Reliability of Assessment of Advance Anchoring

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Abstract. On the example of the construction of the dismantling chamber No. 1 of the Kalinin-Solntsevskaya line of Moscow Metro, and the studies of the stress-strain states of the contour massif of rocks in the bottom zone by the finite element method, evaluating measures were taken to ensure the stability of the enclosing rocks and the camera’s protective screen. Simulation of the rock mass stress state has proved the possibility of soil hardening in the set of workings by advanced anchoring to reduce rock displacements and surface deformation. A model to determine the maximum depth of using a protective screen with long anchors has been compiled.

1. Introduction

The growing volumes of social activity during the continuous development of the central districts of cities significantly complicate the functioning of transport and engineering infrastructures [1–2], which requires the development of underground space. Moreover, the creation of a single network of integrated public spaces with a mutually linked network of underground pedestrian links-galleries [3–6] is most appropriate.

The construction of urban underground structures to reduce costs, in the vast majority of cases, is carried out at shallow depths, that is, in alluvial rocks, characterized by very difficult conditions. The largest volumes of penetration are associated with the construction of long workings where high-performance shield complexes are used. However, the construction of new lines and their connection with existing ones will require the construction of a large number of branched transitions for optimal functioning of transport facilities.

Such workings are characterized by a complex configuration and trajectory, which eliminates the possibility of using effective shield systems. In these cases, technologies are used that are characterized by high labor costs, financial and material resources, as well as difficult and dangerous conditions for the work performance.

Violations of the soil continuity, in its turn, provokes an increase in the speed of moving water in the massif and the formation of groundwater reservoirs [1], which is accompanied by deformations of the soil massif and the surface of the earth.

Over the past 40 years, about 50 major soil failures have been recorded in Moscow [7]. According to experts, only in Moscow the total area of potentially dangerous sites for construction reaches 15 km² [8]. This means that the task of ensuring safety during the construction of underground small length structures in soil massifs remains relevant, since in such conditions the penetration is carried out under protection of a labor-consuming and expensive temporary lining.
2. Methods and materials
In order to improve the complex, labor-consuming and unsafe technology for the development, a study was conducted on the possibility of using advanced anchoring to strengthen the soil in the set of workings [9]. Hardening is carried out by the formation of a protective shield from steel pipes with a diameter of 100-120 mm, being monolithic with cement mortar in wells up to 15 m long [10] to prevent soil displacements ahead of the face. To assess the effectiveness of the protective shield, a study was made of the stress-strain state (SSS) of the near-edge rock mass in the bottom zone of the face by means of using the example of the construction of dismantling chamber No. 1 of Kalinin-Solntsevskaya line of Moscow Metro.

In recent years, due to the improvement of measurement methods and tools, modeling is widely used in assessing the state of an array. One of the most effective modern methods for numerically solving engineering problems using computers is the finite element method [11-14]. The most widespread practice in the calculation and design of building structures of underground structures was the FEM option in movements.

In the practice of underground construction, the main criterion for assessing the stability of rock outcrops is the displacement of the loose rock contour. Therefore, the FEM in this case will make it possible to objectively evaluate the behavior of the face with a complex SSS of the soil mass [7, 15-16].

The main factors determining the displacement of the notch contour are: the magnitude of the rock pressure; physical and mechanical properties of rocks: the size of the output and the magnitude of the entry [17-18]. Since rock pressure depends on the width of the mine, we leave the most generalizing factor - rock pressure, while the width can be excluded from consideration. The remaining factors make it possible to simulate the hardening process by leading anchoring in statement of the problem. To simulate the “soil mass - tunnel - protective screen” system using the finite element method, the Plaxis software package was used.

In the array model, rock characteristics were defined by assigning stiffness characteristics to finite stiff elements. Rock pressure is automatically generated by the program by applying a load to all the finite elements of their own weight when the depth of the mine is changed.

The quantitative assessment of the SSS of an array depends on the accepted model of the mechanical properties of the soils composing the array (elastic, elastic-plastic, elastic-viscous-plastic at present, in geomechanics, the linearly deformable medium model based on the theory of elasticity, together with the fracture model based on the Coulomb-Mohr theory of strength [18], is most widely used to estimate the SSS of a soil massif.

During the construction of underground structures, before the lining is erected, the soil mass on the excavation circuit is unloaded, and then, after the lining is erected, it is reloaded due to continuous creep deformations of the massif. Since there is a cycle of unloading and reloading, it is necessary to use an elastic-plastic model of the massif [5-6]. For an elastic-plastic model, the following soil characteristics must be introduced: deformation modulus – E, Poisson's ratio – ν, internal friction angle – φ, friction – c [5-6, 18].

3. Discussion of simulation results
The deformation schemes obtained as a result of modeling with a protective screen up to 15 m long, even with a development depth of 5 m (Fig. 1), indicate not only displacements of the face that can reach 21.45 mm, but also surface deformations. It is not possible to calculate the model with a depth of 15 m due to the destruction of the model.

On models with a depth of 10 m, it was found out that after passing 1 m of development, before the formation of the next run, the maximum displacements are observed in the bottom hole zone and they amount to 110 mm. This is due to the fact that the anchors of the protective screen in front of the face are located in the prism zone of soil slipping and work according to the most unfavorable scheme - the cantilever. In addition, the steel pipe in the anchor will always be located in the lower zone of the "pressed-in body", and with the cantilever operation scheme of the anchor its effectiveness is...
extremely low, since the cement stone in the upper zone of the "crimping" will not resist tensile stress. Therefore, the anchors of the protective screen, having an insufficient moment of resistance, do not impede the transfer of pressure from the overlying rocks to the ground in the zone of the sliding prism.

![Figure 1](image1.png)

**Figure 1.** Scheme of displacements of bottom hole rocks and protective shield anchors at a depth of 5 m.

It was also pointed out that increasing the length of the anchors to 18 - 24 m allows you to change the working pattern of the anchors and increase the bearing capacity of the protective screen. In this case, the bottom hole part of the anchors will rest on the ground outside the creeping prism (Fig. 2).

![Figure 2](image2.png)

**Figure 2.** Scheme of displacements of bottom soil with elongated protective shield anchors at a depth of 10 m.
Based on the simulation results, stress and displacement concentration zones are established, which are located in the lower part of the face. In this case, to prevent displacements, it is enough to install five polymer-cement anchors with removable supporting elements.

Since the depth of the same excavation can vary depending on the surface topography, to calculate the reliability of technical solutions under the same soil conditions and the dimensions of the excavation, the displacements of the face were calculated for various excavation depths (10, 15, 22, 25, 30, and 40 m).

The calculation results are shown in table 1.

| Estimated Depth, m | 10   | 15   | 22   | 25   | 30   | 40   |
|-------------------|------|------|------|------|------|------|
| Soil displacements, mm | 32,9 | 49,8 | 70,6 | 78,4 | 92,3 | 140,6 |

When approximating the obtained results, it was found out that with sufficient reliability of the approximation, the sample is described by a linear dependence, with a value of the reliability of the approximation of 0.983.

$$Y = 3,4642X - 4,5482$$

where $Y$ is the magnitude of the displacements, $X$ is the depth of development.

Table 2 gives an estimate of the accuracy of the description by equation 3.2 of the simulation results.

| Estimated Depth, m | 10   | 15   | 22   | 25   | 30   | 40   |
|-------------------|------|------|------|------|------|------|
| Soil displacements, mm | 32,9 | 49,8 | 70,6 | 78,4 | 92,3 | 140,6 |
| Design soil displacements, mm | 30,09 | 47,41 | 71,66 | 82,06 | 99,38 | 134,02 |
| Error, %          | 8,59 | 4,85 | 1,56 | 4,62 | 7,69 | 4,68 |

The sampling confidence interval of excavation is $\alpha = 0.063$, and the standard deviation is $\sigma = 0.067$.

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4. Conclusion

In the result of FEM simulation is set ineffective the work of the anchors of the protective screen when they are located in the area of the prism sliding.

To increase the stability of ground outcrops in the face can increase the length of the protective shield anchors to support their ends outside the sliding prism.

The resulting expression (1), taking into account the reliability coefficient of the object under construction, allows us to determine the maximum depth of application that is ahead of anchoring with a screen of steel pipes with a diameter of 106 mm.

As a result of multivariate modeling, taking into account the change in soil properties, a very complex dependence was established with significant errors in the calculated displacements at the boundaries of the ranges of the studied factors. Therefore, given the simplicity of changing the parameters of the soil mass in finished models and the minimum time spent on calculations, we consider the use of a multi-factor model inappropriate.

References

[1] Telichenko V I, Zertsalov M G, Konyukhov D S 2010 State and prospects of development of underground space of Moscow Vestnik MGSU 4(4) pp 24-36
[2] Rudyak M S Rational use of urban underground space for civil objects 2003 (Moscow: publishing house of Moscow state mining University) p 235

[3] Levchenko A N Geotechnological strategy and high technologies of development of underground space of the city of Moscow 2006 Mountain information and analytical Bulletin (scientific and technical journal) pp 14-18

[4] Belyaev V L 2013 Planning of urban development of underground space in Moscow Vestnik MGSU 1 pp 35-46

[5] Degtaryev B M 1998 State, prospects and problems of underground space use in Moscow Proceedings of the international conference "Underground city: Geotechnology and architecture" (Saint-Petersburg) pp 117-119

[6] Lerner V G, Petenko E V, Petenko I E 2000 Development of underground space of big cities Underground construction of Russia at the turn of the XXI centur. Proceedings of the anniversary scientific and practical conference (Moscow) pp 103-110

[7] Svetozarova D V Moscow partially collapsed subway tunnel https://www.vladtime.ru/proish/575641

[8] Malarenko E, Kostina E 2018 In Moscow du ring the construction of the subway there was a failure of soil http://yandex.ru/clck/jsredir?bu=i6i65a&from=yandex.ru%3Bsearch%2F%3Bweb%3B%3B&text=&text=4946.WdF6NngyoTsVgCroF0P3CNUSB3AteukwKv36YPbh0xv_3_R5dOaICWYI_UvWE8XxI12

[9] Failures and subsidence of territory in Moscow http://www.bronepol.ru/y7/i/index.php?ELEMENT_ID=6603

[10] Nikoliski A 2010 Cases of soil failures in Moscow in 2006-2010 http://yandex.ru/clck/jsredir?bu=lep931&from=yandex.ru%3Bsearch%2F%3Bweb%3B%3B&text=&text=4946.N5qceu8ra-g_rZQypPlxuZmYBbwBTdscdtf5xOgD555RUgmTY1ef1ltyw3hPZ-mJJePKK0aLQZpzwIvmd2jMhRh

[11] Nikolski A 2010 Cases of soil failures in Moscow in 2006-2010 http://1prime.ru/MACROECONOMICS/20130206/760995703.html

[12] Bulatova L Moscow on the verge of failure http://homeweek.ru/publics/55

[13] Lerner V G, Petenko E V 1999 Systematization and improvement of underground construction technologies (Moscow: TIMR) p 67

[14] Glavatskikh, V A, Molchanov V S 2006 Construction of subways (Moscow) p 680

[15] Kuchumov R A, Keppler H, Prokop’ev V I 1994 Application of the finite element method to analysis of structures (Moscow: Publishing House of the Association of construction universities) p 353

[16] Trushin S I 2008 Finite element Method Theory and problems (Textbook. Moscow ASV publishing house) p 256

[17] Prokopov A Y 2013 The Study of stress-strain state of the soil massif and mutual influence of underground designs of existing and newly constructed buildings in the coastal zone of the sea port of Taman Engineering journal of don http://www.ivdon.ru/magazine/archive/n4y2013/2104

[18] Fadeev A B 2012 The model Parameters hardening soil program "Plaxis" Numerical methods of calculations in practical geotechnical engineering: collected papers of international scientific-technical conference (SPb: SPSUACE) pp 13-20

[19] Ter-Martirosyan Z G 2005 Soil Mechanics Textbook (Moscow: Publishing House of the Association of construction universities) p 488

[20] Tsytovich N A 2008 Soil mechanics: a short course The textbook (Moscow LKI Publishing house) p 272

[21] Tsytovich N A, Ter-Martirosyan Z G 1981 Fundamentals of applied geomechanics in construction (Moscow: High school) p 317
[22] Dalmatov B I, Bronin V N, Karlov V D 2000 Soil mechanics part 1 Basics of geotechnics in construction Textbook (Moscow: ASV; SPb. The University SU) p 204

[23] Stroková L A 2008 Technology of setting parameters for numerical modeling of soil behavior (News of Tomsk Polytechnic University) 313(1) pp 69-74