Settlement and bearing capacity of soil basis near vertical excavation

Armen Ter-Martirosyan[1] and *Yuliya* Vanina[1*]

[1] Moscow State University of Civil Engineering, Yaroslavskoe shosse, 26, Moscow, 129337, Russia

Abstract. In the present paper, there was provided development of the method determining the statement and analytical solution of a problem which evaluates stress statement, settlement and load-bearing capacity of the weighty soil layer of limited thickness that is rested upon an incompressible soil basis and the excavation pit wall on exposure to a foundation with the distributed load in the vicinity of a wall. The analytical solution for determining the stress state component in an engineering problem is based on the Ribere-Fileon trigonometric series. Analytical solutions for determining the settlements of the soil bases based on nonlinear dependencies, taking into account the nonlinear elastic-plastic deformation properties of soils, as well as nonlinear rheological properties, are presented The obtained solutions enable to assess the deformation of soil basis and load-bearing capacity with respect to non-linear properties that accurately corresponds to the actual behaviour of soil basis under the loading. The results of stress statement were verified by numerical modelling in PC PLAXIS 2D to prove their validity and accuracy.

Keywords: soil basis, retaining wall, elastoplastic properties of soils, viscosity of soil, long-term bearing capacity

1 Introduction

In conditions of dense urban development during the construction of underground transport infrastructure by an open method, as well as underground parts of buildings and structures, including unique and high-rise ones, the most important problem of design is to ensure the safety and possibility of further operation of objects located in the zone of influence of new construction. When an underground structures of a high-rise building interacts with the adjacent soil massif, near a retaining wall of excavation pit and the foundation, heterogeneous stress-strain state arises that is transformed in space – either during construction stage or during the building's operation. Ignoring the elastoplastic and viscoplastic behaviour of the soils in the conditions of finding a building or structure near the excavation pit can lead to a discrepancy between the calculated and actual precipitation and rolls, which entails a failure to ensure the safety and further operation of existing

* Corresponding author: yuli.jullies@gmail.com

buildings and structures. Analytical methods for calculating such problems in a linear and elastic formulation make it possible to take into account these factors and bring solutions to simple formulas used in the design in the first approximation.

The methods of determination of settlements and long-term settlements of soil basis have been developed by following scientists [1-9]. Let us emphasize the works Terzagi and Peck [10], Schmertmann [11], Schultze E. and Sherif, G [12], Janbu N. [13], where the methods for calculating the settlement of the soil basis based on standard penetration tests are defined. Most of these formulas [10-13] contain coefficients, which are determined empirically and depend simultaneously on a large number of factors, which can negatively affect the degree of reliability of the results.

In this paper, the authors have proposed the analytical solution that allows taking into account at the same time a large range of factors that form the stress-strain statement the weighty soil layer of a limited thickness resting upon an incompressible soil basis and the retaining wall of excavation pit with the distributed load near the wall, such as the distance of the building from the pit, the load on the foundation, the dimensions of the foundation for stress-strain state of. To account for the elastoplastic and elastic-viscoplastic deformation properties of soils in the conditions of finding a building or structure near the pit, authors used calculation dependences described below. An author's analytical solution was verified by of numerical modeling of an engineering problem, widely used in the practice of designing bases and foundations of buildings and structures.

2 Methods

2.1 Determination of components of stress statement for the boundary value problem.

The boundary value problem provides the action of the distributed load *q=const* with width *b=2a* at the distance *d* from the edge of retaining wall structure on the soil layer rested upon an incompressible soil basis. The design scheme is shown on Figure 1. The components of stress statement of soil basis are determined by the method, based on the Ribere-Faylon trigonometric series and developed by Z.G. Ter-Martirosyan [16-19].

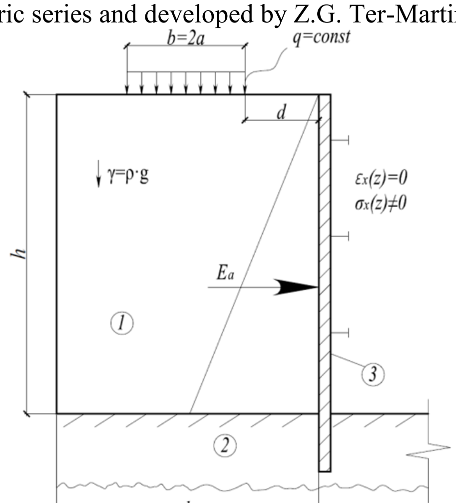

Fig. 1. The design scheme of the interaction of a weighty layer (1) with thickness (*h*) resting upon an incompressible soil base (2) with a vertical fixed retaining wall (3) under a distributed load *q = const* along the strip *b= 2a* at a distance *d* from the retaining wall.

2.2 The system of the Hencky's physical equations

The constitutive system of the Hencky's physical equations [20] allows determining dependencies of stresses on strains has the following form:

$$\varepsilon_x = \chi(\sigma_x - \sigma_m) + \chi^* \cdot \sigma_m ; \gamma_{xy} = 2\chi \cdot \tau_{xy} \tag{1}$$

$$\varepsilon_y = \chi(\sigma_y - \sigma_m) + \chi^* \cdot \sigma_m ; \gamma_{yz} = 2\chi \cdot \tau_{yz} \tag{2}$$

$$\varepsilon_z = \chi(\sigma_z - \sigma_m) + \chi^* \cdot \sigma_m ; \gamma_{zx} = 2\chi \cdot \tau_{zx} \tag{3}$$

where

$$\chi = \frac{\gamma_i}{2\tau_i} = \frac{f(\tau_i, \sigma_m, \mu_\sigma)}{2\tau_i}; \tag{4}$$

$$\chi^* = \frac{\varepsilon_m}{\sigma_m} = \frac{f^*(\tau_i, \sigma_m, \mu_\sigma)}{2\tau_i}; \tag{5}$$

Also, the system of physical equations of Hencky [19, 20] allows to determine dependencies between stresses and strain rates.

Determination of settlement and load-bearing capacity can be implemented in the following manner:

$$\varepsilon_z = \frac{\sigma_z - \sigma_m}{G(\sigma_m, \tau_i / \tau_i^*)} + \frac{\sigma_m}{K}; \tag{6}$$

where $G(\sigma_m, \tau_i / \tau_i^*)$ and $K(\sigma_m)$ – moduli of shear and volumetric strains depending on the average stress σ_m, as well as the relationships between the acting τ_i and the ultimate value τ_i^* of shear stress intensity, i.e. τ_i / τ_i^*, where $\tau_i^* = \sigma tg\varphi + c$.

2.3 Analytical elastoplastic and rheological models of soil basis

The dependency proposed by the academician S.S. Grigoryan was assumed as an analytical model for determining non-linear volumetric strains [15]. It looks as follows:

$$\varepsilon_m(\sigma_m) = \varepsilon^*(1 - e^{-\alpha\sigma_m}) \tag{7}$$

where ε is the volumetric deformation;

ε^* is the limit value of volumetric deformation, achieved at $\sigma_m \to 0$;

α is an experimental constant. For $\sigma_m \to \infty$; $\varepsilon_m \to \varepsilon^*$, and for $\alpha=0$, $\varepsilon^* = \varepsilon_m$, we obtain a linear dependence on $K = \varepsilon_m / \sigma_m$

To describe elastoplastic properties of cohesive soil exposed to shear loading, the dependency proposed by S.P. Timoshenko [5] has the following form in regards to soil grounds:

$$\gamma_i = \frac{\tau_i}{G^o} \frac{\tau_i^*}{\tau_i^* - \tau_i} \tag{8}$$

where γ_i; τ_i; τ_i^*; G_0 are determined from standard triaxial soil tests. A detailed description of the parameters of this analythical soil model is given in the scientific work [18]

Inserting $G(\sigma_m, \tau_i)$ and $K(\sigma_m)$ we obtain non-linear equations:

$$\varepsilon_{z,v} = \varepsilon^*(1 - e^{-\alpha\sigma_m}) \tag{9}$$

$$\varepsilon_{z,\gamma} = \frac{\sigma_z - \sigma_m}{2G_e(\sigma_m, \tau_i / \tau_i^*)}; \tag{10}$$

When $\tau = const$, the new rheological model, developed by the staff of the scientific and educational center "Geotechnics" [14], can be described by the following equation:

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$$\dot{\gamma} = \frac{\tau - \tau^*}{\eta_\gamma(\sigma_m)}\left(\frac{e^{-\alpha\gamma}}{a} + \frac{e^{\beta\gamma}}{b}\right) \tag{11}$$

A detailed description of the parameters of this rheological model is given in the scientific work [14].

The Kelvin - Voigt viscoelastic model [15] allows to determine nonlinear volumetric deformations velocities, taking into account the rheological properties of soil:

$$\dot{\varepsilon}_m(t) = \frac{\sigma_m}{K(\sigma_m)} \cdot \left(\frac{-K}{\eta_v} \cdot e^{\frac{-K}{\eta_v}}\right) \tag{12}$$

where η_v is the volumetric viscosity, t – time, K – volumetric deformation modulus.

Dependences between stresses and strain rates were identified as follows:

$$\dot{\varepsilon}_z = \frac{\sigma_z - \sigma_m}{\eta_\gamma(\sigma_m)} \cdot \left(\frac{e^{-\alpha\varepsilon_z}}{a} + \frac{e^{\beta\varepsilon_z}}{b}\right) + \frac{\sigma_m}{K(\sigma_m)} \cdot \left(\frac{-K}{\eta_v} \cdot e^{\frac{-K}{\eta_v}}\right) \tag{13}$$

3 Results

Calculation of stress statement of soil basis were made in the software complex MathCAD along the entire plane at *z>0* and ±*x* according to the computational scheme (see Figure 1). The stress components were obtained for different d_1=6 m d_2=2 m. The results for d_1=6 m are presented on Figures 2-4.

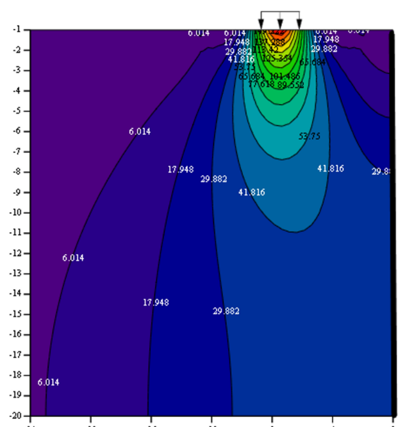

Fig. 2. Vertical stresses σ_y at q = 100 kPa, d_1 = 6 m, and $b = 2a$ = 3 m

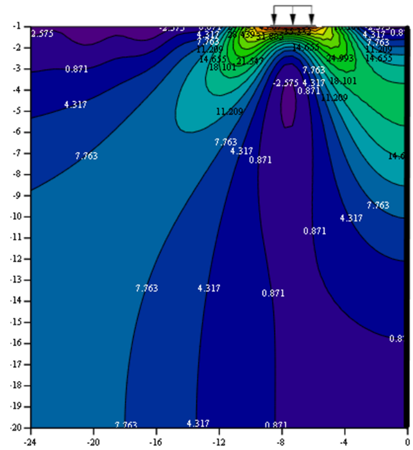

Fig. 3. Horizontal stresses σ_x at q=100 kPa, d_1=6m, and b=*2a*=3m

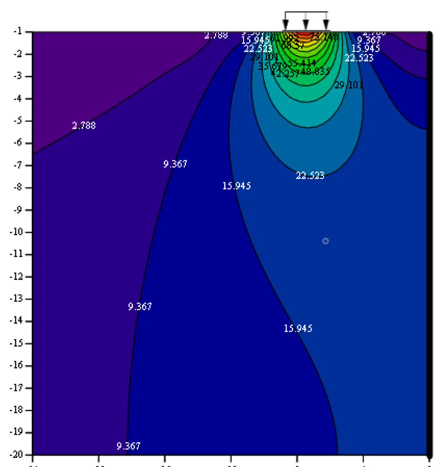

Fig. 4. Mean stress σ_m at q = 100 kPa, d_1 = 6 m, and b = *2a* = 3 m

To compare the vertical stresses obtained when solving the problem of stress distribution in a soil array under the distributed load at a distance from the retaining wall based on solving the problem of stress distribution in a soil basis of finite width and thickness, based on an incompressible base, under the influence of a distributed load at a distance from the pit fence, as well as when solving the problem in PC PLAXIS 2D stress isolines were constructed along the depth in the centre of the distributed load. The results were compared with the parameters of the problem q= 100 kPa, d_1=6 m, b=*2a*=3 m. The results are shown in the figure 5

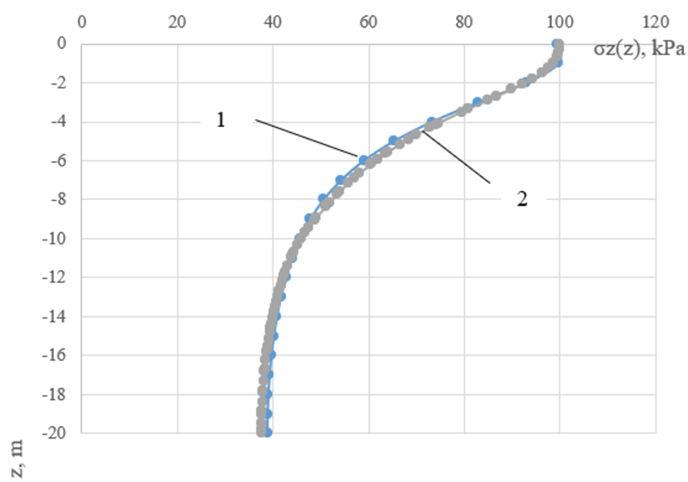

Fig. 5. The distribution of vertical stresses σz obtained by the method of trigonometric Ribiere-Fileon series (blue graph, № 1) and by numerical solution in PC PLAXIS 2D (gray graph, № 2).

For comparison of settlement paths, there were assumed the following distances from the retaining wall to the point of distributed load application d_1=6 m, d_2=2 m. The parameters of mechanical properties of soils were assumed as ε^* = 0.082, α = 0.007, ν = 0.26, Go = 12000 kPa, φ = 29˚ и c = 12 kPa. The results are presented in the Figure 6.

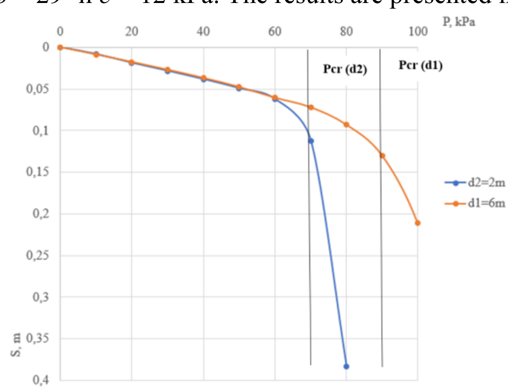

Fig. 6. Curves of the dependency between the total settlement $S(q)$ in the center of the foundation, calculated using equations (9-10) and the load $P=q$ at different distances d_1 = 6 m, d_2= 2 m from the enclosing wall to the distributed load.

Comparing the precipitation graphs vertically corresponding to the center of the distributed load, at a depth within the compressible thickness assumed to be 5 m, it is obvious that the area of the precipitation plot obtained by the solution (Formula 6) is smaller than by the solution obtained in PLAXIS 2D (Figure 7).

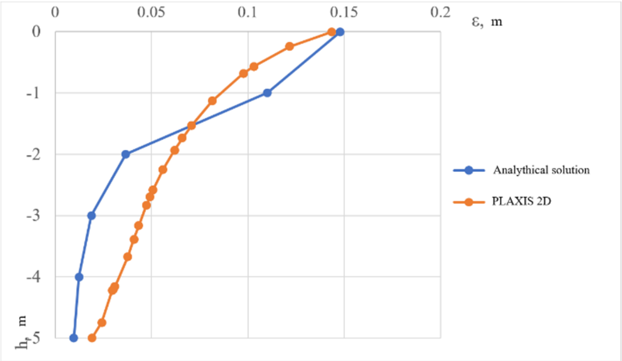

Fig. 7. The plots of the ε_z over the depth of the compressible thickness *h*.

The rheological parameters [15] were used to make curves, describing the creep of compressed soil base layers over time shown on figure 7 (Formulas 11-13). There were obtained graphs, describing the *S'(t) – t* dependence for different values of loading coming from foundation *P=q* (Figure 8) with graph of the long-term stability of the soil basis. The loading values are from100 kPa to 250 kPa.

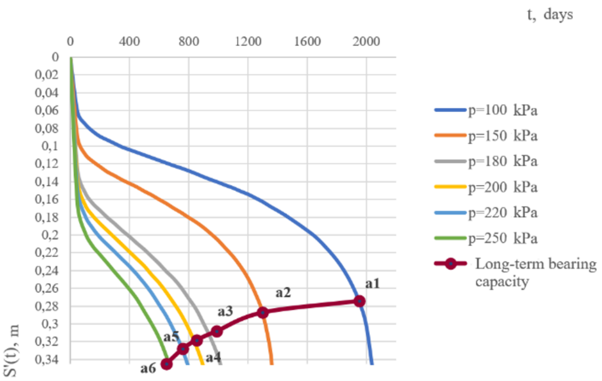

Fig. 8. Total soil basis long-term settlements S'(t)–t for different loadings from the foundation *p=q* with graph of the long-term stability of the soil basis.

4 Discussion

The calculation of deformations showed that when located near the vertical excavation of the foundation under the distributed load at a distance of d_2 = 2 m, the total precipitation of the base is 1.3 times greater than at a distance of d_1 = 6 m. This is caused by the displacement of the horizontal stress isofields towards the side of the pit (Figures 3), which increases the stress deviator at the point of the soil mass (the difference between vertical and horizontal stresses), leading to the achievement of a critical value of τ_i*. Thus, when designing pits, it is necessary to take into account the distance from the edge to the surrounding buildings and structures in order to avoid the development of unstable and excessive deformations of soil basis and foundations, as well as the loss of their bearing capacity. In order to predict the long-term settlement and bearing capacity of the soil base of buildings and structures located near the retaining wall in cases of conservation of the construction of underground parts of buildings and structures a computational rheological model based on the joint use of the new viscoplastic model [14] and the elastic-viscous Kelvin-Voigt model as part of the constitutive system of physical equations of Hencky [20].

5 Conclusion

An analytical solution makes it possible to determine the components of the stress-strain statement of a soil layer resting on an incompressible soil basis under foundation with a distributed load near the retaining wall. This solution, along with numerical calculation methods, can be applied in practice when calculating the precipitation of buildings of nearby buildings, as well as taking into account additional loads from buildings and structures when desighning a retaining wall.

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