter-Martirosyan, & Vanina, 2022), the greatest foundation settlement next to a rigid-retaining wall reached 44 mm for a 15-meter-deep trench, giving f_1 = 0.29%. Meanwhile, research by Zertsalov M.G. et al. (Zertsalov & Kazachenko, 2021) reveals that at depths of 9 meters, maximum soil surface settlements equalled 16 mm just half a meter from the trench's edge, producing f_1= 0.18%. However, generally speaking, the range of $f1$ lies between 0.1% and 10.1%, averaging about 1.1% (Mangushev , Ilyichev, Nikiforova, & Sapin, 2016).

According to Nikiforova N.S. et al.'s work (Nikiforova & Vnukov, 2011), simply installing cutoff screens without additional measures proves insufficient to reduce extra settlements below normative levels in Moscow's engineering-geological conditions. Similarly, studies by Il'ichev V.A. et al. (Ilyichev, Mangushev, & Nikiforova, 2012), Nikiforova N.S. et al. (Nikiforova, Konovalov, & Zekhniev, Geotechnical problems in the construction of unique objects, 2010), Shishkin V.Ya. et al. (Shishkin, Pogorelov, & Makeev, 2011) indicate that jet-grouting cutoff screens may not provide adequate protection against excessive settlement of neighboring building foundations in Moscow and St. Petersburg when working in trenches ranging from 10–20 meters deep.

This study focuses on foundation bases adjacent to deep excavations in clayey soils. Specifically, it investigates the stress-strain behavior of clay soils underlying building foundations positioned directly above excavation edges. Its objective is to simulate deformations occurring in such foundation bases to assess additional foundation settlements. Key tasks include:

- Reviewing current literature related to the subject matter,
- Laboratory-scale modeling of stress-strain conditions in clay soils forming part of building foundations situated above excavation rims,
- Processing and analyzing research outcomes,
- Developing a methodology for calculating additional foundation settlements taking into account alterations in the stress-strain condition of soil supporting foundations adjacent to deep excavations.

2. MATERIAL AND METHODS

When determining soil deformations (both vertical and horizontal) in the foundations of buildings located at the edge of an excavation, the limit state of the system consisting of "excavation retaining structure - soil foundation - foundation" is studied based on the Coulomb theory. It is based on several assumptions:

1. The system is examined in a state of ultimate equilibrium (Figure 1), i.e., at the moment when the wall begins to move and the failure wedge starts sliding.
2. The failure wedge slides along a straight plane through the soil and the back face of the retaining structure.
3. The failure wedge is assumed to be a perfectly rigid body.
4. In classical Coulomb theory, only fill material is analyzed.

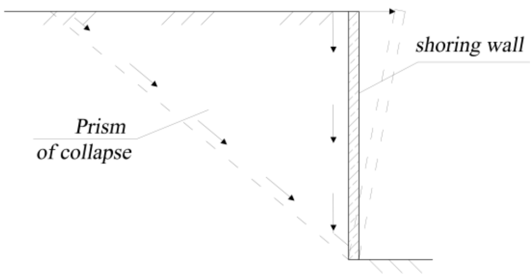


Figure 1. *Ultimate State According to Coulomb Theory*

Field and laboratory experimental studies reveal that soil within the failure wedge undergoes irregular nonlinear deformations, including both horizontal and vertical displacements. Additionally, during deformation processes, vertical and slightly inclined cracks appear. As the failure wedge moves backward away from the excavation, its length increases. Analysis of geotechnical monitoring results for the foundation base of a building near a deep excavation shows uneven distribution of horizontal and vertical soil deformations across the area between the excavation and the building foundation, depending on the rigidity of the retaining structure and the building itself.

Building deformation near the excavation demonstrates that, as the soil base shifts towards the excavation, complex motions occur within the failure wedge – horizontal and vertical displacements, as well as bending within the failure wedge, leading to crack formation. During complicated deformation and crack formation, the failure wedge penetrates further inward toward the building, encompassing a larger portion of the foundation. This phenomenon is clearly demonstrated by the schematic representation of crack formation in the concrete basement floor of a building located near a deep excavation.

Initially, cracks spread over a distance of 1.5 meters from the wall (from the first foundation). Subsequently, these defects expanded, new cracks appeared, affecting adjoining foundations, indicating ongoing deformation of the building's foundation soil base.

Modeling of the foundation was performed in a flat tank with transparent vertical walls (ACIS 0.7.1 manufactured by NPPC Geotec). The test setup is illustrated in Figure 2.

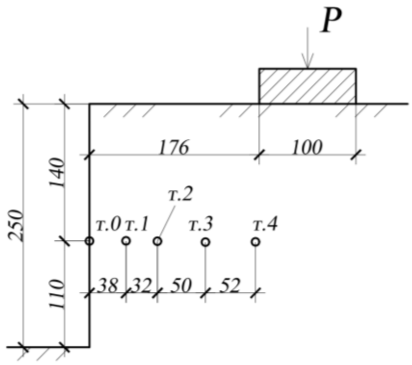


Figure 2. *Test Scheme and Location of Points Under Consideration*

3. RESULTS AND DISCUSSIONS

Experimental and numerical studies of the behavior of soil foundation beneath buildings located at the edge of an excavation demonstrate uneven distribution of horizontal stresses and deformations, as well as vertical deformations (settlements) in the area between the excavation and the building foundation, depending on the rigidity of the retaining structure.

The maximum magnitude of both vertical and horizontal displacements is observed at the retaining structure, and they are approximately equal to each other. Then, as one moves away from the edge of the excavation, both horizontal and vertical displacements decrease.

Figures 4-5 present graphs illustrating the development of vertical displacements (settlements) and horizontal displacements derived from experimental and numerical studies.

Analysis of the obtained results allows us to identify trends in the variation of key parameters describing the stress-strain state of the soil foundation beneath a building located at the edge of an excavation.

Based on the provided charts, we can conclude that horizontal stresses decrease, which subsequently reduces the lateral earth pressure coefficient within the considered soil array. Consequently, within the zone influenced by the excavation, the strength and modulus of soil deformation diminish, while vertical soil deformations increase, ultimately leading to greater overall settlements accounting for shear and vertical deformations.

To determine additional horizontal soil displacements within the active soil pressure prism, proceed as follows. Determine boundary values of horizontal displacements (Δ) at the excavation enclosure (see Figure 3):

$$\Delta = \Delta_1 + \Delta_2 + \Delta_3 \tag{1}$$

Where:

Δ_2 – wall section displacement at the bottom of the excavation;

Δ_3 – displacement arising from rotation of the wall section at the bottom of the excavation;

Δ_1 – deflection of the wall segment with free length, subjected to trapezoidal loading diagram with top ordinate σ_{ah1} and bottom ordinate σ_{ah2}, at excavation depth H, with the stiffness of the retaining structure being EI.

$$\Delta_1 = \frac{H^4}{120EI}\left(11\sigma_{ah1} + 4\sigma_{ah2}\right). \tag{2}$$

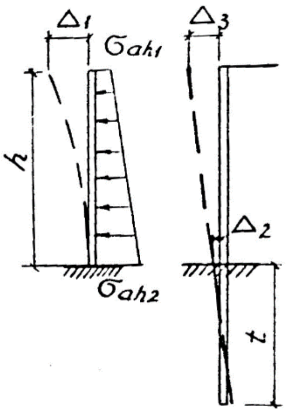


Figure 3. *Diagram for calculating deformations of cantilever wall.*

Based on the graphs presented in Figure 4, additional values of horizontal displacements between the excavation and the foundation are determined.

Using the results of experimental studies, the unevenness of absolute horizontal displacements Δx_i and relative horizontal displacements ε_{xi} within the active soil pressure prism *L* is assessed (as depicted in Figure 4).

$$L = k \cdot H \cdot \tan\left(45° - \frac{\varphi}{2}\right) \tag{3}$$

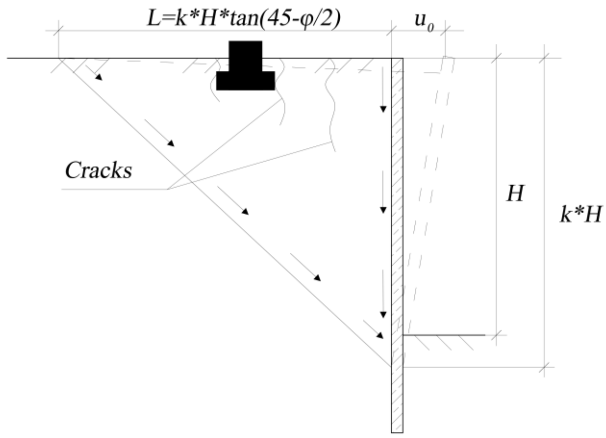


Figure 4. *Scheme for Determining Settlements and Horizontal Displacements of Foundations Near Excavations*.

Within the length of the active pressure prism in different zones of compressible strata, the change in horizontal pressure is calculated using the following formula:

$$\Delta\sigma_{xi} = \varepsilon_{xi} \cdot E_0 \ , \tag{4}$$

$$\varepsilon_{xi} = \frac{\Delta x_i}{L} \ , \tag{5}$$

Then, the total value of horizontal pressure is equal to:

$$\sigma_{xi} = \sigma_{x1} - \Delta\sigma_{xi} \tag{6}$$

$$\xi_{xi} = \frac{\sigma_{xi}}{\sigma_{zi}} = \frac{\sigma_{x1} - \Delta\sigma_{xi}}{\sigma_{zi}} \ , \tag{7}$$

where σ_{x1} represents the horizontal pressure in the soil after applying the load.

The vertical stress from the self-weight of the clay soil is calculated.

External load P*P* acting on the foundation is divided into steps, taking into account time and duration of application. The height of the compressible layer is determined according to SP 22.13330.2016.

$$H_S = Z;\ \ \sigma_{zp} = 0{,}5 \cdot \sigma_{zg} \tag{8}$$

Where:

H_S – thickness of the compressible layer taken at depth Z.

σ_{zp} – vertical normal stress at depth Z due to additional load on the foundation along the vertical axis of the structure.

σ_{zg} – vertical normal stress at depth Z due to the self-weight of the soil.

The active compression zone is divided into separate layers along the depth of the foundation, taking into account stratifications of the soil.

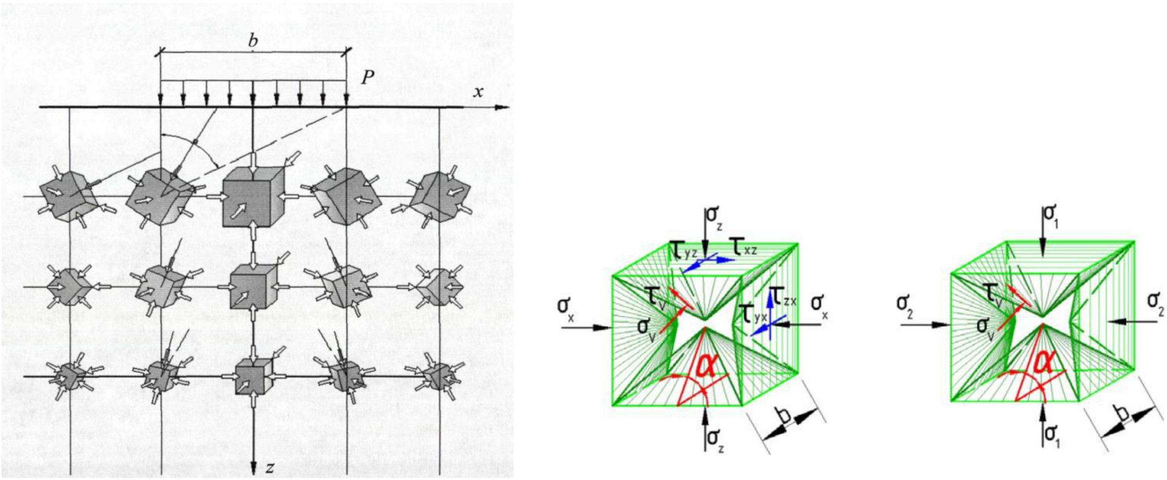
The natural stress state due to the self-weight of the clay soil is determined. At the level of the foundation sole and midway through each layer below the foundation sole, vertical stresses σ_{zg} and horizontal stresses σ_{xgi}, σ_{ygi} from the self-weight of the soil are computed. The magnitude of lateral pressure σ_{xgi} and σ_{ygi} constitutes fractions ξ_{xg} and ξ_{yg} respectively of the vertical pressure σ_{zgi}. Values of ξ_{xg} and ξ_{yg} are accepted as equal to each other and numerically equal to 0.25.

Additional vertical stresses σ_{zpi} are determined using a model of the foundation as a linearly deformable homogeneous isotropic semi-space: $\sigma_{zpi}=P\cdot\alpha$, where α is the stress dissipation factor.

Additional horizontal stresses σ_{xpi} and σ_{ypi} are determined assuming one-dimensional consolidation with a lateral pressure coefficient ξ =0.5.

Summated stress values are calculated from the combination of soil self-weight and additional loads on the foundation (Figure 8).

$$\sigma_{zi} = \sigma_{zgi} + \sigma_{zpi} \tag{9}$$
$$\sigma_{xi} = \sigma_{xgi} + \sigma_{xpi} - \Delta\sigma_{xi} \tag{10}$$
$$\sigma_{yi} = \sigma_{ygi} + \sigma_{ypi} - \Delta\sigma_{yi} \tag{11}$$



a) b)

Figure 5. *a) Structure of stress-strain state of soil under the foundation footing; b) Structure of stress-strain state in elementary soil volume.*

The mean normal stress and stress intensity are calculated as follows:

$$\sigma = \frac{\sigma_{xi} + \sigma_{yi} + \sigma_{zi}}{3} \tag{12}$$

$$\sigma_i = \frac{1}{2}\sqrt{\left(\sigma_{xi} - \sigma_{yi}\right)^2 + \left(\sigma_{yi} - \sigma_{zi}\right)^2 + (\sigma_{xi} - \sigma_{zi})^2 + 6\left(\tau_{xy}^2 + \tau_{zx}^2 + \tau_{zy}^2\right)} \tag{13}$$

When the axes of principal stresses and strains coincide with the central axis of the foundation, we determine the values of volumetric strain and strain intensity.

$$\varepsilon_v = \varepsilon_1 + \varepsilon_2 + \varepsilon_3 \tag{14}$$
$$\Delta\varepsilon_v(t) = \Delta\varepsilon_1 + \Delta\varepsilon_2 + \Delta\varepsilon_3$$
$$\varepsilon_i = \frac{2}{3}(\varepsilon_3 - \varepsilon_1) \tag{15}$$

$$\Delta\varepsilon_i = \frac{2}{3}(\Delta\varepsilon_3 - \Delta\varepsilon_1)$$

In other cases, we use the condition of coaxiality of tensors of incremental stresses and strains:

$$\Delta\tau = \Delta\sqrt{I_{2\sigma}}$$

$$\frac{\Delta(\varepsilon_x - \varepsilon_y)}{\Delta(\sigma_x - \sigma_y)} = \frac{\Delta(\varepsilon_y - \varepsilon_z)}{\Delta(\sigma_y - \sigma_z)} = \frac{\Delta(\varepsilon_z - \varepsilon_x)}{\Delta(\sigma_z - \sigma_x)} = \frac{\Delta\varepsilon}{\Delta\sigma_i} = \chi$$

Values ε_1, ε_2, ε_3 are adopted based on the results of laboratory tests, which correspond to soil stresses at each point according to the soil profile and creep profile documentation.

Conditional instantaneous moduli are defined: K_V – bulk modulus, and G_V – shear modulus, characterizing the transition from the natural state of the foundation to the state after local load application. Instantaneous moduli K_V and G_V take into account the transition at the moment of applying additional vertical load.

$$K_V(t) = \frac{\Delta\sigma}{\Delta\varepsilon_V + \Delta\varepsilon_V(t)}, \quad (16)$$

$$G_V(t) = \frac{\Delta\sigma_i}{3(\Delta\varepsilon_i + \Delta\varepsilon_i(t))}. \quad (17)$$

Increment of axial strain in the *i*-th layer after horizontal soil displacement within the failure wedge is determined by the formula:

$$\Delta\varepsilon_{zi} = \frac{\Delta\sigma_z(t)}{G_V(t)} - \Delta\sigma\frac{3K_V(t) - G_V(t)}{3K_V(t) \cdot G_V(t)} \quad (18)$$

Settlement of the foundation divided into equal layers up to the conditional depth of the compressible layer at time *t* is calculated using the formula:

$$S = \sum_{i=1}^{n}(\varepsilon_{z0} + \varepsilon_{zpl} + \Delta\varepsilon_{zi}) \cdot h_i \quad (19)$$

where:
h_i – thickness of the *i*-th layer;
ε_{z0} – increment of axial strain at the moment of external load application;
ε_{zpl} – increment of axial strain due to creep;
$\Delta\varepsilon_{zi}$ – increment of axial strain of the *i*-th layer considering horizontal soil displacement within the active pressure soil prism;
n – number of layers.

4. CONCLUSION

The research findings reveal a pattern of uneven change in the stress-strain state of soils in the foundation of buildings located at the edge of excavations. The deformation of the soil mass beneath the foundations of buildings situated within the collapse prism occurs non-linearly and inconsistently. Deep excavation works cause changes in the stress-strain state and induce additional vertical, horizontal, and flexural deformations in the soil mass supporting the foundations located at the rim of the excavation. This leads to variations in the ratios of vertical and horizontal stresses, which affect the deformation and rheological characteristics of the soil, ultimately resulting in increased settlement of the foundation of buildings situated at the edge of deep excavations.

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