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Analysis of reinforced soil sustainability while tunnel construction

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Abstract. Object – reinforced ground array stability analisis. We have not found straight and general method of analysis in theory of limit equilibrium of soils (TLES). Methods – theory of stability (Culman method), method of limit parameters of TLES. A general scheme for solving the problem of the stability of vertical slopes reinforced with horizontal rods (rough and smooth) is developed, which takes into account the behavior of reinforcing elements not only in pulling and tension, but also in the vertical direction, whereby part of the weight of the wedge of failure is transferred over its boundaries to a fixed array.

1. Introduction

Horizontal reinforcement of the soil mass with rods of circular cross-section is most often used in two cases – when fastening the sides of excavation pits – the so-called dowel fastening – and when fastening the excavation face during tunneling.

Dowel fastening became widespread first in the domestic metro construction, and then moved from there to industrial civil construction.

The technology of tunnel excavation face shoring using horizontal rods (advance support) has received the greatest industrial development in Italy. Typical examples are: Alpine tunnels (Italy-France), tunneling of the Prapontin road tunnel (Italy); construction of a single-crown station "Ubaldi" (Italy) and others. In those tunnel structures an approach was used, which was called ADECO-RS – Analysis of Controlled Deformation in Rocks and Soils. Being successfully used in various types of soils, this approach allowed to find solutions in numerous difficult tunneling situations where the application of traditional methods could no longer justify itself.

So, in [1], it is recommended that the soil of the tunnel be pre-strengthened by introducing fiberglass reinforcing elements into the core using the following methods:

- reinforcing of the tunnel working space using fiberglass elements;
- reinforcing of the tunnel working space using fiberglass elements simultaneously with creating advance temporary shotcrete shells around the working space;
- reinforcing of the tunnel working space using fiberglass elements simultaneously with creating advance temporary shells around the working space of the tunnel in the form of horizontal jet grouting;
- a combination of reinforcing the tunnel working space using with fiberglass elements and jet grouting of the soil mass, as well as simultaneously creating advance temporary shells around the tunnel working space in the form of horizontal jet grouting with fiberglass elements.

Thus, horizontal reinforcement is effectively used to strengthen the tunnel working space in underground construction, to increase the stability of the tunnel working space and has a similar principle of working with the dowel fastening of vertical slopes of excavation pits.

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It should be noted that in water-saturated soils, the use of this type of reinforcement in underground construction is undesirable.

Turning to the existing methods for calculating reinforced soils, the article of V.G. Fedorovsky and S.G. Bezvolev [2] should be noted, where the one-dimensional problem of the theory of calculating vertically reinforced foundations was formulated. Then it received significant development in the works of domestic and foreign scientists. Unfortunately, it is not directly applicable to the horizontal reinforcement problem, and here a number of issues remain, including those of a fundamental nature.

Soil reinforcement of underground structures is closely related to the problem of loss of soil stability. Comparing the methods for calculating the slope stability (for example, [3]) and the various forms of soil stability loss ahead of the face during the construction of underground structures (for example, [4, 5-8]), it is clearly seen that, in fact, very similar processes take place, therefore, similar methods for solving problems can be applied.

The papers [9, 10] are devoted to issues related to the numerical analysis of the stability of transport tunnels faces reinforced with horizontal elements [11, 12], and the work [13] is devoted to the analysis of the tunnel reinforcement schemes with horizontal reinforcing elements in cohesive soils. Additionally, the work [14] should be emphasized, in which the loss of face stability by the kinematic method of the theory of ultimate equilibrium of soils is considered.

The behavior of rods in linearly deformable media is considered in sufficient detail in [15, 16]. Of interest there is also a study [17] which examines the effect of the stress-strain state of soil located behind a face on the stability of the face. The same authors analyzed the stability situation during stepwise excavation of the face [18].

It should be noted that to date, a fairly large number of studies have already been performed, in which an analysis of a variety of contact problems in a nonlinear setting is performed [19, 20]. These solutions find quite wide practical application, however, they contain some uncertainty regarding the correctness of the description of the limit state of soil. Namely, at this state, soil masses reinforced with horizontal rods work in pits and in the faces of tunnels. And here, while one of the most popular methods are the methods of the classical Coulomb's wedge theory.

In addition to the classical Coulomb's wedge theory methods, which have received recognition and numerous experimental confirmation, for example, [21] and others, to date, linear programming methods [22], which show stable convergence with well-tested results of Coulomb's wedge theory solutions and with experimental data, are increasingly gaining confidence.

Separate consideration deserves issues related to the preparation of reinforcing elements [23], especially during tunnel face shoring. It should also be noted that in tunnel construction there are a number of technologies competing with the advance support [24, 25].

Reliability assessment of a single system "reinforcing element-soil" can be performed by standard methods of reliability of building structures [26].

The issues of assessing the stability of rocky massifs are still one of the most difficult in geomechanics and, in addition, differ in a variety of design schemes [27, 28] and force effects [29, 30]. Often they are solved only in conjunction with an analysis of the monitoring results throughout the entire tunneling [31].

Thus, a theoretical analysis of the method of fastening of soil massifs with horizontal reinforced rods, given its comparative youth, is a relevant object. So, we have to solve next tasks:

- To develop a general scheme for solving the problem of the stability of vertical slopes reinforced with horizontal rods (rough and smooth), which takes into account the behavior of reinforcing elements not only in pulling and tension, but also in the vertical direction, whereby part of the weight of the wedge of failure is transferred over its boundaries to a fixed array.
- To develop a technique for determining the force of active pressure on the retaining structures for two cases – with the presence between the horizontal reinforcing elements and the shotcreting reinforcement and the absence of such bond.
- To develop a practical recommendations for the calculation of soil massifs reinforced with horizontal or slightly inclined reinforcing elements.

2. Methods

Let us consider a solution to the stability problem of a vertical slope reinforced by horizontal rod elements of a circular cross-section, on the basis of statics equations and the Coulomb law [32].

The following designations are adopted: γ, φ and c are specific weight, angle of internal friction and soil specific cohesion; H is the slope height; p is external pressure on the edge; l_i, h_i and $d = 2r$ are the length, depth and diameter of the i-th reinforcing element; a_h, a are horizontal and vertical spacing of reinforcing elements. The horizontal spacing should ensure the stability of the tunnel face between the vertical rows of reinforcing elements.

The general behavior scheme of the reinforced soil and the failure area formation pattern is determined, first of all, by the reinforcement density. With a very high reinforcement density, the sliding surface is formed outside the reinforced soil massif, which consequently acts as a quasi-solid gravity wall [33] (Fig. 1, a).

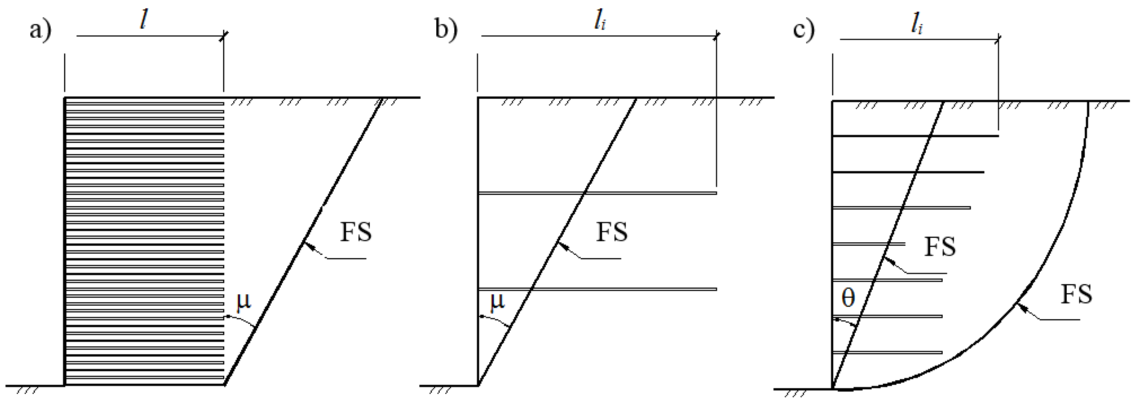

Figure 1. Failure schemes of the reinforced soil slopes with high (a), low (b), and "operating" (c) reinforcement density (*FS*-surface surface; $\mu = \pi/4 - \varphi/2$; $\theta < \mu$).

At a very low reinforcement density, the stability loss form of the soil massif will hardly differ from that of a homogeneous unreinforced slope, and the surface of failure intersects the reinforcing rods (Fig. 1, b). With a certain intermediate (conditionally speaking – "operating") reinforcement density, the failure can go along one of the two possible sliding surfaces – the first one crosses the reinforcing elements, and the second forms outside the reinforced soil mass (Fig. 1, c).

The intermediate scheme is of practical interest (see Fig. 1, c), since the first two are, in fact, limiting cases. Well-known methods of stability calculation can easily be performed along slip surfaces that form outside the reinforced soil massif. The main problem is considering the influence of reinforcing bars on the stability when collapsing surface crosses the reinforcing elements.

To solve this problem, we will consider the basic behavior scheme of a horizontal reinforcing element in a vertical tunnel face (Fig. 2). At the moment of the slope's stability loss, the reinforcing bar "cuts through" the wedge of failure. The force interaction of the reinforcing element and the wedge of failure is characterized by limit stresses – vertical p_u and shear τ_u, with:

$$\tau_u = \sigma_u \tan\psi, \qquad (1)$$

where σ_u is the component of the limit pressure p_u normal to the reinforcing element surface; $\tan\psi$ is the friction coefficient of the reinforcing element against the ground.

As a result, the force of the "cutting" will be transmitted to the fixed soil mass:

$$Q = p_u A_{pr}, \quad M = Q \cdot l_{pr} / 2, \quad N = \tau_u A_{pr},$$

where $A_{pr} = l_{pr}\pi r$ is the contact area of the ground and reinforcing element within the wedge of failure.

Consequently, the work of the horizontal reinforcing bar will consist of transferring a part of the load from the weight G of the wedge of failure to a fixed soil mass [32].

Fig. 3 shows the design model of a reinforced slope. As mentioned above, each reinforcing element will transfer the forces Q_i, M_i, N_i to the fixed part of the foundation.

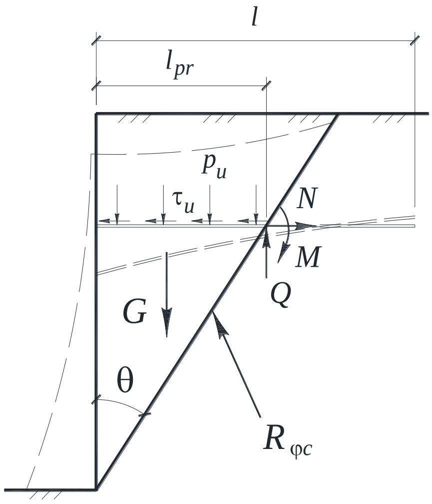

Figure 2. Principal behavior model of the reinforcing element (hatching shows a deformed view).

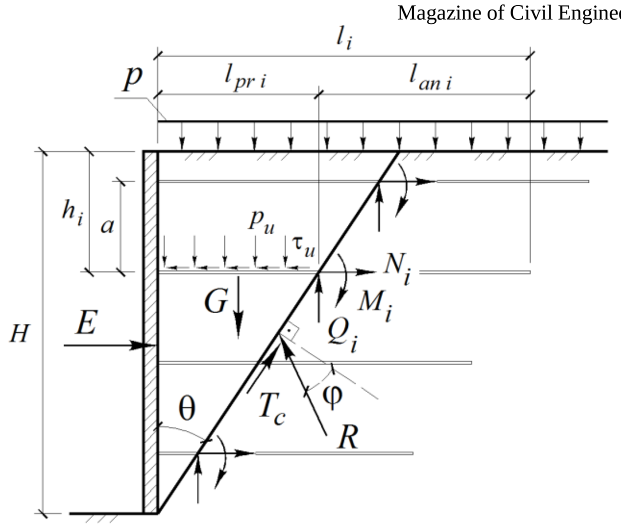

Figure 3. The design model of a vertical slope reinforced with horizontal rods.

Projecting the external forces and reactions acting on the prism at the wedge of failure at the moment of its collapse on the vertical and horizontal axes, we obtain the following equations [32]:

$$E+T_c\sin\theta-R\cos(\varphi+\theta)+\sum N_i=0;$$
$$P+G-T_c\cos\theta-R\sin(\varphi+\theta)-\sum Q_i=0, \quad (2)$$

where $G=0.5\gamma\cdot a_h\cdot H^2\cdot\tan\theta$, $P=p\cdot a_h\cdot H\cdot\tan\theta$, $T_c=c\cdot a_h\cdot H/\cos\theta$, $N_i=\tau_u l_{pr,i}\pi r$, $Q_i=p_u l_{pr,i}\pi r$.

The equation of moments establishes a correspondence between the coordinates of the points of application of the forces E and R, and is not considered here.

Solving the system (2) with respect to E, we obtain

$$E=(P+G-\sum Q_i)\cot(\varphi+\theta)-\sum N_i-T_c\cos\varphi/\sin(\varphi+\theta)\,. \quad (3)$$

The strength of the active pressure (along the length of the slope of a_h) is equal to:

$$E_a=\max E(\theta)\,. \quad (4)$$

For absolutely smooth rods, the position of the dangerous slip line is determined by the angle $\theta=\pi/4-\varphi/2$; for rough ones, a numerical search of θ is performed.

If $Ea\le 0$, then the ground face is retained only by reinforcing rods, and retaining structures are only needed to ensure local soil stability (against "caving" between rods).

If $Ea>0$, then in order to retain the ground face, in addition to reinforcement, a shotcreting reinforcement is required, but at the same time reinforcing elements reduce the value of E as compared to the unreinforced ground face. Here it seems possible to consider two design cases – when the shotcrete is connected with reinforcing elements, and when they work independently.

In the first case, collapse will not occur until the load-carrying capacity of the reinforcing elements for pulling out of the fixed ground is exceeded (it is assumed that the strength of the reinforcing elements bonds and shotcrete is sufficient):

$$E_a+\Sigma N_i\le\Sigma N_{an,i}=\Sigma\tau_u A_{an,i}\,. \quad (5)$$

where $A_{an,i}=l_{an,i}\pi r$ is the area of contact, which is equal to the half-surface of the reinforcing element, since it, being in a fixed array, undergoes significant bending deformations under the forces Q_i and M_i (see Fig. 2); τ_u is the same as in formula (1).

When the condition (5) is satisfied, the retaining wall works virtually only for local stability. If condition (5) is not satisfied, then it is necessary to perform the calculation of the shotcreting reinforcement for the impact of the force $E_a-\Sigma N_{an,i}$.

In the second case (the shotcreting reinforcement and the reinforcing element are not connected), the retaining shotcreting reinforcement should be calculated for the active pressure with the resultant E_a.

The next important question is that of the value of $l_{an,i}$ of embedding of the reinforcing bar into a fixed soil massif. In principle, the value $l_{an,i}$ must provide two conditions: the rod behavior in pulling by the force N_i and the rod stability against "reversing" by the forces Q_i and M_i.

The check for pulling is carried out according to the formula

$$N_i \le N_{an,i} = \tau_u A_{an,i}. \quad (6)$$

The check against "reversing" is expressed in ensuring the equilibrium of the part $l_{an,i}$ of the length of the reinforcing element in the embedding by the forces Q_i and M_i. This can be done in accordance with one of three schemes.

In the first scheme, it is assumed that the reinforcing element deforms in a fixed array according to the Fuss-Winkler theory, and the contact stresses reach the limit values $p_{u,b}$ at only one point (Fig. 4, a). In the second scheme, it is assumed that the contact stresses reach the limit values in individual sections, with the bottom and top limit values obviously being different – they will be denoted by $p_{u,b}$, and $p_{u,t}$ (Fig. 4, b). In the third scheme, the following limiting case is considered: in the region where the reinforcing element is pressed into the soil, stresses are equal to $p_{u,b}$, and in the region where the reinforcing element is bending upwards, "cutting" the overlying soils stresses are equal to $p_{u,t}$ (Fig. 4, c). The third scheme gives the minimum allowable amount of embedding.

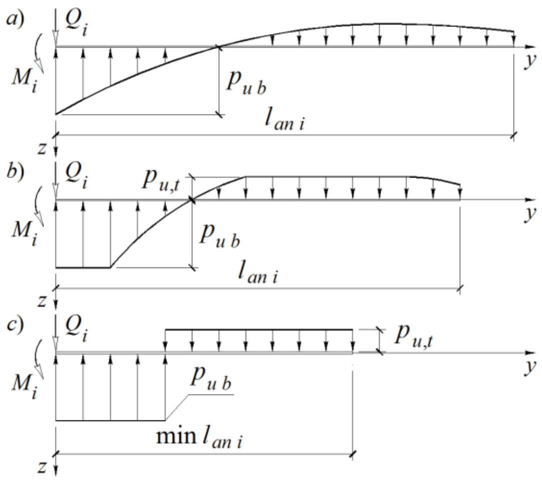

Figure 4. Schemes for determining the value of embedding the reinforcing element: a – completely safe embedding, b – with limiting pressure regions, c – minimal embedding.

In all three cases, the embedding value $l_{an\,i}$ is determined from the static equilibrium of the beam, on which the force Q_i, the moment M_i and the contact stresses, whose determination has been considered earlier.

Thus, for numerical realization of the method proposed here, it is necessary to know the values of the limiting stresses at the contact "reinforced element-ground" p_u, $p_{u\,b}$ and $p_{u\,t}$.

3. Results and Discussion

Let us consider solutions to the theory of limiting soil equilibrium of soils (TLES) to determine the values of the limiting stresses at the "reinforced element-ground" contact and their components – vertical p_u, $p_{u,b}$, $p_{u,t}$ and normal σ_u, $\sigma_{u,b}$, $\sigma_{u,t}$ to the contact surface.

Since the sequence of boundary-value problems plays the determining role in the solutions of the TLES, it is necessary to have some initial understanding of the geometry of the limiting equilibrium regions to come up with the solution. For this purpose, we make experimental and numerical studies to investigate the formation of zones of collapse in an array around the reinforcing elements when they “cut” the soil [32].

The numerical simulation is justified by the fact that for a wide range of practically important problems of the finite element method (FEM), which does not always allow reliable values of the ultimate load to be obtained, can still give a fairly correct picture of the deformed state. Therefore, in this case, FEM can be used in combination with the experiment results for a ground failure qualitative assessment.

In both cases (both the "cutting" of the wedge of failure and the "pushing" of the fixed soils by the reinforcing element), the formation of the limiting equilibrium regions proceeds from the reinforcing element upwards, which served as the basis for further theoretical analysis by the TLES methods, determining the sequence of boundary-value problems of the TLES (Fig. 5).

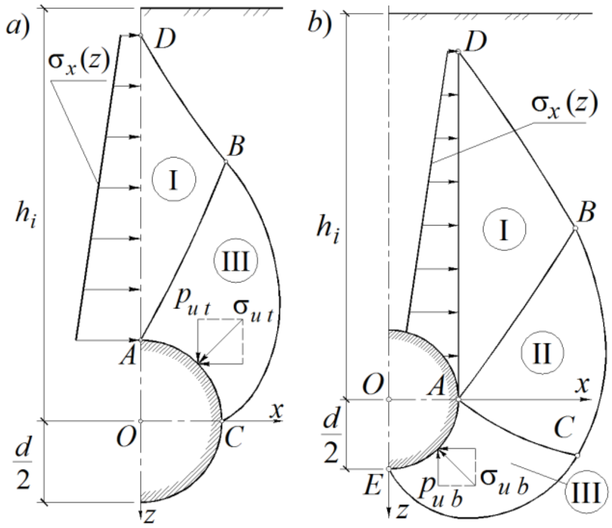

Figure 5. Solution sequence of boundary value problems: a – for p_u (or $p_{u\,t}$), b – for $p_{u\,b}$.

The solution was achieved by integrating the canonical system of equations of the static state of the granular medium, compiled from the characteristics (failure lines) of two families:

$$dx = dz\,tg(\alpha \pm \mu)\,, \qquad d\sigma \pm 2\sigma \tan\varphi\, d\alpha = \gamma(dz \mp dx \tan\varphi)\,,$$

where $\tan\alpha = 2\tau_{xz}/(\sigma_z - \sigma_x)$ is the angle between the direction of σ_1 and the Oz axis; $\sigma = (\sigma_x + \sigma_z)/2 + c \cot\varphi$ is the reduced average stress.

A feature of these schemes is the construction of a field of limiting stresses in the boundary value problem's zone III.

Fig. 6 shows the grids of the failure lines obtained for both schemes.

In both cases, the solution was detailed for two types of boundary conditions: first with taking into account the depth of the reinforcing element, if $\sigma_x(z) = \xi\gamma(h_i + z)$ at the boundary AD (where ξ is the lateral pressure coefficient); and the second without accounting for depth, if $\sigma_x(z) = 0$ at the boundary AD, which guarantees a certain margin of safety.

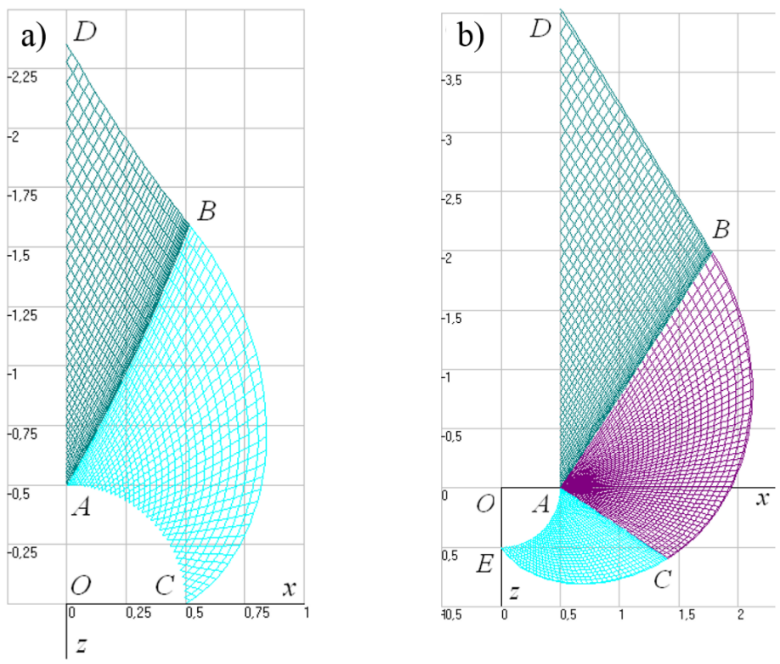

Figure 6. Examples of failure line grids: a – for p_u (or $p_{u,t}$), b – for $p_{u,b}$.

An analysis of the results showed that the obtained limiting stresses can be approximated by expressions in the form of the classical three-term K. Terzaghi formula. For the problem of "cutting" of the above-located soil by a reinforcing element, the expressions for the vertical and normal (to the surface of the contact) components of the limiting stresses are:

$$p_u = p_{ut} = N_\gamma \gamma d + N_q \gamma h_i + N_c c, \quad (7)$$

$$N_\gamma = [(-0.193\xi^2 - 0.127\xi + 0.32) + (0.159\xi^2 - 1.521\xi + 0.488)\, e^{(-6.63\xi^2 + 7.978\xi + 3.393)\tan\varphi}]/\pi$$

$$N_q = (2\xi e^{3.207\tan\varphi})/\pi, \quad N_c = (2.13 + 4.222 e^{2.372\tan\varphi})/\pi.$$

$$\sigma_u = \sigma_{ut} = N_\gamma \gamma d + N_q \gamma h_i + N_c c, \quad (8)$$

$$N_\gamma = [(0.0884\xi^2 + 0.634\xi + 0.526) +$$

$$+(1.122\xi^2 - 3.935\xi + 1.219)\, e^{(-6.673\xi^2 + 8.327\cdot\xi + 3.806)\tan\varphi}]/\pi,$$

$$N_q = (3.14\xi e^{3.207\tan\varphi})/\pi, \quad N_c = (4.414 + 6.97 e^{2.858\tan\varphi})/\pi.$$

Herewith the depth of the reinforcing bar must be greater than the maximum height of the development of the marginal equilibrium regions:

$$h_i > (0.206 + 0.609 e^{2.38813\tan\varphi})\, d.$$

Similar relations were obtained for the case of pushing the reinforcing element into the underlying soil. The required components of the ultimate stresses can be calculated by the formulas:

$$p_{ub} = N_\gamma \gamma d + N_q \gamma h_i + N_c c,$$

$$N_\gamma = [(-56.28\cdot\xi^2 + 58.504\cdot\xi - 2.202) +$$

$$+(2.581\cdot\xi^2 - 4.393\cdot\xi + 1.341) e^{(-11.414\cdot\xi^2 + 14.96\cdot\xi + 7.835)\tan\varphi}/\pi$$

$$N_q = [(0.193\cdot\xi^2 - 0.805\cdot\xi) + (2.14\cdot\xi\cdot e^{7.085\tan\varphi})]/\pi,$$

$$N_c = (8.279 + 5.985 e^{4.826\tan\varphi})/\pi.$$

$$\sigma_{ub} = N_\gamma \gamma d + N_q \gamma h + N_c c,$$

$$N_\gamma = [(-62.036\cdot\xi^2 + 64.106\cdot\xi - 1.969) +$$

$$+(3.011\cdot\xi^2 - 5.129\cdot\xi + 1.565)\, e^{(-11.356\cdot\xi^2 + 14.875\cdot\xi + 7.761)\tan\varphi}]/\pi,$$

$$N_q = [(0.222\cdot\xi^2 - 0.067\cdot\xi) + (3.04\xi e^{6.891\tan\varphi})]/\pi,$$

$$N_c = (12.381 + 8.318 e^{4.648\tan\varphi})/\pi.$$

Herewith the depth of the reinforcing bar must be greater than the maximum height of the development of the marginal equilibrium regions:

$$h_i > (-0.497 + 1.333 e^{2.945\tan\varphi})\, d.$$

At a lower value of h_i, the contact stresses should be considered equal to the natural pressure.

To confirm the obtained theoretical data, a full-scale field experiment in the Novosibirsk region was carried out in a specially organized testing area adjacent to the construction site of a tunnel-type overpass on the Baryshevo-Orlovka-Koltsovo highway. This experiment was carried out to correct preliminary design decisions, particularly the parameters of the soil reinforcement with horizontal rods ahead of the tunnel slope (the outstripping support) and the fastening of the near-slope part of the embankment [34].

The pressure was calculated according to the proposed procedure was 32.6 kPa, and the collapse of the slope occurred when the actual loading of the pit edge was equal to 37 kPa. Since the design pressure from the external load was 70 kPa, the results of the experimental-theoretical studies were corrected accordingly.

On this basis, practical recommendations were developed for pit edge and soils calculations ahead of the tunnel face reinforced with horizontal elements. The main calculation stages in the developed practical recommendations are:

1. Collecting loads on the edge of the slope and locating them along the width of the edge (for designing the outstripping tunnel support – according to the hypothesis of arching or the scheme of natural stresses in the calculated level).

2. Preliminary assignment of geometric reinforcement parameters (diameter of reinforcing elements, vertical and horizontal spacing of reinforcing elements).

3. Determination of the limiting pressures and their components at "reinforcing element-ground" contact (see the formulas (7) ... (8)). At the same time, within the wedge of failure, the calculation of the limiting pressure for $p_{u,t}$ should be performed at $\xi = 0$.

4. Determination of the most disadvantageous position of the failure surface and the value of the resultant active pressure (see the formula (4)).

5. Correction of reinforcement parameters depending on the adopted slope reinforcement scheme (with or without a retaining wall).

6. Determination of the embedding length for reinforcing elements in the fixed ground (see Fig. 4, c).

7. Checking the reinforced soil massif as a quasi-solid retaining wall for shear along the underlying unreinforced soil.

8. Checking the reinforced soil massif as a quasi-solid retaining wall for overturning.

9. Testing the overall stability (deep-seated shift) of the reinforced soil massif as a quasi-solid retaining wall.

10. Checking material strength of the reinforcing element, as well as the wall and its connection with reinforcing elements (if available).

By varying the parameters d_a, a_h, *a* based on the above calculation scheme, an optimization algorithm is constructed for the entire retaining structure for one or another selected optimization parameter.

The proposed technique can also be used if the reinforced elements have a small angle of inclination to the horizon – up to 15 °.

In [14] the method face stability analysis of tunnel face is described, but tunnel face is not reinforced by beams. A finite difference procedure for deep tunnels is presented in [9]. The aims in geomechanics solved by finite element or finite difference methods have to be verified by TLES formulas.

4. Conclusions

1. A general scheme for solving the problem of the stability of vertical slopes reinforced with horizontal rods (rough and smooth) is developed, which takes into account the behavior of reinforcing elements not only in pulling and tension, but also in the vertical direction, whereby part of the weight of the wedge of failure is transferred over its boundaries to a fixed array.

2. A technique has been developed for determining the force of active pressure on the retaining structures for two cases – with the presence between the horizontal reinforcing elements and the shotcreting reinforcement and the absence of such bond. The reinforcement sufficiency condition of a soil massif is formulated without additional retaining structures.

3. In order to quantitatively evaluate the interaction between the soils of the wedge of failure and the fixed region of the foundation, on the one hand, and the reinforcing elements on the other, new rigorous solutions of the TLES are applied to the limiting state of the soil surrounding the reinforcing element of circular cross-section (with and without depth of its location). The solutions obtained are reduced to the standard form of the three-term Terzaghi formula.

4. Practical recommendations for the calculation of soil massifs reinforced with horizontal or slightly inclined reinforcing elements are developed for two main practical schemes: the stability of the foundation pit edges and the stability of the ground ahead of the face during tunnel excavation.

References

1. Lunardi, P. Progetto e costruzione di gallerie – Analisi delle deformazioni controllate nelle Rocce e nei suoli. Milano, 2006. 572 p.
2. Fedorovsky, V.G, Bezvolev, S.G. Metod analiza svaynykh poley i drugikh vertikal'no armirovannykh mass grunta [Method of pile fields and other vertically reinforced soil masses analysis]. Soil Mechanics and Foundation Engeneering. 1994. No. 3. Pp. 17–27. (rus) DOI:10.1007/BF02335022
3. Senenta, S., Mollonb, G., Jimeneza, R., Tunnel face stability in heavily fractured rock masses that follow the Hoek–Brown failure criterion International Journal of Rock Mechanics and Mining Sciences. 2013. 60. Pp. 440–451. DOI:10.1016/j.ijrmms.2013.01.004
4. Elkayama1, I., Klarb, A. Upper bound of tunnel face stability using asymmetric yielding. Underground Space. 2018. 3(4). Pp. 288–296. DOI:10.1016/j.undsp.2018.03.001
5. Wilson, D.W., Abbo, A.J., Sloan, S.W., Lyamin, A.V., Undrained stability of a square tunnel where the shear strength increases linearly with depth. Computers and Geotechnics. 2013. 49. Pp.314–325. DOI:10.1016/j.compgeo.2012.09.005
6. Xiang, Y., Song, W., Upper-bound limit analysis of shield tunnel stability in undrained clays using complex variable solutions for different ground-loss scenarios. International Journal of Geomechanics. 2017. 17(9). Pp.113–119. DOI:10.1061/(ASCE)GM.1943-5622.0000946
7. Zhang, C., Han, K., Zhang, D., Face stability analysis of shallow circular tunnels in cohesive frictional soils. Tunnelling and Underground Space Technology. 2015. 50. Pp. 57–63. DOI:10.1016/j.tust.2015.08.007
8. Zhang, F., Gao, Y.F., Wu, Y.X., Zhang, N. Upper-bound solutions for face stability of circular tunnels in undrained clays. Géotechnique. 2018. 68(1). Pp. 76–85. DOI:10.1680/jgeot.16.T.028
9. Oreste, P. Face stabilization of deep tunnels using longitudinal fibreglass dowels. International Journal of Rock Mechanics and Mining Sciences. 2013. 58. Pp. 127–140. DOI:10.1016/j.ijrmms.2012.07.011
10. Li, B., Hong, Y., Gao, B., Qi, T.Y., Wang, Z.Z., Zhou, J.M. Numerical parametric study on stability and deformation of tunnel face reinforced with face bolts. Tunnelling and Underground Space Technology. 2015. 47. Pp. 73–80. DOI: 10.1016/j.tust.2014.11.008
11. Paternesi, A., Schweiger, H.F., Scarpelli, G. Numerical analyses of stability and deformation behavior of reinforced and unreinforced tunnel faces. Computers and Geotechnics. 2017. 88. Pp. 256–266. DOI: 10.1016/j.compgeo.2017.04.002.
12. Maghous, S., Bernaud, D., Couto, E. Three-dimensional numerical simulation of rock deformation in bolt-supported tunnels. Tunnelling and Underground Space Technology. 2012. 31. Pp. 68–79. DOI:10.1016/j.tust.2012.04.008
13. Anagnostou, G., Perazzelli, P. Analysis method and design charts for bolt reinforcement of the tunnel face in cohesive-frictional soils. Tunnelling and Underground Space Technology. 2015. 47. 2015. Pp. 162–181. DOI:10.1016/j.tust.2014.10.007
14. Pan, Q., Dias, D. Face stability analysis for a shield-driven tunnel in anisotropic and nonhomogeneous soils by the kinematical approach International Journal of Geomechanics ASCE. 2016. 16(3). DOI:10.1061/(ASCE)GM.1943-5622.0000569
15. Klar, A., Osman, A.S. Load-displacement solutions for piles and shallow foundations based on deformation fields and energy conservation. Géotechnique. 2008. 58(7). Pp. 581–589. DOI:10.1680/geot.2008.58.7.581
16. Klar, A., Randolph, M.F. Upper-bound and load displacement solutions for laterally loaded piles in clays based on energy minimization. Géotechnique. 2008. 58(10). Pp. 815–820. DOI:10.1680/geot.2007.00197
17. Pankratenko, A., Isaev, A. The Analysis of the stress-strain state of the system "equipment complex - support - rock mass" in the bottomhole area of the shaft. E3S Web of Conferences. 2018. 41. DOI: 10.1051/e3sconf/20184101038.
18. Pankratenko, A. Vliyaniye poshagovogo vskrytiya zaboya pri prokhode krupnykh uchastkov na geomekhanicheskoye sostoyaniye porody akkomodatsionnogo massiva [The effect of step-by-step face opening during road-heading of large sections on geomechanical state of rocks of accommodating massif]. Gornyi Zhurnal. 2002. Pp. 93–99. (rus)
19. Zertsalov, M.G., Znamenskiy, V.V., Khokhlov, I.N. Interaction of drilled shafts in rock under the action of lateral load IOP Conference Series: Materials Science and Engineering. 2018. 365. 042050. DOI:10.1088/1757-899X/365/4/042050
20. Zertsalov, M.G., Merkin, V.E., and Khokhlov, I.N. The influence of the interface of drilled socketed shafts in rock. Geomechanics and Geodynamics of Rock Masses. 2018. Pp. 937–943.
21. Karaulov, A.M., Experimental and theoretical research on the bearing capacity of ring-foundation beds. Soil Mechanics and Foundation Engineering. 2005. 42(3). Pp. 75–80. DOI: 10.1007/s11204-005-0028-5
22. Karaulov, A.M. Experimental and theoretical research on the bearing capacity of ring-foundation beds. Soil Mechanics and Foundation Engineering. 2006. 43(2). Pp. 37–40. DOI: 10.1007/s11204-006-0021-7
23. Baydjanov, D.O., Abdrakhmanova, K.A., Kropachev, P.A., Rakhimova G.M. Modified concrete for producing pile foundations. Magazine of Civil Engineering. 2019. 86(2). Pp. 3–10. DOI: 10.18720/MCE.86.1
24. Sabri, M.M., Shashkin, K.G. Improvement of the soil deformation modulus using an expandable polyurethane resin. Magazine of Civil Engineering. 2018. 83(7). Pp. 222–234. DOI: 10.18720/MCE.83.20
25. Sabri, M.M., Shashkin, K.G., Zakharin, E., Ulybin, A.V. Soil stabilization and foundation restoration using an expandable polyurethane resin. Magazine of Civil Engineering. 2018. 82(6). Pp. 68–80. DOI: 10.18720/MCE.82.7
26. Pichugin S.F. Reliability estimation of industrial building structures. Magazine of Civil Engineering. 2018. 83(7). Pp. 24–37. DOI:10.18720/MCE.83.3
27. Sammal, A.S., Antsiferov, S.V., Deev, P.V., Sergeev, S.V. Estimate of rock mass stability in surface-borehole mining of high-grade iron ore. IOP Conference Series: Earth and Environmental Science. 2017. 53(1). DOI:10.1088/1755-1315/53/1/012007
28. Sammal, A.S., Antsiferov, S.V. Influence of mutual position of a noncircular tunnel and a dynamic impact source on stress distribution in lining. IOP Conference Series: Earth and Environmental Science. 2018. 134(1). DOI:10.1088/1755-1315/134/1/012054.
29. Antsiferov, S.V., Sammal, A.S., Deev, P.V. Stress state estimation in multilayer support of vertical shafts, considering off-design cross-sectional deformation. IOP Conference Series: Earth and Environmental Science. 2018. 134(1). DOI:10.1088/1755-1315/134/1/012001.
30. Korovkin, V.S., Ragulin, K.G., Kuzina, A.D., Smolenkova, A.V., Orlova N.S., Computation of mooring quay in the form of pile grillage. Magazine of Civil Engineering. 2017. 71(3). Pp. 10–20. DOI: 10.18720/MCE.71.2
31. . Deev P., Sammal A., Antziferov S. Evaluation of mine support stress state on base of convergence measurement data // International Multidisciplinary Scientific GeoConference Surveying Geology and Mining Ecology Management, SGEM. 2017. 17(13). Pp. 321–326. DOI:10.5593/sgem2017/13/S03.041

32. Kuznetsov, A.O. Prakticheskaya metodologiya rascheta sklonov, armirovannykh gorizontal'nykh sterzhney kruglogo secheniya [Practical methodology for calculating slopes reinforced with horizontal rods of circular cross-section]. Izvestia VNIIG B.E. Vedeneeva. 2017. Pp. 88–96.
33. Ovchinnikov, I.I., Ovchinnikov, I.G. Application of nonlinear deformation model for the analysis of behavior of reinforced plates on elastic foundation, interacting with chloride-containing environments. The basic relations. Magazine of Civil Engineering. 2013. 36(1). Pp. 27–38. DOI: 10.5862/MCE.36.4
34. Oparin, V.N., Yushkin, V.F., Polyankin, G.N., Grishin, A.N., Kuznetsov, A.O., Rublev, D.E. Geomechanical monitoring of temporal lining in railway tunneling in complex geological conditions. Journal of Mining Science. 2015. 51(4). Pp. 839–859. DOI: 10.1134/S1062739115040226

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