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Method for predicting the stress-strain state of the vertical shaft lining at the drift landing section in saliferous rocks

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The article proposes a method for predicting the stress-strain state of the vertical shaft lining in saliferous rocks at the drift landing section. The paper considers the development of geomechanical processes in the saliferous rock in the landing area, the support is viewed as a two-layer medium: the inner layer is concrete, the outer layer is compensation material. With this in view, the paper solves the problem of continuum mechanics in a spatial setting, taking into account the long-term deformation of salts and the compressibility of the compensation layer. Long-term deformation of saliferous rocks is described using the viscoplastic model of salt deformation into the numerical model, and the crushable foam model to simulate the deformation of the compensation layer. This approach considers all stages of the deformation of the compensation layer material and the development of long-term deformations of saliferous rocks, which makes it possible to increase the reliability of the forecast of the stress-strain state of the vertical shaft lining.

Key words: geomechanics; saliferous rock mass; prolonged deformations; viscoplastic model; compensation layer; landing section; lining

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Introduction. The sinking of vertical shafts in saliferous rocks is associated with deformations of the rock boundaries over time. The data of field observations indicate that these deformations are non-steady [5, 6] and can develop during the operation of a vertical shaft. At low depths (up to 200 m), the shafts passing through saliferous rocks were not strengthened, however, with an increase of the mining depth (over 300-400 m), the rock started to actively show plastic deformation, which, according to researches, occurs at tangential stresses of the shaft boundaries of 10-15 MPa [2, 10]. At great depths (1000 m and more), the development of rheological processes is accompanied by significant deformations of the rock boundaries of vertical shafts, and the load on rigid lining increases to 20 MPa or more over time.

Thus, when designing the sinking of vertical shafts in saliferous formations, one should calculate the support for a given operational period. Special attention should be paid to the support of the landing areas of vertical shafts with drifts [16]. The lining of vertical shafts is usually made of materials with high deformation characteristics, which prevents the development of deformations of the bed boundaries. At the same time, the lining cannot undergo significant deformations until its load-bearing capacity is lost. The bearing capacity may be lost even before the end of its operation period due to the peculiarities of the development of bearing stresses from the interaction of support and rock. The development of rheological processes in saliferous rocks can lead to the destruction of the lining, which was observed in many mines [11, 12]. So, at the Fourth Saligorsk Potash Mine, the monolithic concrete lining with thickness of 600 mm, located at a depth of about 800 m, collapsed. It failed due to significant deformation of the lining-rock mass system. A similar situation was observed in the mine n.a. Sverdlov and Kulushsky mine n.a. the 50-letie Oktyabrya. The displacement of the shaft walls in the latter exceeded 200 mm. The analysis of the previous researches and the conclusions presented in [9] show that with greater mining depth, the pressure on the vertical shaft lining increases.

To reduce the negative impact of the development of deformations in saliferous rocks due to the stress-strain state of the support, the intermediate compensation layer is introduced between the lining and the shaft rock boundaries [2, 13, 14]. The pliable layer during its deformation is com-

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pacted due to changes in structural porosity, forming a reduced (in comparison with the full) load on the lining. This effect lasts until the material of the compensation layer reaches the compression limit and can no longer compensate the load transmission. Usually, the parameters of the compensation layer are selected based on the expected value of the radial displacements of the rock mass. The problems of calculating the support of workings in saliferous rock are thoroughly studied in the works of N.S.Bulychev [2, 3], however, the mechanical behavior of the compensation layer is not explicitly described, but is specified through the equivalent deformation index, which can negatively affect the forecast of the stress state of the lining during uneven loading of the shaft lining or when considering the landing of the vertical shaft and drifts. In this paper a method for predicting the geomechanical processes of rock mass in the vertical shaft landing area and the stress state of the support based on numerical spatial modeling is proposed. The long-term saliferous rock deformation is considered explicitly, and the problem is solved based on the time factor. We also pay attention to the deformation of the compensation layer and propose a model of deformation of this porous medium that allows predicting the behavior of the material of the compensation layer in the entire range of deformation.

Development of the method for predicting the vertical shaft landing support load considering the long-term deformation of saliferous rock. The analysis of field data is carried out on the results obtained at the Verkhnekamsk deposit of potassium and magnesium salts [1, 4]. The rock mass at the Verkhnekamsk potash deposit is characterized by clear rheological properties and interbedding of sylvinite and rock salt, often separated by thin layers of clay. Displacements and deformations of rocks near the developing entries, not affected by the heading and clearing operations [1], were studied at measuring stations equipped with deep and mapping reference points in the roof, soil, and walls of the mine (Fig.1, *a*). The stations were located at the depth of 330 m in the developed header panel entry along the mined Krasny-II sylvinite layer at the Second Bereznikovsky mine department. The Krasny-II layer (seam thickness is 4.5 m) consists of seven sylvinite and thin halite layers. The roof has a 70-centimeter suite bench of interbedded layers of rock salt and clay, there is the rock salt layer above and in the soil. The bedding is horizontal and complicated by faulting. The results of three-year observations of the deformations of the displacements of mine delineation map are presented in Figure 1, *b*. The displacements develop over time and do not show a tendency to weakening. In comparison with the initial stage, the rate of displacement decreases and after 200-300 days it practically stabilizes. The observation results allow us to make another important conclusion: there are no obvious differences in convergence of reference points in the working of a circular cross-section in different directions. It allows us to assume that the stress distribution in the undisturbed saliferous rock mass is close to hydrostatic (the lateral pressure coefficient in the mass is $\lambda \approx 1$).

In the work of A.S.Ermashov [4], the results of field observations of the deformation of the rock boundaries in underground workings are given considering their mutual influence (Fig.1, *b*). Measurements were reduced to registering the vertical and horizontal movements of contour reference points in time. Based on the conditions presented in the work [4] we developed a numerical model to compare the results of field observations of deformations of mine workings and calculated values based on the accepted rheological model. The average depth of the rooms is 400 m. The dimensions of the rooms are 3.1 × 5.5 m and 3.9 × 5.5 m. Although there is a different displacement of the walls in workings, in general over the observation period (6 years) mutual displacement between reference points along the base equals Δh = 6-26, and along the base – Δl = 18-32 mm. Having discarded the data that are out of the general trend, we take the displacements of 22 and 32 mm corresponding to the finite displacements for the Δh and Δl bases, respectively, as typical dependences of the development of deformations in time.

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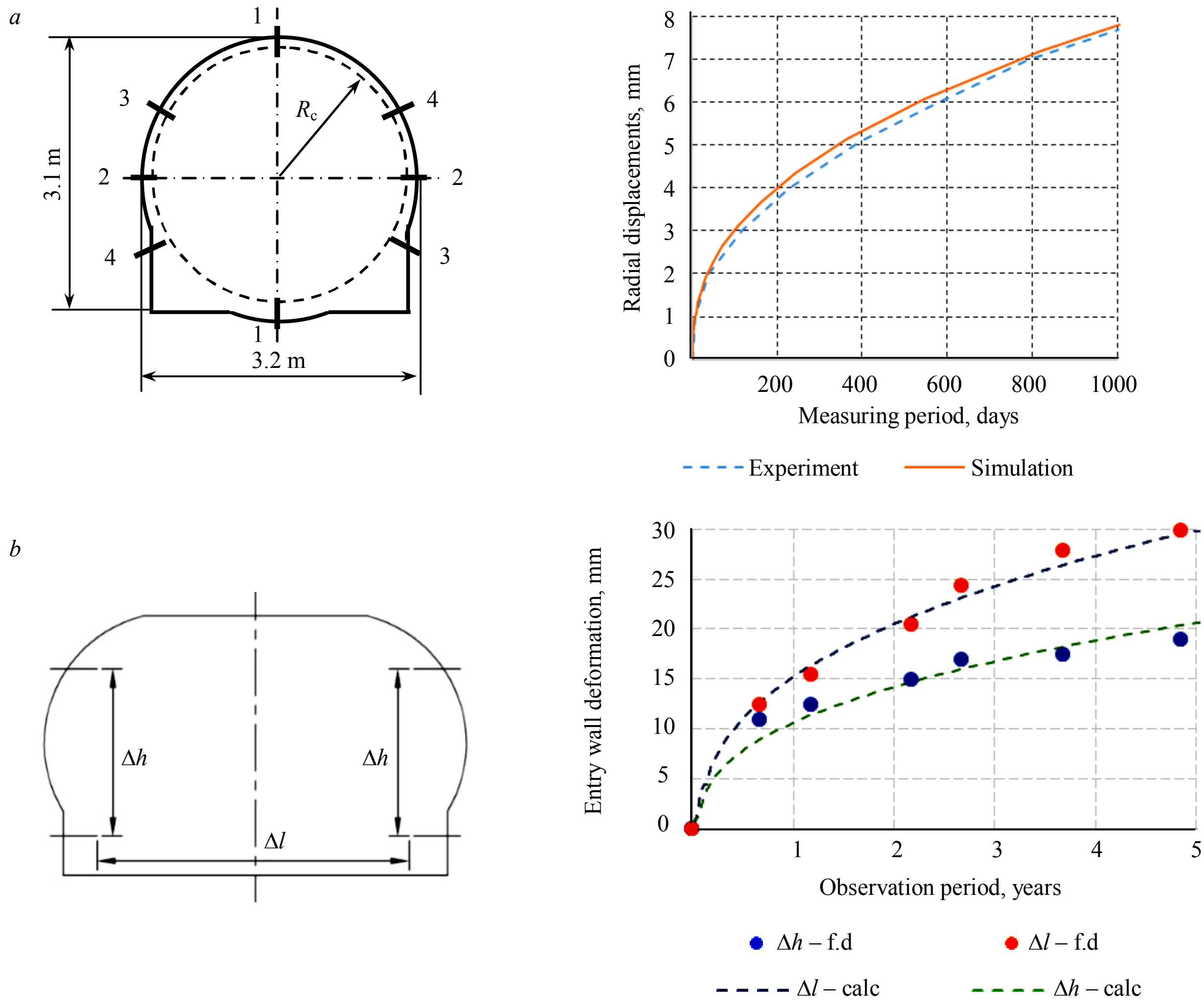

Fig.1. Cross-section of belt road and measured [1] and calculated displacements of rock mass of entry cross-section (*a*), typical pattern of reference pionts placement at underground observation stations and the results of comparing field measurements with the results of numerical modeling (*b*) [13]: Δh – vertical measurement base; Δl – lateral meausrement base; f.d – field data; calcualtion – numerical modelling.

The development of deformations of rock outcrops was used to justify the parameters of the rheological model of saliferous rock behavior, to forecast the geomechanical processes in the rock mass near the vertical shaft, and to determine the lining design and the load on the support over time.

We applied the phenomenological model based on the power law to describe long-term deformations [17]. Although this model allows us to describe all three stages of soil creep by introducing the function of long-term strength development, only the first two stages are considered in the paper: transitional and steady-state soil creep, and the third stage – progressive creep – is not included in the calculation process. The relationship between the increment of creep deformation and the state of the rock mass is presented in the following analytical form:

$$\dot{\varepsilon}_{cr} = \left(A(\overline{\sigma}_{cr})^h[(m+1)\overline{\varepsilon}_{cr}]^m\right)^{\frac{1}{m+1}}, \quad (1)$$

where A, m, n – model indicators; $\overline{\sigma}_{cr}$ – equivalent creep stresses; $\overline{\varepsilon}_{cr}$ – equivalent relevant creep deformations.

The results of testing the model for predicting the development of long-term deformations of the saliferous rock mass are presented in Fig.1. As we can see, the adopted viscoplastic model gives satisfactory convergence with the results of field observations from a qualitative and a quantitative viewpoint. The parameters of the model of a viscoplastic medium are given in Table 1. Line 1 describes justified indicators for depths of 300-400 m. The development of geomechanical processes in saliferous rocks at different levels of stress state can be better described through a set of indicators presented in the second line (Table 1).

Table 1

The parameters of the rheological model of salts

N s/n	Elasticity indicators		Soil creep model indicators			Long-term uniaxial compression strength σ_s^∞, MPa	Long-term strength coefficient k_l
	Deformation modulus E_i, MPa	Lateral deformation coefficient v_0	A	n	m		
1	20000	0.32	$1\cdot10^{-11}$	1	−0.59	12.5	0.5
2	20000	0.32	$1\cdot10^{-25}$	3	−0.43	12.5	0.5

Based on the results of testing the material of the compensation layer (the studies were carried out in the laboratory of the Scientific Center for Geomechanics and Mining Issues of the Saint-Petersburg Mining University), diagrams of the development of stresses and strains under conditions of odometric compression were obtained (Fig.2, *b*). The diagram shows two main areas of deformation. The first area is the section of primary deformation, which extends until the stresses reach values corresponding to the moment of the beginning of compaction of the porous structure of the compensation material. This section covers a small part of the overall deformation diagram of the compensation material and can be represented by a linear curve. The second section characterizes the compaction of the structure of the porous material, which is accompanied by a loss of stability of the pore walls. This section covers relative deformations in the range from 0.02 to 0.95.

To describe the mechanical behavior of compensation materials under the influence of an external load, the model of the crushable foam model was adopted [15]. The model is designed to describe the plastic deformation of the material, which volume deformations occur due to the loss of stability of the walls of the porous structure. Two stages of deformations are distinguished: 1 – elastic deformations; 2 – plastic deformations. The surface of the plastic changes of crushable foam model (Fig.2, *a*) is determined by the formula

$$F=\sqrt{q^2+\alpha^2(p-p_0)^2}-B=0, \tag{2}$$

where q – von Mises equivalent stresses; p – average stresses; α – plastic deformation surface shape coefficient; B – parameter determining the size of the plastic deformation surface along the q axis; p_0 – center of the plastic deformation surface along the p axis.

The law of plastic strengthening of the crushable foam model can be represented as

$$p_s\left(\varepsilon_{\text{tot}}^{\text{pl}}\right)=\frac{\sigma_s\left(\varepsilon_{\text{uniax}}^{\text{pl}}\right)\left[\sigma_s\left(\varepsilon_{\text{uniax}}^{\text{pl}}\right)\left(\dfrac{1}{\alpha^2}+\dfrac{1}{9}\right)+\dfrac{p_t}{3}\right]}{p_t+\dfrac{\sigma_s\left(\varepsilon_{\text{uniax}}^{\text{pl}}\right)}{3}}, \tag{3}$$

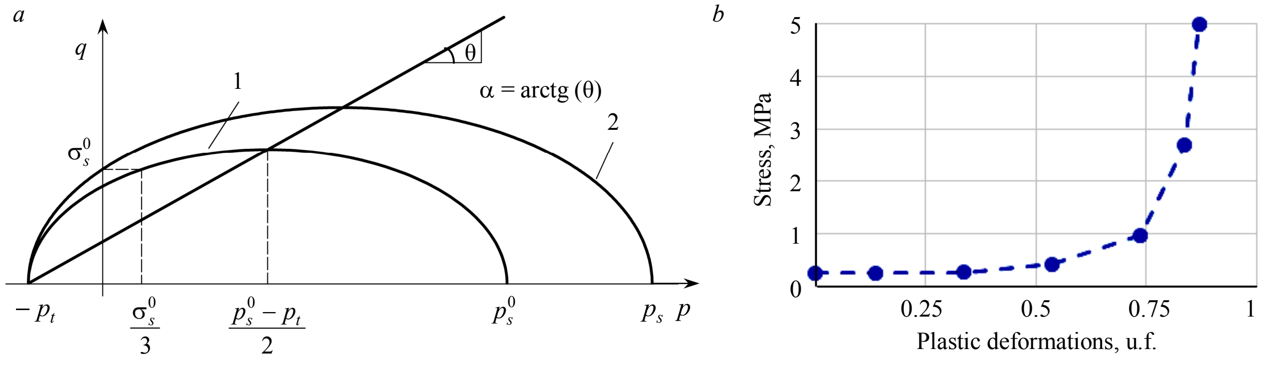

Fig.2. The surface of the plastic deformation of the crushable foam model (*a*) and the compaction diagram of the compensation material (*b*), that were adopted for numerical simulation

1 – initial position of the surface of the plastic deformation; 2 – plastic deformation surface after strengthening

where ε^{pl}_{uniax} – relative plastic strains under uniaxial compression; ε^{pl}_{tot} – relative plastic strains under isotropic compression; p_s – position of the surface of the plastic deformation in the compressed zone on the p axis; p_t – fixed value of the position of the surface of the plastic deformation in the stretched zone on the p axis; $\sigma_s(\varepsilon^{pl}_{uniax})$ – functional relationship between the indicators σ_s and ε^{pl}_{uniax}; σ_s – starting position of the surface of the plastic deformation at a given value ε^{pl}_{uniax}.

The formation of the load on the vertical shaft lining considering the long-term deformation of saliferous rock mass. Saliferous rocks, as noted above, are prone to significant deformations. The construction of underground structures in such an environment causes instantaneous and long-term deformations of the rock boundaries, which are associated with the rheological features of saliferous deformation process. The magnitude of the disturbances mainly depends on the mechanical behavior of the salt and the size of the rock outcrop. The installation of rigid supports can be rational at low depths and small cross-section areas.

The load on the vertical shaft lining in saliferous rocks directly depends on the rate of development of soil creep deformations. The paper [6] presents the results of field observations of the formation of the load on the vertical shaft lining and its stress state. According to the results of field observations in shaft N 9 of the Zakarpatsky mine, the dependences of the load formation on the support were obtained (Fig.3, *b*). We can see that at the end of the measurements, the tangential stress in the concrete lining was 23 MPa, in cast-iron liners it was 74.2 MPa. The average load on the vertical shaft lining was 4.5 MPa. As the authors note, the tendency to stabilize the deformation of the lining, and accordingly the changes in its stress state, was not observed. Similar studies were conducted at the Piylo mine for seven years. The observation results are given for a period of 230 days. Tangential stresses in the back wall of the liner after 90 days reached 81 MPa [6].

Let us consider the formation of the load on the vertical shaft lining according to the method proposed above and compare it with the actual measurements (Fig.3). Construction conditions and the main parameters of the vertical shafts support are given in Table 2. The elastic modulus of iron cast SCh25 is assumed to be 100 GPa, the Poisson's ratio is 0.3. The concrete deformation modulus is conventionally assumed equal to 30 GPa, the Poisson's ratio is 0.2. The cast-iron lining was considered as a two-layer ring, the first layer of which modeled the behavior of the rib, the second layer reflected the behavior of back walls. The parameters of each layer were determined on the reinforcement coefficient [3], which was taken equal to one for back walls, and for the ribs it was determined based on their total actual thickness and the height of the lining.

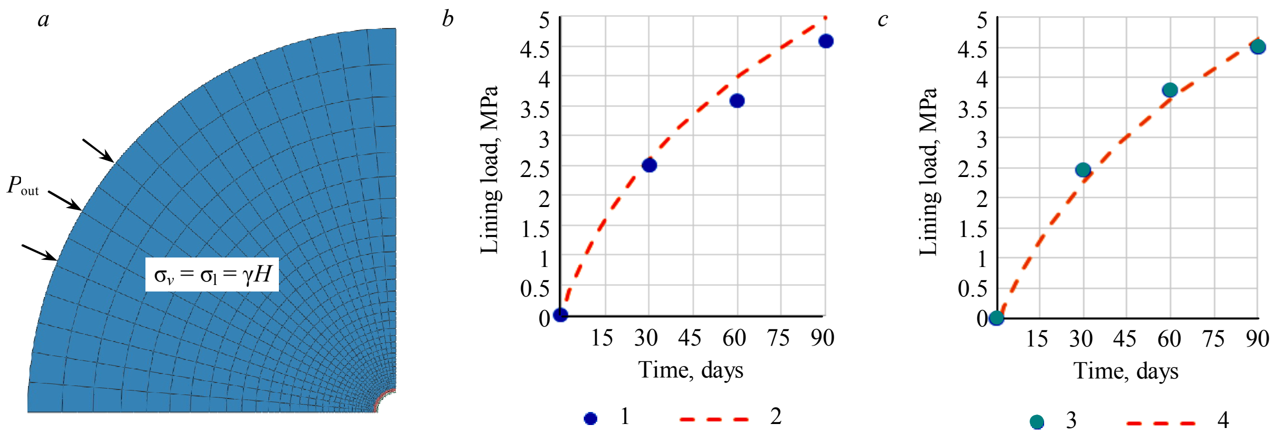

Fig. 3. Model for predicting SSS of the vertical shat lining (*a*) and the development of radial stresses in the «lining-rock mass» contact area in the shaft number 9 of the Zakarpatsky salt mine at a depth of 492 m (*b*), in the shaft of the Piylo mine at a depth of 491 m (*c*); FD – field data; NS – numerical simulation
1 – Zakarpatsky mine (FD); 2 – Zakarpatsky mine (NS); 3 – Piylo mine (FD); 4 – Piylo mine (NS)

Table 2

Construction conditions and parameters of vertical shafts support [6]

Construction conditions	Target research objects	
	Shaft N 9 of Zakarpatsky mine	Ventilation shaft of Piylo mine
Depth, m	492	491
Estimated value of longitudinal stress, MPa	10	10
Mining and geological conditions	Rock salt	Saliniferous clay
Work area	Newly built	Reinforcement section
Shaft drilling diameter, m	6.0	7.0
Type of cast-iron lining	Lining 6.0-60	Lining 7.0-60
Thickness of the back walls of the cast-iron lining, mm	60	60
Thickness of the ribs of the cast-iron lining, mm	210	215
Lining rib reinforcement coefficient	0.185	0.165
Cast iron material	Cast-iron SCh25	Cast-iron SCh25
Concrete layer thickness, mm	430	500

A comparison of the results of numerical modeling (Fig.3, *b*) with the actual measured values of the load on the lining showed their good convergence in the considered time period (90 days). The subsequent development of support loads according to field observations for the two considered shafts varies: if in the first case (shaft N 9 of the Zakarpatsky mine) the support loads continue to grow, then in the second (ventilation shaft of the Piylo mine) its stabilization is observed. The calculation results also show a further increase in the load on the lining, and their nature of development qualitatively and quantitatively corresponds to the measured data. Good convergence is also observed in terms of tangential stresses in the back walls of lining. Thus, the calculated value of the tangential stresses in the lining of the shaft N 9 of the Zakarpatsky mine was 84.2 MPa, and the lining of the ventilation shaft of the Piylo mine was 84.8 MPa.

In general, we can note a good convergence of the predicted and actual indicators of the lining stress state, and the adopted approach and model parameters can be taken as a first approximation when calculating the stress state of the vertical shaft lining at the landing section.

Numerical modeling of the development of long-term deformations and the load on the lining of a vertical shaft near the landing areas. As many researchers have noted, the vertical shaft landing section is the most unfavorable in terms of ensuring the stability of rock outcrops and the formation of loads on the lining of the mine workings. The operation of salt mines has shown that the technical condition of pairing vertical shafts with drifts is unsatisfactory. The main reasons for this are the intensive development of geomechanical processes, which manifests itself in the form of excessive deformations, and the destruction of rocks in the border region of the shaft. An increase in the exposure surface, as well as the formation of many free surfaces, leads to accelerated development of deformations, the magnitude of which exceeds the displacement rate in the area of the extended part of the vertical shaft by ten times [7]. The load on the vertical shaft lining in the landing sections in saliferous rocks can significantly exceed the weight of the overlying rocks.

The article describes the landing of the vertical shaft and a drift at depths over 1000 m, where the normal stress tensor components in an undisturbed rock mass are averagely equal to 25 MPa, which exceeds their instantaneous strength for individual salt rocks and long-term strength for all saliferous rocks. Considering the concentration of stresses near the rock outcrops of the vertical shaft and drifts, we can speak with confidence about the development of loads higher than the bearing capacity of rigid monolithic concrete or reinforced concrete linings. Thus, the use of rigid types

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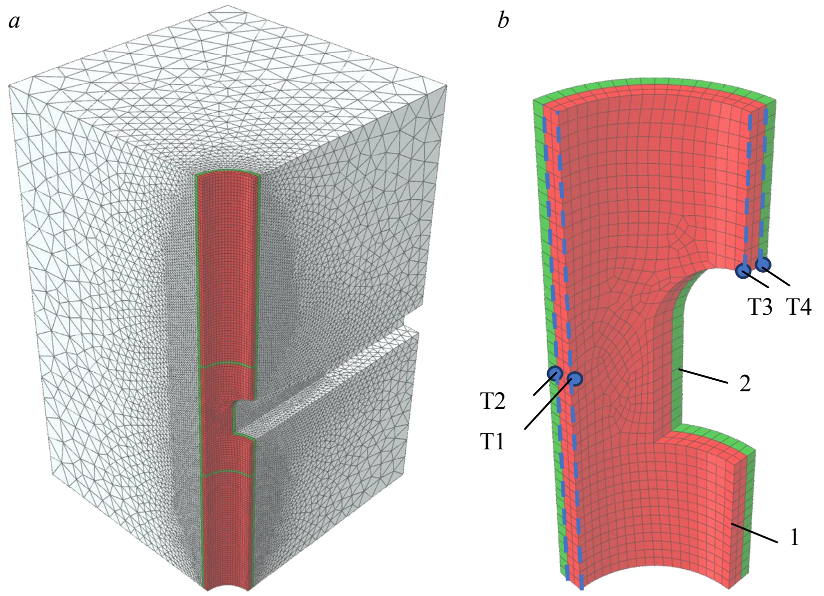

Fig.4. The finite-element model of the bilateral junction of the vertical shaft and drifts: *a* – rock mass and the landing section; *b* – model of lining at the landing section.

1 – monolithic reinforced concrete lining; 2 – compensating layer;
T1-T4 – measuring points

of supports in this section of the vertical shaft is unacceptable (the possibility of using cast-iron lining in the vertical shaft landing section is not considered in the paper).

Pliable types of lining for an extended section of the shaft make it possible to compensate for deformations of the «lining – rock mass» contact area and, with the right choice of the pliable layer, ensure the bearing strength of the lining throughout the entire period of operation of the vertical shaft. However, the effectiveness of this type of lining in the landing areas is not confirmed. As permanent support, it is rational to use a monolithic reinforced concrete lining, which main purpose is to redistribute local stresses in the event of an uneven load at the «lining – pliable layer» contact area.

Numerical modeling of the formation of a stress-strain state in the vertical shaft and drifts landing was performed in a spatial setting (Fig.4). The model included the rock mass, a compensation layer of the required thickness and a monolithic reinforced concrete lining with a thickness of 0.65 m. The initial thickness of the pliable polystyrene layer was 0.35 m. The drilling diameter of the vertical shaft was 7.0 m. The calculation was carried out in an unsteady setting, which allowed to consider the development of prolonged deformations of saliferous rock. We used the soil creep model based on a power law expressed in terms of the recorded creep deformation values to describe the saliferous rock mass. When considering the bilateral junction area, the model included a quarter of the object, since it was possible to draw two planes of symmetry for which the deformations of the rock mass are the same. The finite element representation of numerical models is shown in Figure 4.

The parameters of the rheological model of the environment are given in Table 1. The salt deformation modulus is determined based on laboratory tests of rocks under uniaxial compression. The axial deformations at which the deformation modulus was determined are 0.1 %, which corresponds to the initial salt deformation modulus. Thus, the total deformation in the landing area will manifest due to viscoplastic deformation.

The mechanical properties of the model of crushable foam for expanded polystyrene are as follows: density – 300 kg/m^3; modulus of elasticity – 10 MPa; Poisson's ratio – 0.2; elastic limit – 0.3 MPa; k = 1.1; k_t = 0.1; compressibility index – see Fig.2, *b*.

We used the following steps for performing numerical simulation:

1) formation of the initial field of the stress state of saliferous rocks;

2) modeling the redistribution of the stress-strain state of the rock mass in the landing of the vertical shaft and drifts as a result of headwork;

3) modeling of long-term deformation of the rocks until the installation of the permanent lining (conventionally assumed to be 100 days);

4) transferring the load to the support of the landing support of the vertical shaft (50 years is considered).

Results of forecasting the stress and strain state of the vertical shaft lining in the landing area. Let us consider the formation of tangential stresses in the vertical shaft lining in an extended section. Tangential stresses on the internal contour of the lining during rigid contact with the rock mass sharply increase at the initial stage of operation of the lining and gradually, the increment of stresses per unit time will decrease. After 50 years, the stress in the concrete lining would have reached 104 MPa, which is significantly higher than its strength. Unlike the rigid concrete lining, the concrete lining with a pliable layer, after the initial growth of tangential stresses up to 2 MPa, the values of stresses in the lining are preserved for a rather long time. After 50 years, the stresses in the lining would have reached 12.5 MPa, which is less than the calculated concrete resistance to compression. Subsequently, the load transfer to the shaft lining through the pliable layer is accelerated, and the stress increases sharply. We can see that the presence of a pliable layer allows controlling the development of stresses in the lining of a vertical shaft; however, its thickness and mechanical properties should be adjusted according to the expected life cycle of the layer.

The calculation results of the SSS forecast for the vertical shaft lining at the landing area are presented in the form of diagrams of the development of the main maximum stresses on the internal and external contours of the shaft support (Fig.5, *b-d*). The development of stresses in time repeats

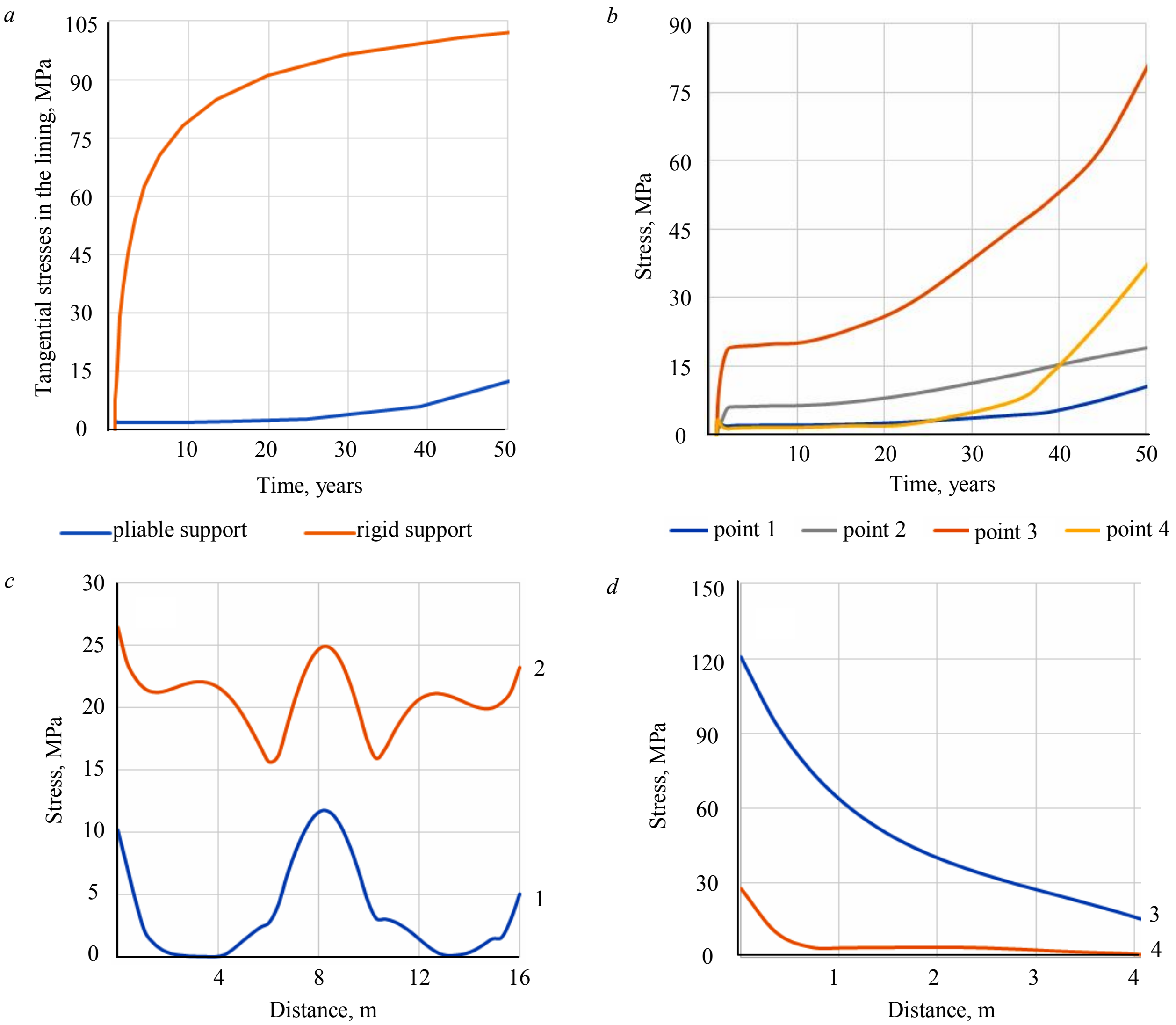

Fig.5. The development of the main maximum stresses in the vertical shaft lining: *a* – the extended section; *b* – the landing section (see Fig.4); *c, d* – distribution of tangential stresses outside and inside the opening area respectively (see Fig.4)

1, 2 – internal and external contours, respectively (outside the opening area); 3, 4 – internal and external contours, respectively (the opening area)

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the results of the formation of the stress state in the lining on the extended section of the vertical shaft. At the initial stage, the stresses increase, then they stabilize for quite a long period. In the future, as the yielding capacity is exhausted, stresses increase sharply. In this case, the stress values outside the opening section reach 20 and 11 MPa on the external and internal support lining; respectively, their values are less than the tensile strength of the calculated concrete compressive resistance. At the opening, compressive stresses of the internal counter equal to 90 MPa and in the external – 39 MPa. Thus, the stress values after 50 years from the beginning of the operation of the landing structure significantly exceed the calculated concrete compressive resistance.

Let us consider the distribution of stresses in the landing section support formed after 50 years (Fig.5, *c, d*). We can see that in the lining section outside the opening, the maximum compressive stresses are formed on the external contour – from 16 to 25 MPa along the length of the considered section. On the inner lining contour, compressive stresses range from 0 to 12 MPa. It means that tensile stresses are formed in separate areas of the lining. This type of stress state development is associated with a complex type of deformation of the lining; there is a deflection of the lining towards the center of junction area. In the section of the opening, the maximum stresses are in the zone of intensive development of saliferous rock deformations and are concentrated above the opening section. As you move away from the section of the opening, the compressive stresses quickly decrease. So, on the internal contour, the stress decreases from 120 to 11 MPa, on the external – from 30 to 2 MPa. Thus, in the lining in the opening section, the destruction of the lining from the compressive stresses is possible. The distribution of the failure (damage) area of the lining was not considered in the work, however, it can be assumed that the formation of lining local destruction areas will cause stress redistribution, as a result, the destruction areas will cover a significant part of the lining of the vertical shaft landing section.

The analysis of the formation of the lining stress state at the landing section of the vertical shaft showed that the introduction of the pliable layer in the «lining-rock mass» system makes it possible to significantly increase the lining life cycle with preserving its standard technical condition, but after a long period of time, the lining begins to collapse. Similar conclusions are also presented in [6], where it is noted that the pliable layer can significantly extend the service life of the vertical shaft lining until it is completely damaged and out of operation. Thus, the introduction of the pliable layer and its structural design will prolong the service life of the lining in the landing sections, and the calculation of the lining design parameters should be carried out for a specified period.

Conclusion. The calculation of the SSS of the shaft lining in the saliferous rock mass and especially in the drift landing areas should be based on the joint interaction pattern, where the process of transferring the load to the support is carried out considering the time factor. The paper shows the effectiveness of introduction of the pliable layer to ensure the long-term bearing capacity of the vertical shaft lining for the extended section of the shaft. At the same time, the stress state of the lining in landing sections of the vertical shaft and drifts can significantly exceed the strength of concrete, so the bearing capacity of such lining will be exceeded. Preliminarily performed numerical calculations based on the strength characteristics of concrete, allow us to assume the formation of significant damage areas of concrete lining at the landing sections. Thus, it is necessary to make changes to the design of the landing lining, since it does not meet the safety requirements for vertical shafts, one of the options of which can be to increase the thickness of the pliable layer. However, such calculations have not yet been carried out.

When performing mathematical modeling, a simplified approach to considering the mechanical behavior of the pliable layer can lead to an incorrect prediction of the moment when the lining reaches its bearing capacity. When performing further research, it is necessary to consider such important features of the mechanical properties of concrete as creep and plastic behavior, which will increase the reliability of the SSS forecast for the concrete lining. In general, the method presented in the paper can be recommended for predicting the development of the stress state of complex spatial underground structures under the conditions of the active development of rheological processes in the rock mass.

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