Influence of the deep pit construction on a settlements of surrounding buildings

I T Mirsayapov and N N Aysin
1 Professor, KSUAE, Kazan, Russia
2 Student, KSUAE, Kazan, Russia
E-mail: mirsayapov1@mail.ru

Abstract. Various methods exist for calculating the subgrade settlements of foundations built on soil subgrade without defects and damages. The physical and mechanical characteristics of such soil subgrade are usually determined using the results of engineering and geological surveys and other methods that characterize their in-situ conditions before construction. It is usually assumed that these characteristics are constant throughout their life cycle regardless whether they get later disturbed by buildings and/or structures. In fact, the mechanical characteristics of the soils of the subgrade vary and are subject of continuous changes due to various factors. Therefore, the use of existing calculation methods often lead to inadequate estimates of settlements of subgrade and foundations of buildings and structures. Assessment of vertical deformations of soil subgrade in the zone of influence of deep pits usually falls in this category. The method for calculating the settlement of bases and foundations of buildings was developed taking into account the formation and development of defects and damages in the soil structure in the zone of influence of a deep pit. It is based on a method of layer-by-layer summation, taking into account the spatial stress state of the soil mass and changes in the mechanical state of the soil due to surcharge conditions.

1. Introduction
During construction of a deep pit, located in a developed urban 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. Forecasting the influence zone of the pit construction should take into account the settlement of the foundations of the surrounding buildings due to diaphragm wall construction [1-4]. The diaphragm wall is used as a temporary support of the excavation for a deep pit. 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 take into account 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 is 20-50 mm. Settlements of existing building foundations located within a new construction zone of influence shall meet as follows:

\[ s_{ad} \leq s_{ad,u} \]  

(1)

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

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 changes at the location of new pit construction near existing buildings [16-20]. In this regard, it is necessary to develop methods for calculating the settlement of subgrade, which take into account the features of deformation under such conditions.

2. Description of a case

2.1. 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. 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 class B25 concrete. 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 building foundation is 4.150 m and strip foundation under the brick walls.

2.2. 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 (within the depth of the survey).

From the surface to 25.0 m depth the geological and lithological structure is represented by the following engineering and geological elements (IGE): Bulk soil (IGE-1), Clay loam (IGE-2), Loam is (IGE-3), Clay (IGE-4), Sandstone (IGE-5).

3. Theory
The deep pit surrounds the surveyed building in the immediate nearness in axes A, 9 and 15. 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 prediction of the subgrade geomechanical behavior must take into account main parameters changes that characterize the stress-strain state of the soil over time.

At the developing analytical equations of soil behavior under regime alternating long-term static loads, the calculation model of soil under three-axis compression developed by I.T. Mirsayapov and I.V. Koroleva [21-23] for regime loads is used as a basis.

![Figure 1](image)

Figure 1. a) Soil elementary volume stress-state 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 in the ultimate equilibrium surfaces in the soil elementary volume.
The accepted spatial model of dilating soil under regime loading 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 at 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 results of the study of soil deformations performed by the authors show that under regime triaxial loads, the modulus of soil deformations and deformations changes due to the formation and development of micro and macro cracks in the equilibrium surfaces. Therefore, to determine the nonlinear part of creep deformation, first of all, it is necessary to establish the relation between the length of micro and macro cracks and the deformation of the soil.

Let’s consider the volume of soil under triaxial loading after the formation of microcracks. We assume that the volume of soil contains some microcracks of an average length of $2l$, equally distributed in the direction of compression. In the initial stage the concentration of microcracks is small and their mutual influence can be ignored.

Let's select a square with side $a$ containing one microcrack with length $l<<a$ and arbitrarily oriented relative to the compression axis. For this square, we calculate the effective deformation characteristics - modulus $E$, longitudinal and transverse deformations $\varepsilon_1$ and $\varepsilon_2$, respectively.

The stress-strain state of the considered square divided into two stages: first, the actual loading with slippage (shear) crack; the second, under loads providing a clutch of crack banks (Fig. 6).

![Figure 2](image-url)

**Figure 2.** Stress-strain state of a square with a crack under compression: a) the actual and stressed state; b) the conditional stress state

Applying the reciprocity theorem of work on deformations produced by two forces:

$$
\sigma \cdot a \cdot \Delta \sigma = \sigma_1 \cdot a \cdot \Delta \sigma \int_{-l}^{l} \tau (u_1^+ - u_1^-) dx,
$$

where

$$
\tau = \sigma_1 \cdot \sin \beta \cdot \cos \beta;
$$

$$
u_1^+ - u_1^- = \frac{x + 1}{4\mu} \cdot \sigma \{\sin 2\beta - \rho(1 - \cos 2\beta)\} \cdot \left(l^2 - x^2\right)^{1/2};
$$

where

- $\rho$ - ground friction coefficient;
- $\mu = \frac{E_b}{2(1+\varphi)}$ - Lamé constant;
- $\gamma$ – Poisson’s ratio.

When the banks of the crack slip, the modulus of soil deformation decreases and the conditional modulus is realized, and when loading with conditions that ensure the adhesion of the banks, the initial elastic modulus is realized. Then:

$$
\Delta \sigma_1 = \frac{\sigma_1 a}{E_{b0}}, \quad \Delta \sigma = \frac{\sigma a}{E_{bt}}.
$$
Taking into account (5), we will rewrite the expression (2) as:

$$\sigma \cdot a = \frac{\sigma_1 a}{E_{b0}} = \frac{\sigma_1 a}{E_{b0}} - \int_{-l}^{l} \tau (u_1^- - u_1^+) dx.$$  \hspace{1cm} (6)

After some transformations, equation (6) is given in the form:

$$\frac{1}{E_{b0}} = \frac{1}{E_{b0}} + \int_{-l}^{l} \tau (u_1^- - u_1^+) \frac{\sigma_1 a^2}{\sigma_1 a^2} dx.$$  \hspace{1cm} (7)

Taking into account (4) from the expression (7), we obtain the value of the soil deformation modulus for a square with a crack:

$$E_{b0} = E_{b00} \left[ 1 + \frac{2\pi l^2}{a^2} \cdot \sin \beta \cdot \cos \beta \left( \sin \beta \cdot \cos \beta - \rho \cdot \sin^2 \beta \right) \right]^{-1}. \hspace{1cm} (8)$$

Assume that the considered soil array consists of a large number \((m)\) of the above squares, located one after the other and forming a sufficiently long strip. At the same time, we consider that the distribution of crack directions in these squares is equally probable.

Calculate the mean longitudinal deformations of the considered strip under uniaxial \(\sigma\) compression:

$$\varepsilon_{l} = \frac{\sigma}{m} \left\{ \frac{\sigma}{E_{b0}} + \int_{0}^{\frac{\pi}{2}} \left[ 1 + \frac{2\pi l^2}{a^2} \cdot \sin \beta \cdot \cos \beta \left( \sin \beta \cdot \cos \beta - \rho \cdot \sin^2 \beta \right) \right] d\beta - \frac{m}{2} \int_{0}^{\frac{\pi}{2}} d\beta \frac{2m}{\pi} \right\}, \hspace{1cm} (9)$$

where \(a = \arctg \frac{1}{\rho}\).

When writing equation (9), the summation of individual squares is replaced by integration, since the average step of changing the direction of cracks is assumed to be small.

Calculating the integral in (9), we obtain the final equation for the nonlinear part of the creep deformations:

$$\varepsilon_{pl}^{(n)}(t) = \frac{\sigma_1}{E_{b0}} \left\{ \frac{\sigma_1}{2l_{cr}^2} \left( \arctg \frac{1}{\rho(t,t)} - \frac{\rho(t,t)}{1 + \rho^2(t,t)} \right) \right\} \hspace{1cm} (10)$$

where \(l_{cr} = \frac{k_2}{\pi n_{cr}}\) – critical crack length;

\(l(t,t)\) – total length of cracks in the material;

\(\rho(t,t)\) – variable coefficient of friction of the soil.

As can be seen from (10), nonlinear creep deformations mainly depend on the total length of macro-cracks.

### 3.1. Development of the nonlinear part of creep deformations under various regimes of long-term static loading

Due to the fact that the development of the nonlinear part of creep deformations depends on the laws of development of micro- and micro-cracks, it is necessary to distinguish nonlinear creep deformations corresponding to the initial and main stages of crack development in the soil body. At the initial stage, the development of the nonlinear part of deformations is described by the laws of microcracks development. The main stage for the development of the nonlinear part of creep deformations corresponds to the stage of development of microcracks in the soil body and is described by the laws of development of microcracks. Then the equations of nonlinear deformations of soil creep under stationary and non-stationary regimes of long-term static loading at various stages of crack development in the material have the form:

a. For initial stage of stationary loading regime:

$$\varepsilon_{pl}^{(n)}(t,t) = \frac{\sigma_{pl}^{max}}{E_{gr(t)}} \left\{ \arctg \frac{1}{\rho(t,t)} - \frac{\rho(t,t)}{1 + \rho^2(t,t)} \right\} \hspace{1cm} (11)$$

\(l(t_0) + \left\{ l_m(k_{gr(t)}) \left( \frac{\sigma_{pl}^{max}}{E_{gr(t)}} \right)^2 \right\} \hspace{1cm} (11)

where \(\Delta N_1 = N_1 - N_{l2t_k}\);
Nonlinear creep deformations in a system of developed cracks in the material are described based on

\[ N_{ult} \]  
- duration of the incubation period, days;

b. For main stage of stationary loading regime:

\[ \varepsilon_{pl}^*(t, T) = \frac{\sigma_{max}^{Gr}}{E_{Gr}(t_0)} \left[ \left| (l(t_0T)) + \sum_{i=1}^{m} \tau(t_i) \Delta N_i \right|^2 \left( \arctg \frac{1}{P} - \frac{P}{1+P^2} \right) \right] \cdot \left( \arctg \frac{1}{P} - \frac{P}{1+P^2} \right). \]

\[ \left\{ \begin{array}{c}
\frac{\Delta N_i}{N_i} = 1 - N_{ult} \Delta N_i ;
N_i \text{ duration of loading at the initial stage of development of cracks in the ground at } (l(t, T) < 2 ds; \]
\[ N \text{ total duration of loading.} \]

c. For initial stage of consistently increasing regime of non-stationary long-term static loading:

\[ \varepsilon_{pl}^*(t, T) = \frac{\sigma_{max}^{Gr}}{E_{Gr}(t_0)} \left[ \left( \arctg \frac{1}{P} - \frac{P}{1+P^2} \right) \right] \cdot \left( \arctg \frac{1}{P} - \frac{P}{1+P^2} \right). \]

\[ \left\{ \begin{array}{c}
\frac{\Delta N_i}{N_i} = 1 + \sum_{i=1}^{y} N_i - N_{ult} \Delta N_i ;
y \text{ number of increasing blocks (stages) of loading during which the crack can be classified as microcracks;}
N_{ult} \text{ duration of the incubation period, days;}
\end{array} \right. \]

d. For main stage of consistently increasing regime of non-stationary long-term static loading:

Nonlinear creep deformations in a system of developed cracks in the material are described based on the expressions, taking into account the influence of previous loading stages, which is manifested in changes in \( k_i, \Delta W_{npi(i)} \), i.e., in the preliminary plastic deformation of microelements in the pre-fracture zone and an increase in the crack length at the previous stage:

\[ \varepsilon_{pl}^*(t, T) = \frac{\sigma_{max}^{Gr}}{E_{Gr}(t_0)} \left[ \left| (l(t_0T)) + \sum_{i=1}^{m} \tau(t_i) \Delta N_i \right|^2 \left( \arctg \frac{1}{P} - \frac{P}{1+P^2} \right) \right] \cdot \left( \arctg \frac{1}{P} - \frac{P}{1+P^2} \right). \]

\[ \left\{ \begin{array}{c}
\frac{\Delta N_i}{N_i} = 1 + \sum_{i=1}^{y} N_i - N_{ult} \Delta N_i ;
y \text{ number of increasing blocks (stages) of loading during which the crack can be classified as microcracks;}
N_{ult} \text{ duration of the incubation period, days;}
\end{array} \right. \]
e. For initial stage of consistently decreasing regime of non-stationary long-term static loading. The equation of nonlinear creep deformations at the initial stage is written taking into account the delay in the development of cracks:

$$
\varepsilon_{pli}^n(t, t) = \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \frac{1}{\frac{2l}{u}} \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] = \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \frac{1}{\frac{2l}{u}} \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right],
$$

$$
\left\{ \begin{aligned}
\frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} & \left[ \frac{1}{\frac{2l}{u}} \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] \\
\int_{m(k_{gr}R_{gr,t})}^{m_1^2(t,t)} & m_t^2(t, t)\left[ \frac{1}{E_{gr}(t_0)} \right] \left[ \frac{1}{E_{gr}(t_0)} \right] + C_0 \Pi_{k=1}^{k_{gr}} k_{gr} \psi_{\theta_i} \right] \right] .
\end{aligned} \right.
$$

$$
\sum_{i=1}^{n} \Delta N_i + l(t_0 t) = \sum_{i=1}^{n} \Delta N_i \left[ \frac{1}{E_{gr}(t_0)} \right] \left[ \frac{1}{E_{gr}(t_0)} \right] + C_0 \Pi_{k=1}^{k_{gr}} k_{gr} \psi_{\theta_i} \right] \right] .
$$

$$
\sum_{i=1}^{n} \Delta N_i = \sum_{i=1}^{n} N_i - N_{ultk} - N_D ;
$$

$$
\Delta N_0 - \text{exposure time, days;}
$$

$$
N_{ult} - \text{duration of the incubation period, days;}
$$

$$
N_D - \text{days of} \ n_{ultk}.
$$

f. For main stage of consistently decreasing regime of non-stationary long-term static loading:

$$
\varepsilon_{pli}^n(t, t) = \sum_{i=1}^{n} \sum_{i} \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] = \sum_{i=1}^{n} \sum_{i} \left[ \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \frac{1}{\frac{2l}{u}} \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] \right],
$$

$$
\left\{ \begin{aligned}
\frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} & \left[ \frac{1}{\frac{2l}{u}} \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] \\
\int_{(k_{gr}R_{gr,t})}^{m_1^2(t,t)} & m_t^2(t, t)\left[ \frac{1}{E_{gr}(t_0)} \right] \left[ \frac{1}{E_{gr}(t_0)} \right] + C_0 \Pi_{k=1}^{k_{gr}} k_{gr} \psi_{\theta_i} \right] \right] .
\end{aligned} \right.
$$

$$
\sum_{i=1}^{n} \Delta N_i = \sum_{i=1}^{n} N_i - N_i - N_{ultk} - \sum_{i=1}^{n} N_D .
$$

3.2. The equation for the total inelastic deformations

The total creep deformations depending on the stage of crack development in the soil and the regime of long-term static loading are described by following equations:

a. For initial stage of stationary loading:

$$
\varepsilon_{pli}^n(t, t) = \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] + \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right],
$$

$$
\sum_{i=1}^{n} \Delta N_i = \sum_{i=1}^{n} N_i - N_i - N_{ultk} - \sum_{i=1}^{n} N_D .
$$

b. For main stage of stationary loading:

$$
\varepsilon_{pli}^n(t, t) = \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] + \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right] + \frac{\sigma_{gr}^{max}}{E_{gr}(t_0)} \left[ \left( \arctg \left( \frac{1}{P(t, t)} - 1 + \frac{P(t, t)}{2} \right) \right) \right],
$$

$$
\sum_{i=1}^{n} \Delta N_i = \sum_{i=1}^{n} N_i - N_i - N_{ultk} - \sum_{i=1}^{n} N_D .
$$
where $A^* = \left(k_{pgr}, R_{grr}, t\right)^2 \cdot m_j^2(t, \tau) \left[\frac{1}{E_{grr(t)}} + C_0 \prod_{k=1}^{k=g} k_k \alpha \varphi_{\theta i}\right]$;

c. For initial stage of consistently increasing regime of non-stationary long-term static loading:
\[
e_p(t, \tau) = \sum_{i=1}^{N_1} C_0 \prod_{k=1}^{k=g} k_k \alpha \varphi_{\theta i} \cdot \sigma_{grr}^{max} (1 - \mathcal{P}_1) \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] + \sum_{i=2}^{n} N_i (C_0 \prod_{k=1}^{k=g} k_k) \cdot \alpha \varphi_{\theta i} \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] \Delta \sigma_{gri} + \sum_{i=1}^{n} \mathcal{P}_i \sigma_{grr}^{max} c_{oo}(t, \tau) f_i(t, \tau) + \sum_{i=1}^{n} \sum_{i=1}^{N_i} \sigma_{gri}^{max} \cdot \left(\arctg \frac{p}{1 + p^2} \right) \right] \frac{\Delta N_1 + l(t(\tau)) + l_i(t(\tau))}{2 l_u^2} \right]^2;
\]

d. For main stage of consistently increasing regime of non-stationary long-term static loading:
\[
e_p(t, \tau) = \sum_{i=1}^{N_1} C_0 \prod_{k=1}^{k=g} k_k \alpha \varphi_{\theta i} \cdot \sigma_{grr}^{max} (1 - \mathcal{P}_1) \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] + \sum_{i=2}^{n} N_i (C_0 \prod_{k=1}^{k=g} k_k) \cdot \alpha \varphi_{\theta i} \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] \Delta \sigma_{gri} + \sum_{i=1}^{n} \mathcal{P}_i \sigma_{grr}^{max} c_{oo}(t, \tau) f_i(t, \tau) + \sum_{i=1}^{n} \sum_{i=1}^{N_i} \sigma_{gri}^{max} \cdot \left(\arctg \frac{p}{1 + p^2} \right) \right] \frac{\Delta N_2}{l(t(\tau)) + \frac{1}{E_{grr(t)}} \frac{m_j^2(t, \tau) \left[\frac{1}{E_{grr(t)}} + C_0 \prod_{k=1}^{k=g} k_k \alpha \varphi_{\theta i}\right]}{2 l_u^2} \right]^2;
\]

where $A^* = \left(k_{pgr}, R_{grr}, t\right)^2 \cdot m_j^2(t, \tau) \left[\frac{1}{E_{grr(t)}} + C_0 \prod_{k=1}^{k=g} k_k \alpha \varphi_{\theta i}\right]$;

e. For initial stage of consistently decreasing regime of non-stationary long-term static loading:
\[
e_p(t, \tau) = \sum_{i=1}^{N_1} C_0 \prod_{k=1}^{k=g} k_k \alpha \varphi_{\theta i} \cdot \sigma_{grr}^{max} (1 - \mathcal{P}_1) \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] + \sum_{i=2}^{n} N_i (C_0 \prod_{k=1}^{k=g} k_k) \cdot \alpha \varphi_{\theta i} \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] \Delta \sigma_{gri} + \sum_{i=1}^{n} \mathcal{P}_i \sigma_{grr}^{max} c_{oo}(t, \tau) f_i(t, \tau) + \sum_{i=1}^{n} \sum_{i=1}^{N_i} \sigma_{gri}^{max} \cdot \left(\arctg \frac{p}{1 + p^2} \right) \right] \frac{\Delta N_1 + l(t(\tau)) + l_i(t(\tau))}{2 l_u^2} \right]^2;
\]

f. For main stage of consistently decreasing regime of non-stationary long-term static loading:
\[
e_p(t, \tau) = \sum_{i=1}^{N_1} C_0 \prod_{k=1}^{k=g} k_k \alpha \varphi_{\theta i} \cdot \sigma_{grr}^{max} (1 - \mathcal{P}_1) \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] + \sum_{i=2}^{n} N_i (C_0 \prod_{k=1}^{k=g} k_k) \cdot \alpha \varphi_{\theta i} \left[1 + (1 - \alpha \varphi_{\theta i})^{N_{i-1}}\right] \Delta \sigma_{gri} + \sum_{i=1}^{n} \mathcal{P}_i \sigma_{grr}^{max} c_{oo}(t, \tau) f_i(t, \tau) + \sum_{i=1}^{n} \sum_{i=1}^{N_i} \sigma_{gri}^{max} \cdot \left(\arctg \frac{p}{1 + p^2} \right) \right] \frac{\Delta N_1 + l(t(\tau)) + l_i(t(\tau))}{2 l_u^2} \right]^2;
where

\[ A^* = \left( k_{R_{gr,t}} R_{gr,t} \right)^2 \cdot m^2(t, \tau) \left[ \frac{1}{E_{gr}(t)} + C_0 \prod_{k=1}^{k=g} k_k a \psi_{\theta_j} \right] \]

After determining the vertical deformations of the soil under the considered loading regimes, the settlement of the base is determined.

**Figure 3.** The stress state of the foundation base under regime long-term loading.

The settlement of the base divided to the conditional depth of the compressible thickness is calculated by layer-by-layer summation:

\[ S = \sum_{i=1}^{n} \left[ \varepsilon_{i0} + \varepsilon_{P_{li}}(t, \tau) \right] \Delta h_i \]

**4. Results**

Based on the above methodology, numerical studies of the stress-strain state of the soil base of the foundations of the building 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 the analytical calculation soil model taking into account the formation and development of micro and macro cracks. The obtained calculation results are shown in fig. 8, 9, which confirm the general picture of deformation of the building under study.

**Figure 4.** Vertical movement (mm) of building foundations.
Figure 5. Vertical movement of the soil mass.

5. Discussions

Numerical studies have shown that both vertical and horizontal deformations occur under the building foundations. The zone where deformations exceed 5 mm encompasses almost half of building. The deformations of the subgrade are cause of deformations of the entire building frame and impose additional stresses on the building structural elements. The maximum horizontal and vertical movements of the foundations were 40 and 70 mm respectively.

To summarize, the analytical and numerical methods used to arrive at foundation settlements result in an acceptable level of convergence with related geo-monitoring data, while taking into account the influence of the deep pit excavations within the adopted numerical model. To derive acceptable foundation settlement results, (being within slight/acceptable deviation from the measured ones), it is necessary to take into account the rigidity of the overlying structures, as well as the influence of the pit wall. Calculations could be performed using finite element method software and represent numerical model three-dimensionally to include the entire building or only the underground part. In the calculation model, the foundation could be considered resting on an elastic base model. To account for the plastic properties of soil, changes in the volumetric stress-strain state of soil, creep, unloading, long-duration surcharge load, and formation and development of micro and macro cracks detected in the soil base, it is recommended to use the proposed analytical model that takes into account changes in volume and shear modulus of the base soil under triaxial regime loading.

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