

Analysis of slope at increase of the static load

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Summary. Based on the example of the pilot area in Kiev the influence of the increased static load on the superstructure of the stress-strain state of the slope was studied. The efficiency of the proposed methodology when considering the work of "home-slope-retaining structure" depending on natural and anthropogenic factors was demonstrated.

Key words: slope, soil, landslide-prone areas, finite element method, retaining wall.

INTRODUCTION

The construction industry constantly needs to attract an increasing number of potential sites for the design and construction of buildings for various purposes [1-4]. This requires an urgent need for land use that in the recent past was not considered as possible to use for construction sites at all. These include heavily eroded areas and landslide-prone areas. Efficient and safe use of areas within the impact structures on slopes requires determination of stress-strain state (SSS) as the action of soil separate weight and with additional static loading (e.g. impact of the house that is being projected or an addition to the existing building). Regulations require consideration of the stress-strain state and determination its slope stability factor K_{st} the value of which depends on many natural and

man-made factors. K_{st} limit values for basic load combinations influence the choice of anti protective structures and their location on the plan [1]. The actual value of K_{st} can only be found out while identifying the location of a potential slip surface (PSS), which in most cases has a complex configuration of [6]. In practice, the most accurate geometrical form of PSS can be received by using instrumental networks (eg inclinometers). But the efficient, sustainable use of such equipment is only possible by immersing them in a potentially weakened areas within the geotechnical elements (GTE) of a landslide-prone slope [5, 7]. Therefore, the solution of such a difficult, complex task, as a true picture of changing SSS of the landslide-prone slopes on condition of the additional static load is possible only within the nonlinear soil mechanics using numerical calculation methods [9-13].

PURPOSE OF WORK

To consider the effect of increasing of the existing superstructure static load on the stress-strain state of "house-slope-retaining structure." To evaluate the proposed methodology at various stages of loading of such a system.

NUMERICAL SOLUTION PROCEDURE

The most common and versatile numerical method for solving this class of problems is the ITU [12]. Under this method in order to consider SSS moment ITU scheme is used. It ensures that no deformation takes place when moving rigid body as a whole and excludes (removes) the problem of "false bias" [12]. This is particularly important for the calculation of the slopes where soil destruction in the local area still does not lead to its global displacement.

Deformation of the soil environment is characterized by the simultaneous occurrence of a large number of complex physical and mechanical processes. Many modern nonlinear models of the soil environment are oriented at solving this class of problems through the use of or a significant number of input parameters, or vice versa unduly minimizing their numbers. But the benefits of a model is primarily grounded by the minimum number of input parameters, simplicity and reliability of their definition in accordance with the current regulatory literature. In the present program complex SATER.SOIL a modified model of soil protection is used, which is based on the dilatation theory by V.N. Nikolayevskii [8, 9]. It has a theoretical and experimental basis, a clear physical interpreting and limited required number of known input parameters of the base. The criterion limiting condition for this complex is modified condition of Mises-Schleicher-Botkin.

Let us consider the vertical section of the site with the appropriate configuration of surface and defined physical and mechanical properties of soil. In order to describe the geometry and topology of the element of the dispersed environment basic coordinate system is introduced. Along with the baseline local curvilinear coordinate system for the solutions is introduced (Fig.1).

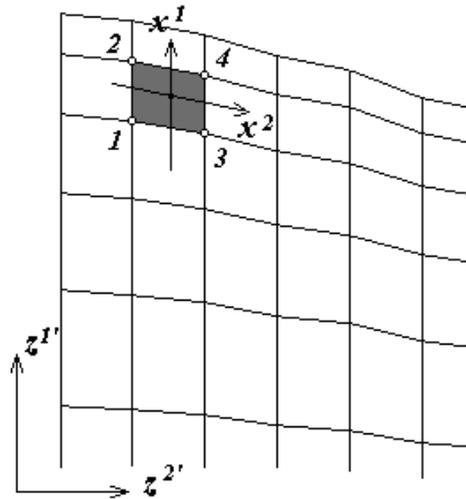


Fig. 1. Cross section of the soil with local and global coordinate systems

The center of the local coordinate system is associated with the elementary volume environment. The relationship between global and local coordinate systems is defined by the coordinate transformation of the tensor whose components are represented by the following relations:

$$c_i^{k'} = \frac{\partial z^{k'}}{\partial x^i}, \quad i, k = 1, 2 \quad (1)$$

At the deformation of the volume of the soil environment every point gets movement, that is characterized by a vector, which is represented by the covariant components in the local coordinate system. Strain tensor is represented by covariant components that are submitted by the components of the displacement vector:

$$\varepsilon_{ij} = \frac{1}{2} \left(c_j^{k'} y_{k',i} + c_i^{k'} y_{k',j} \right),$$

$$y_{k',i} = \frac{\partial y_{k'}}{\partial x^i}, \quad i, j, k = 1, 2 \quad (2)$$

In terms of elastoplastic deformation calculation of soil environment plastic deformations in the boundary condition is

based on the associated law of plastic flow theory. It is assumed that increases of the strain tensor $d\varepsilon_{ij}$ are equal to the sum of elastic increments $d\varepsilon_{ij}^e$ and plastic $d\varepsilon_{ij}^p$ components. Elastic deformation are small and associated with stresses were generalized by the Hooke's law. According to current theories of law associated plastic deformation develop to the normal of the surface fluidity [8].

$$d\varepsilon_{ij}^p = d_F \frac{dF}{d\sigma^{ij}} \quad (3)$$

where d_F – small scalar multiplier, F – function of fluidity, $F = 0$ – law of the limiting soil conditions, σ^{ij} – contravariant components of the stress tensor in the local coordinate system. As the surface fluidity we accept generalized condition of plasticity, which in the space of the main stress is a combination of conical and cylindrical surfaces:

$$F = \begin{cases} I_s + \sigma_o tg\varphi - \tau_{npu} \sigma_o \leq p_o \\ I_s + p_o tg\varphi - \tau_s npu \sigma_o > p_o \end{cases}, \quad (4)$$

where I_s – intensity of tangential stresses $\left(I_s = \sqrt{\frac{1}{2} s^{ij} s_{ij}} \right)$; σ_o – hydrostatic pressure; τ_s – threshold shifting voltage; φ – angle of internal friction of soil; p_o – parameter of the soil environment.

Based on the proposed method of correction component of the stress tensor elementary volume of soil, which is located in the boundary condition deviator s^{ij} and layer σ_o^{ij} parts of the stress tensor σ^{ij} can be represented in proportion to the respective components s_e^{ij} i σ_{oe}^{ij} of the tensor σ_e^{ij} , calculated on the assumption of elastic deformation of soil on growth of deformation $\Delta\varepsilon_{ij}$:

$$s^{ij} = q_1^\tau s_e^{ij}, \quad \sigma_o^{ij} = q_2^\tau \sigma_{oe}^{ij}. \quad (5)$$

Coefficients q_1^τ i q_2^τ are defined with the expressions:

$$q_1^\tau = 1 - \frac{3G_1}{2G_2 tg^2 \varphi + 3G_1} \cdot \frac{I_{2e} + \sigma_{oe} tg\varphi - \tau_s}{I_{2e}}; \quad (6)$$

$$q_2^\tau = 1 - \frac{2G_2 tg\varphi}{2G_2 tg^2 \varphi + 3G_1} \cdot \frac{I_{2e} + \sigma_{oe} tg\varphi - \tau_s}{I_{2e}}.$$

Here the constants of the soil environment G_1 i G_2 are submitted on the basis of elastic modulus and Poisson's ratio:

$$G_1 = \frac{E}{1+\nu}, \quad G_2 = \frac{E}{1-2\nu}. \quad (7)$$

where E – modulus of elasticity, ν – Poisson's coefficient, I_{2e} – second invariant of the stress deviator calculated on condition of elastic environment.

These value let us simulate the stress-strain state of soil environment, to determine the pressure distribution in the ground under its own weight and various external influences considering heterogeneity environment, changes in topography and physical-mechanical characteristics during deformation.

An example of the practical use of the complex is SATER.SOIL the analysis of SSS changes of the slope with additional static load on one of the pilot sites in Kiev.

Table 1

№ GTE	S, kPa	φ , grades	E, MPa	ρ , t/m ³	ν
1 Eolian layer	—	—	—	1.50	—
2. Loess sandy loam	14	22	14	1.65	0.30
3. Loam	18	19	19	1.79	0.35
4. Fine sand	1	32	25	1.87	0.30

In the numerical implementation the slope based on the engineering research was presented as discrete finite-element model of the grid cell that has 4 pieces, each of which is a separate geotechnical element (GTE). The parameter of the environment that are used in the numerical calculations are presented in the Tab.1.

At the first stage the problem of determining SSS slope from the action of mass forces was being solved, and for this and future productions K_{st} was calculated according to the Lombardo V.M. method [2]. As it is shown in Fig. 2 the small concentration of plastic deformation is concentrated in the basis of the existing rubble retaining wall.

The slope itself is in a stable condition, there is almost no plastic deformation ($K_{st} = 1.47$). The horizontal movement of the upper part of the existing wall is 1...2 mm, it means it is within the margin of error. This defined SSS of mass action forces (own soil weight) was accepts as a base for further calculations.

The next step was to determine and calculate the change of the SSS slope during its interaction with the three-storey building on strip foundations (Fig. 3), which passed the uniformly distributed load to the soil foundations $q = 1.7\text{kh/cm}^2$.

Generally under the foundations familiar SSS changes of the base are observed, as well as the concentration of shear plastic deformation. But under the foundations that are closer to the curb slope there is a tendency to increase. The directions of the distribution of plastic deformation in the direction of the slope goes to the zone that has been established as a result of the inclusion of a retaining wall in the system "home-slope-retaining structure." This is evidenced by the horizontal movement of wall to 5...7 mm. Those deformations do not affect the normal operation of the building and do not lead to the deformation of the stability of the slope and in this case $K_{st} = 1.31$.

The project remodeling supposes the addition of the two floors (one of which attic), which increased the burden on the soil base to

$q = 2.3\text{kh/cm}^2$. At the same time the measures to ensure the stability of the slope were not stipulated, and the estimated value of K_{st} was 1.2. But taking into account the threshold value of K_{st} for given geotechnical conditions and dynamics of SSS of the loaded slope we had to examine and solve the problem of ensuring its sustainability.

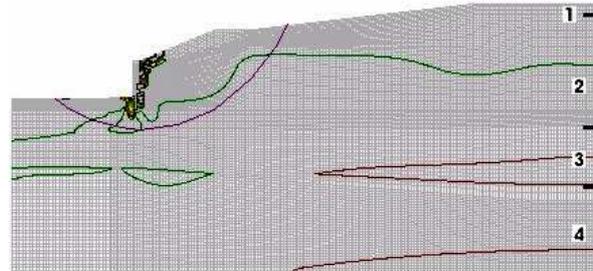


Fig. 2. Contours of the intensity of plastic deformation from the effects of the own soil weight

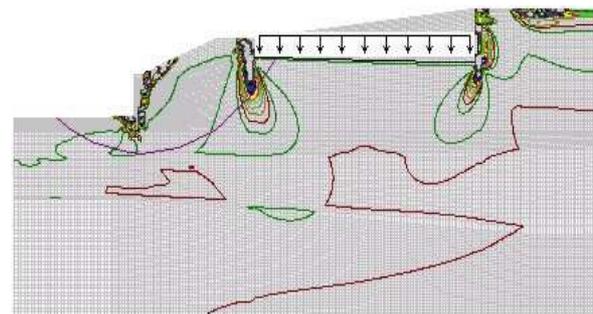


Fig. 3. Contours of the intensity plastic deformation

In the first stage of numerical calculation the changes of SSS of slope with increasing load on the soil base were clarified and their impact on slope stability without placing landslide protections. Calculations showed that the direction of concentration of the contours of the plastic deformation tends to be a combination of progressive deformation zone under the existing retaining wall and a subsequent possible release of the free surface (Fig. 4).

K_{st} value for the potential slip surface for a given trajectory of the SSS slope change was equal to 0.95. This horizontal movement of the existing retaining wall is 52 ... 63mm. All this indicates to the need for additional anti-

structures for stability of the slope. One of the design options of the provision of the normal operation of the building with additional designed floors is using bored piles as retaining walls. This wall should be set up at the curb of the slope, in a place that has the largest concentration of plastic strain contours (Fig. 5).

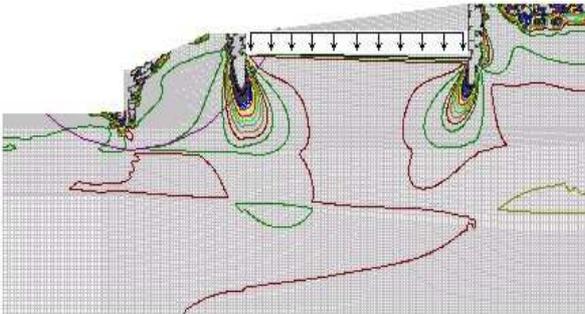


Fig. 4. Contours of the intensity of plastic deformation

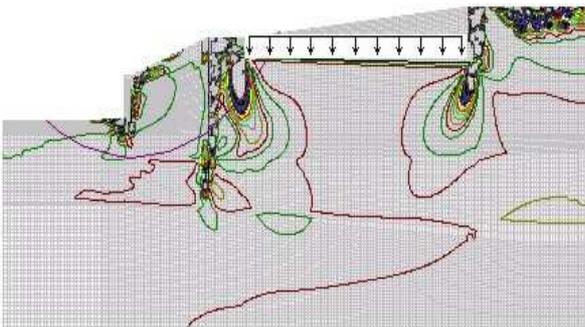


Fig. 5. Change of the intensity of the contours of plastic deformation and placing a second retaining wall

For efficient and effective use of anti-landslide measures retaining wall of bored piles $d = 0.8\text{m}$ lodged in sandy soils (EGE-4) was consistently constructed in these conditions.

When using such a retaining wall the SSS of the system "house-slope-retaining structure" has changed. The tendencies of the progressive plastic deformation zones to free surface were not observed. Maximum plastic deformations are concentrated near the projected retaining wall. Under these conditions its horizontal movement was 3 ... 5mm, and of the existing

lower walls, respectively, 1 ... 2 mm. For such a problem the value of K_{st} was 1.34, i.e. at the level of the slope to the superstructure. Therefore, further calculations and searching for effective design solutions landslide events are not needed.

CONCLUSIONS

Therefore, the proposed methods allow to:

1. Assess the SSS of the system "house-slope-retaining structures" as to the effects of both natural and anthropogenic factors specifying the needed parameters (e.g. K_{st}).
2. Trace the dynamics of the adjusted maximum plastic deformations, find a tendency to their accumulation or reduction at all stages of loading the soil environment.
3. Determine rational and efficient location of landslide protection works, they possible movement and influence of the design features on the state of the slope.

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АНАЛИЗ СОСТОЯНИЯ СКЛОНА ПРИ
УВЕЛИЧЕНИИ СТАТИЧЕСКОЙ НАГРУЗКИ

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Аннотация. На примере экспериментальной площадки в г. Киев рассмотрено влияние увеличения статической нагрузки при надстройке здания на напряженно-деформированное состояние склона. Показана эффективность предложенной методики при рассмотрении работы системы “здание-склон-подпорное сооружение” в зависимости от природных и техногенных факторов.

Ключевые слова: склон, грунт, оползнеопасная территория, метод конечных элементов, подпорное сооружение.