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Prediction of the stress-strain state and stability of tunnel face at the intersection of disturbed zones of the soil mass

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Abstract. The article presents a numerical solution of the spatial elastic-plastic problem of determining tunnel face stability at the intersection of disturbed zones of the soil. The relevance of the study is related to the need to take into account the zones of disturbed soils when assessing the face stability to calculate the parameters of the support. Based on the finite element method implemented in the PLAXIS 3D software package, the design of a finite element system "soil – disturbance – tunnel face support" and modeling of the intersection of the disturbed zones of the soil were performed. To assess the stability of soils, deformation and strength criteria are taken. The deformation criterion is expressed by the value of the calculated displacement of the tunnel face surface, and the strength criterion – by the safety factor until the maximum values of the stress state are reached according to the Coulomb – Mohr criterion. The results of the study are presented in the form of histograms of the safety factor dependences on the distance to the disturbance at different bending stiffness of the tunnel face support structure, as well as the isofields of deformation development. The parameters of rockfall formation in the tunnel face zone at the intersection of zones of disturbed soils were determined. The local decrease in strength and deformation properties in the rock mass along the tunnel track should be taken into account when assessing the stability of the tunnel face and calculating the parameters of the support. Within the framework of the constructed closed system, a qualitative agreement of the simulation results with the case of a collapse in the tunnel face during the construction of the Vladimirskaya-2 station of the Saint Petersburg Metro was obtained.

***Keywords*:** tunnel face stability; stability forecast; tunnel face support; static calculation; rockfall formation

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Introduction. The underground space of Saint Petersburg has a complex structure and is characterized by the presence of tectonic faults [1], areas of lifting and lowering of geological blocks with low geodynamic mobility [2]. This creates unfavorable conditions for the construction and operation of metro tunnels. Localized disturbing zones have been formed in the rock mass, the need to cross them may complicate the conditions for the construction of tunnels. The term "disturbing" in the article refers to a change in the strength and deformation properties of an engineering-geological element for man-made or other reasons. Studies have revealed localization of a decrease in strength parameters and deformation characteristics of soils in the ground up to two times [2-4]. Among the reasons leading to a decrease in strength properties are the processes of biochemical weathering, decompression and softening with increasing humidity, as well as the living activity of natural and foreign microbiota [5, 6]. The cases of the rockfall during the construction of the Saint Petersburg metro are given in [7].

The presence of disturbing zones in the ground determines new requirements for the design of the construction of tunnels and transfer stations. One of the risk factors in the construction of an underground structure is the possibility of soil collapse in the tunnel face [8-10]. The probability of this risk increases significantly when the tunnel face approaches the site of a local decrease in the strength of the ground. In this regard, when assessing the tunnel face stability, it becomes necessary

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to take into account the structural features. The relevance of the study is related to the complexity and insufficient investigation of the stress-strain state of a physically nonlinear ground in disturbed zones [11, 12].

Most of the work on determining stability is based on justifying the necessary pressure on the tunnel face, which is true for mechanized tunneling [13-16]. In the works [17, 18], the tunnel face stability of flat-lying formation is analyzed. An approach to assessing the tunnel face stability is described from the point of view of two limiting state, exceeding the criteria of which leads to the impossibility of construction the tunnel, loss of serviceability, determined by subsidence on the surface [19]. An analytical review of approaches to the assessment of subsidence on the surface is presented in [20]. The work [21] is devoted to the prediction of the open tunnel face stability. In the article [22], a study of the tunnel face stability during the advanced tunnel at a depth of 8 m from the bottom of quaternary sediments was carried out. Together with the open face situation, situations with different pressure on the tunnel face surface were investigated [23, 24]. A decrease in displacements was revealed as the resistance from the support side increased. Works [25, 26] are devoted to the analysis of the effect of the tunnel face support on the stress-strain state. The method of the tunnel face anchoring is described in the patent [27]. In the article [26], a method of modeling anchor support has been developed taking into account the anchoring in the lock. The results of field observations of the tunnel face surface deformations are presented. The influence of the bending stiffness of the supporting constructions on the deformation of the ground is demonstrated. The work [28] is devoted to the assessment of the stress state in the ground. It can be concluded that the methodological approach associated with the determination of stability through distributed pressure on the face does not take into account the formation of the stress-strain state of the tunnel face soils depending on the bending stiffness of the face suppoting structure. The paper [29] describes the stability of an open vertical outcrop in an undrained rock mass with undrained shear strength. The work [30] is devoted to the face stability in a drained rock mass with adhesion and internal friction. Works [31, 32] and research [33] on the stress-strain state of face soils in zones of heterogeneous soils are devoted to the assessment of the stress-strain state of workings near the disturbance. The method of calculating the parameters of the support in the face by the method of limiting balance was considered in [34]. The criterion of stability of soil outcrops adopted in domestic engineering practice is described in [35].

Problem statement. Rockfall formation during tunneling is associated with the activation of deformation processes in the face zone, which will reach the ground surface and lead to additional yield of the buildings foundation. Insufficient knowledge of the process of rockfall formation in clays under the influence of a combination of factors and the lack of a method for calculating the load on the tunnel face support in disturbed ground cause the need to predict the stress-strain state in the tunnel face.

The purpose of the study is to investigate the regularity of the rockfall formation and to study the state of face stability in the zone of disturbed soils. The presence of a predictive view of the dynamics of changes in the state of stability and loss of face soils balance at the intersection of the zone of disturbed soils will allow to qualitatively change the approach to the design of the temporary face support, reduce the probability of the risk of the unpredictable deformations development on the earth's surface, and to preserve the buildings undermined by an underground structure. The achievement of the purpose is possible with the calculation of the parameters of the support that ensure the stability of the face soils and their balance in the face. The tasks of the study are: assessment of the impact of the stiffness of the support on the tunnel face stability [36]; comparison of the stiffness of the support with the value of the equivalent resistance from the support; study of the factors causing the loss of stability and balance of the soils of the tunnel face; identification of the dynamics of changes in the stress-strain state and the stability criterion as the tunnel face approaches the disturbance zone, taking into account the properties of rocks in the disturbed zone [37].

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The paper proposes, on the basis of an elastic-plastic model with hardening and a strength criterion based on the ratio of maximum shear stresses to their limiting values, to predict the stable state and the zone of possible collapse.

Research methods. The problem under consideration in the mathematical formulation belongs to the class of elastic-plastic problems with an unknown boundary, which is determined in the process of its solution. It is known [35, 38] that these problems are difficult to obtain an analytical solution even in a 2D formulation. The paper considers the 3D formulation of the problem, the solution of which has not yet been developed by the analytical method. Therefore, a numerical solution method was used. The predictive model is based on the method of 3D finite element modeling of geomechanical processes during the construction of a tunnel in the zone of disturbance of the ground with implementation in the software package.

There is an active redistribution of stresses and deformations in the face zone. To assess the stress level, it is proposed to use the strength criterion, the deformation level – the deformation criterion [39]. As a deformation criterion, the stability criterion adopted in engineering practice is taken, expressed in terms of the calculated displacement of the contour of the underground structure. Depending on the value of the implemented contour displacements and the type of rocks, four categories of ground stability are identified.

The condition [40] is accepted as a strength criterion of stability in the outcrops of the tunnel face soils:

$$SF = \frac{\tau}{\tau_{sf}},$$

where τ – the value of the greatest shear stress associated with the Mohr – Coulomb strength criterion, MPa; τ_{sf} – the estimated minimum value of the shear stress required for balance in the design situation, MPa.

To estimate the *SF* parameter, numerical modeling was performed in a spatial formulation using a sequential decrease in the strength properties of the ground in the PLAXIS 3D software package. The calculation algorithm was reduced to calculations of a set of balanced states of the system with consistently introduced unbalanced forces associated with a decrease in strength. At the end of each calculation stage, the geomechanical system was in a equilibrium state, however, at the phase of numerical calculation of *SF*, the algorithm provides for a decrease in the strength properties of the ground. To do this, unbalanced forces were introduced into the model, and the equilibri-

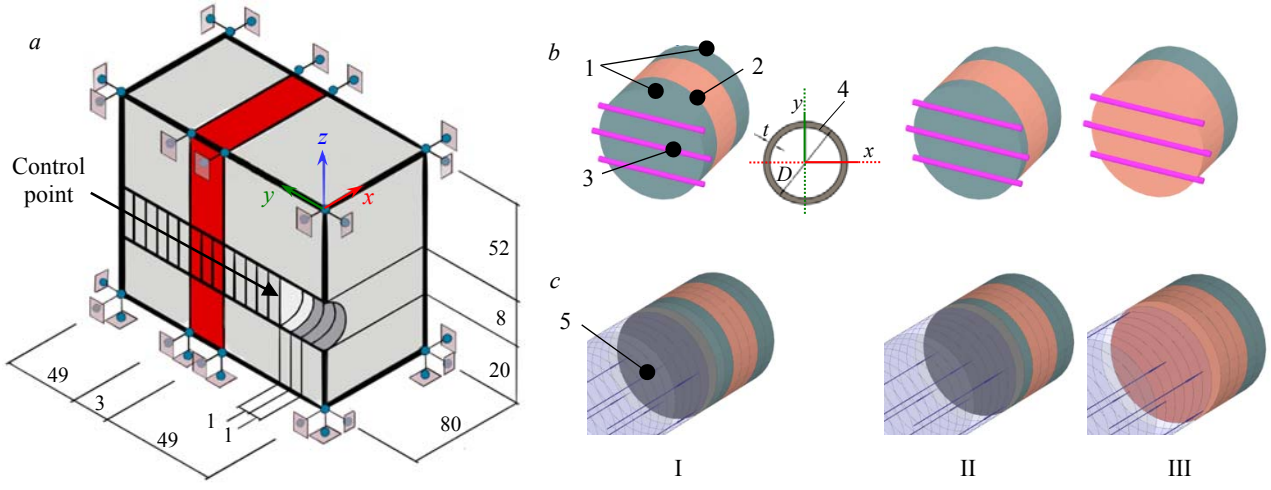


Fig.1. The calculation scheme (*a*) and the stages of the tunnel: modeling the work of the support in an explicit form (*b*); in an equivalent form (*c*) at a distance of 2 m from the disturbed (I), 1 m (II), on the disturbed (III)
1 – unbroken rock mass; 2 – structural and mechanical disturbance; 3 – beam element of finite bending stiffness (bunton); 4 – cross section of bunton; 5 – uniformly distributed equivalent pressure

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um of the system was disturbed. The imbalance of forces led to the development of deformations. At the same time, the incremental ratio of deformations caused by the introduction of new unbalanced forces at two adjacent calculation steps was subjected to constant analysis. This algorithm was repeated until an avalanche-like growth of deformations was recorded at the control point (Fig.1) between successive steps of introducing unbalanced forces. At this stage of the calculation, a constant value of *SF* is achieved, numerically reflecting the criterion of soil strength as the ratio of shear stresses in the ground to the limiting stresses at which a shift along the slip site is realized.

The system "ground – disturbance – tunnel face support" was modeled as follows. Two calculated geological elements were used as a continuous medium, one of which was described by the properties of an unbroken rock mass, the other – by the properties of a disturbed one. Plate elements with a thickness of 0.35 m were used as lining elements. The temporary face support was modeled using beams element of various pipes diameters. The sealing of beams into the rock mass was assumed to be equal to 0.35 m. The parameters of the physical and mechanical properties of the ground are presented in the table.

Material Model Parameters

Parameter	Outside the disturbing zone	In the disturbing zone
Volume weight of non-saturated soil γ_{unsat}, kN/m^3	22	22
Volumetric weight of water-saturated soil γ_{sat}, kN/m^3	22	22
The failure ratio R_f	0.9	0.9
Initial porosity coefficient e_{int}	0.5	0.5
The exponent parameter m	0.75	0.75
Poisson's ratio during unloading v_{ur}	0.2	0.2
Strength criterion	Mohr – Coulomb	Mohr – Coulomb
Secant stiffness E_{50}^{ref}, MPa	75	50
Unloading stiffness E_{ur}^{ref}, MPa	225	150
Effective cohesion c', kPa	75	25
Effective angle of internal friction φ', deg.	12	6
Tensile strength σ_t, kPa	400	0
Over-consolidation ratio *OCR*	10	3

The sequence of carrying out a predictive assessment of the tunnel face stability in the disturbed zone: assignment of boundary conditions of the 3D model; justification of the size of the disturbed zone and the accepted physical and mechanical parameters of the accepted model; allocation in the rock mass the area of influence of the disturbed zone on the controlled parameters of the stress-strain state of the medium; determination of the dynamics of changes in the *SF* coefficient based on modeling the work of the support at each stage of tunnel construction when approaching the disturbance.

The vertical boundaries of the model are fixed from moving in normal directions, but can move freely in the plane of fixation. The lower boundary of the model is fixed from moving in any direction, while the soil surface is free from fixing (Fig.1). The diameter of the tunnel was set to 8 m. The thickness of the zone of weak ground is 3 m. The vertical lines in Fig.1 show the stages of tunnel development. The amount of lagging of the lining from the tunnel face is 1 m. The width of the lining ring is 1 m. The change in the bending stiffness of the beam elements was achieved by changing the diameter of the buntons. The grade of steel and the thickness of the pipe wall remained constant.

Results. *Determination of the effect of the stiffness of the support on the stability of ground in the tunnel face.* To determine the category of stability, isofields of movements of the face surface were constructed depending on the stiffness of the temporary support (Fig.2). Despite the fact that the safety factor on the face surface (*SF* = 1.16) will be met in case of a disturbed when supporting

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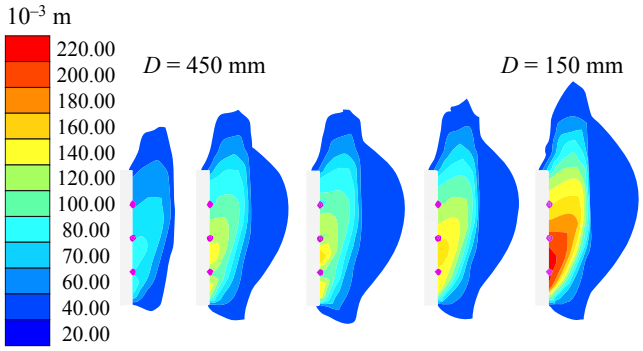


Fig.2. Isofields of displacements during the calculation phase

the face with buntons with a diameter of 150 mm, according to the deformation criterion of stability at a displacement level of 240 mm, the condition of the ground is classified as unstable.

With a diameter of 450 mm of the buntons, deformations of about 7 cm are realized in the face, which, in accordance with the deformation criterion proposed by N.S.Bulychev, belongs to the category of medium-stable state of sedimentary rocks. In the study of strength for the situation of supporting the face with buntons with a diameter of 450 mm, the safety factor will also be satisfied (*SF* = 1.33), and according to the deformability factor, the condition of the soils in the face will be medium-stable.

Based on the criterion of stability in deformability and strength, it can be concluded that with a decrease in the bending stiffness of the temporary face supporting, a change in the category of stability occurs, i.e. the rationally selected stiffness of the temporary face support can affect the stability of ground in the tunnel face.

Determination of the value of the equivalent pressure on the tunnel face. The influence of pressure distributed over the tunnel face surface on deformation was investigated in [41]. In the work [42], the dependence of the resistance pressure on the change of the strength parameters of the ground was revealed. To investigate the possibility of setting the stiffness of the support by equivalent pressure from the side of the support, modeling and comparison of collapse zones from the Mohr – Coulomb strength criterion were carried out. Using the *SF* detection algorithm, the volume of rock involved in the displacement processes was allocated in the ground. The formed rockfall can be visualized at the stage of determining the *SF* criterion with the help of deformation increment isofields. Figure 3 shows the results of determining the shape of the loss of balance of the face rocks with the diameter of temporary structures for the face timbering with beam elements with a diameter of 150 mm (Fig.3, *a*) and an equivalent pressure of 18.87 kPa (Fig.3, *b*). The blue color in Fig.3 indicates the surface that is the boundary of the increment of displacements at the *SF* determination stage, beyond this surface, deformations associated with the introduction of unbalanced forces are not realized in the rock mass and the Mohr – Coulomb strength condition is met.

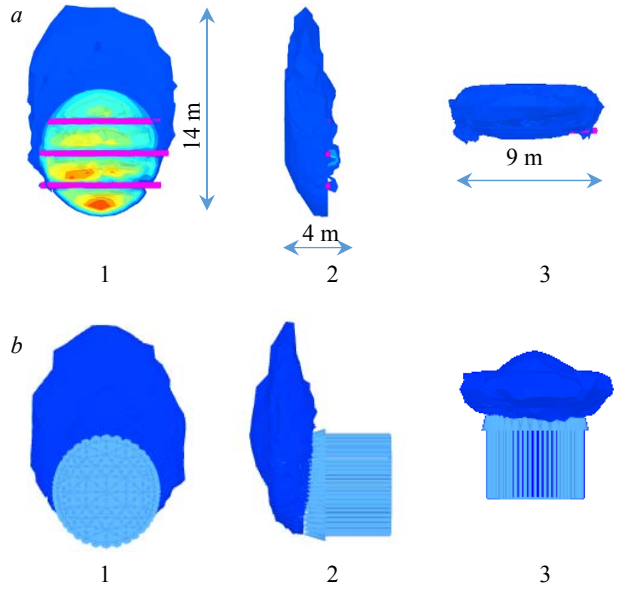


Fig.3. The result of forecasting the zone of possible rockfall under load on the face:
a – the main species with the diameter of buntons of 150 mm;
b – the main species at an equivalent pressure of 18.87 kPa
1 – front view; 2 – left view; 3 – top view

The compared volumes of soils involved in deformation allow to conclude that the loss of strength occurs according to similar situations. Elongated upward rockfall is typical for a disturbed ground and is associated with the accumulation and increase of plastic shear deformations and shear stresses that increase when approaching the disturbance.

Determination of the effect of changes in the stress-strain state on the strength of rocks in the face. When the face approaches the zone of disturbed soils, for all calculated situations of the

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tunnel face supporting, an increase in the volume of disturbed soils involved in shear processes occurs. For a qualitative comparison of the results by the strength criterion, the ratios of the stresses acting in the rock mass to the limiting shear stresses of the open face of the tunnel were determined (Fig.4). According to the stress isofield, the volumes of disturbed soils involved in shear processes were determined.

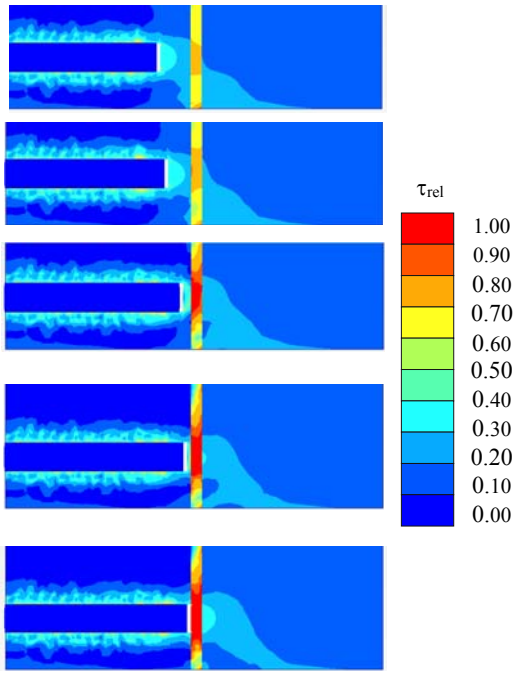


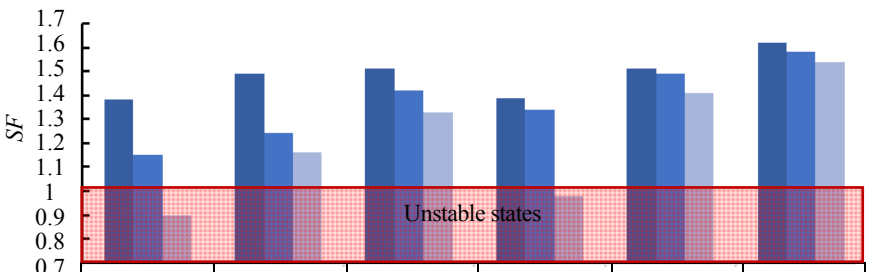
Fig.4. Isofields of the ratio of shear stresses acting in therock mass to the limiting ones

Dynamics of changes in the SF criterion depending on the distance to the zone of disturbance and the type of support. The study of the dynamics of changes in *SF* began in the zone of influence of disturbance at a distance of 2 m from the disturbance. The determination of the zone of influence is justified by preliminary calculations of displacements along the axis of the tunnel as the tunnel is built. The zone of influence was considered to be the distance at which the deformation of the face surface increased by more than 5 % relative to the average displacements. The technology of the tunnel advanced by full cross-section with a depth of 1 m. For clarity of the presentation of the stages of tunneling, Figure 1 shows three-dimensional elements isolated from the rest of the model, corresponding to the stages of tunneling.

The calculated situations of the face approach to the zone of disturbed soils, depending on the parameters of the support, are divided into six groups (Fig.5): 1) the face approaches the zone of disturbance without any supporting; 2) the diameter of the bunton elements is 150 mm; 3) the diameter of the bunton elements is 450 mm; 4) equivalent pressure on the front of the face – 2.23 kPa; 5) equivalent pressure on the front of the face – 37.18 kPa; 6) equivalent pressure on the front of the face – 75.52 kPa.

According to the design scenario describing the entry of the face into the zone of disturbance influence without support (group 1), there is a loss of strength of the face soils on the disturbance, before the approach to the disturbed the margin reached 38 %. At an equivalent pressure of 2.23 kPa, the entry of the face into the disturbance zone will be accompanied by a sharp excess of shear stresses relative to the limit due to the fact that the disturbance zone is a “storage” of historical plastic shear deformations.

Factors causing the loss of stability and balance of the face soils. At the position of the face at a distance of 2 m from the disturbed in the level of the upper bunton, a zone of concentration of volumetric deformations is formed. Normal stresses are responsible for the appearance of volumetric deformations, which are realized by extracting the volume of soil and putting the support into operation. The



	1	2	3	4	5	6
–2	1.38	1.49	1.51	1.39	1.51	1.62
–1	1.15	1.24	1.42	1.34	1.49	1.58
0	0.9	1.16	1.33	0.98	1.41	1.54

Fig.5. Histogram of *SF* dependence on the distance to the disturbance (–2; –1; 0 m) with different parameters of the face support

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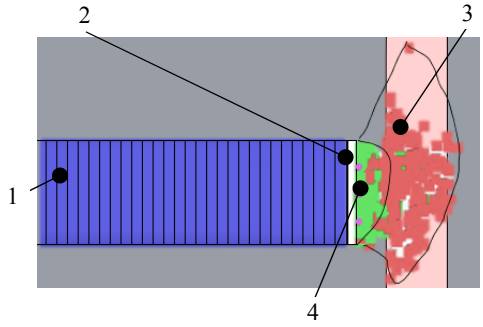


Fig.6. A fragment of the model at the stage of static balance
1 – tunnel lining; 2 – construction of a temporary face support; 3 – areas of exceeding the maximum shear stresses on the site; 4 – areas of exceeding the maximum tensile stresses

further redistribution of stresses revealed by the *SF* determination algorithm shows that as the face enters the zone of disturbed soils, areas of exceeding the tensile strength will form at the face. The rockfall formation takes the shape of a lens, the contour of which is indicated in green in Fig.6. A decrease in the strength of the rock mass leads to an increase in the volume of soils involved in shear processes. A new rockfall formation is being formed. The areas of localization of volumetric deformations are shifted to the future sliding surface and form the contour of the second rockfall formation in the form of a drop.

The relatively small volume of the first collapse is associated with exceeding the tensile stress limits. This is confirmed by the visualization of individual areas in the calculation model in which the tensile strength has been exceeded (green dots in Fig.6). Shear stresses contribute more to the further collapse of soils. The red color in Fig.6 shows the points at which the maximum shear stresses at the sites are exceeded. At the same time, the picture of the visualized regions of the limit state may have larger geometric parameters than on the displacement isofields. This is due to the involvement of not the entire volume in the processes of active movement.

Thus, the flexural stiffness of the temporary supporting of the face affects the state of stability in the face. Regardless of the method of supporting the face soils, shear stresses and angular plastic deformations of shape change accumulate on disturbed soils. When the face approaches the zone of disturbed soils in scenarios of lack of support or insufficient resistance (less than 2.23 kPa) from the side of the support in the face soils, the strength criterion for tensile stresses and shear stresses is not met.

Discussion. For a comparative analysis, the case of the rockfall formation of the soil in the tunnel face of the transition corridor of the interchange node of the station "Vladimirskaya-2" under construction from the operating station "Vladimirskaya", which occurred in October 1989, was considered. "On October 21, 1989, the 4th bunton unexpectedly sagged and was squeezed out, the rock fall from the face with a volume of 10-15 m^3. An hour later, a rockfall from the roof of the face with a volume of about 2 m^3 occurred again, and after another rockfall, all the roof fastening, the face and the filling were demolished. In the roof above the 6th ring of the lining, a dome with a height of up to 7 m with a volume of 170 m^3 was formed" [30]. The experience of measuring loads on the structure of the leading support of an underground structure is described in [40]: "The load on the support from the rock pressure on the 60th day was 110 kPa", which for the conditions under consideration is equivalent to a pressure of 0.1 γN. The distributed pressure from the weight of the rocks collapsed in the "Vladimirskaya-2" face over the face area is about 80 kPa, which is less than recorded in the study [43]. Despite the fact that the loads were less than expected, the collapse that occurred destroyed the face support and exposed structures on the surface to the risk of excessive displacements.

Thus, the calculations made it possible to describe the collapse that occurred in 1989. At the same time, there is a qualitative convergence of the assumed parameters of the collapse. The numerical value may differ due to the lack of data on engineering and geological parameters at the construction site.

Conclusion. Using the proposed spatial elastic-plastic model with strengthening, a prediction of the stress-strain state in the ground with disturbed soils was made. Based on the safety factor and loading capacity, a forecast of rockfall in the face zone of the tunnel is made. The spatial outlines of the rockfalls and their dimensions have been established, which are the theoretical basis for calculating vertical loads on the temporary support of the face in the disturbed zone.

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Numerical experiments on spatial elastic-plastic models have revealed the mechanism and patterns of formation of zones of the limiting state as they approach the zone of disturbed soils. Zones of excess tensile stresses develop on the face surface and a sliding surface is formed.

Modeling of finite elements showed the dynamics of changes in stability and the *SF* parameter for various design situations of the tunnel face entering the disturbance zone. The analysis of the obtained dependencies showed that the largest value of *SF* is characteristic of the undisturbed soil area, and the decrease in the *SF* parameter occurs as it approaches the disturbance zone in all design situations with different dynamics.

The use of the method of iterative reduction of strength parameters in assessing the stability of the tunnel face determined the forms of possible loss of stability. When comparing the obtained forms of the rockfall at the "Vladimirskaya-2" station with the data of field observations, a qualitative agreement of the form of the rockfall was established.

A predictive assessment of stability and the formation of loss of balance based on an elastic-plastic model with strengthening will make it possible to detail the technological stages of the tunnel, taking into account the stresses historically formed during the previous stages of tunneling. The proposed method for assessing the stability of the tunnel face differs from the known methods by taking into account the volumetric stress state of the disturbed ground, the possibility of determining the disturbed zones in the face depending on the action of tensile stresses and limiting states during plastic deformation.

The result of using the proposed method is a qualitatively comparable shape of the spatial outline of the rockfall and the determination of the magnitude of the loads on the temporary support of the tunnel face. This makes it possible to calculate the support according to the scheme of the specified loads and consider many design situations within the framework of one predictive geomechanical model.

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