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THE PREDICTION OF ELASTIC-PLASTIC STATE OF THE SOIL MASS NEAR THE TUNNEL WITH TAKING INTO ACCOUNT ITS STRENGTH ANISOTROPY

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ABSTRACT

In this work the strength and deformation characteristics of clay formations in the underground space of megacities are investigated. It is found that they have anisotropy of the mechanical characteristics, which are directly connected with the processes of soil formation. The clay layered mass is seen as a transversely isotropic medium, the strength and deformation characteristics of which depend on the direction of load application relative to the plane of bedding. From the experimental results we have obtained indicators of the strength and deformation characteristics which are used as model parameters of a transversely isotropic medium in mathematical and numerical modeling. We have developed geomechanical model of a plastically anisotropic soil mass and estimated the influence of anisotropy on the limiting condition of the soil mass around the tunnel. In order to solve the problem the method of small parameter and finite element method have been used.

Key words: Anisotropy, Modeling, Zone of Limiting Sate, Tunnel.

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1. INTRODUCTION

The paper discusses issues related to the investigation of geomechanical processes in a layered soil, exemplified by Proterozoic clay being solid claystone-like rock. By taking into account the trends of recent years aimed at improving of industrial safety requirements in the field of impact assessment of construction of underground structures located on the earth's surface buildings, structures and infrastructure, the special attention should be paid to peculiarities of deformation of the Proterozoic clays in a wide range of deformation from extremely small deformations ($1•10^{-6}$) to very large ($1•10^{-1}$) one [1]. This wide range of deformation is characteristic of deformation of rock mass in the area of underground facilities. The knowledge of these characteristics will allow to improve the accuracy of prediction of deformations of the earth's surface [2, 3]. Another important aspect that should be taken into account when considering layered media is their structure which has a significant influence on their mechanical behavior, forming anisotropy of strength and deformation characteristics. For clay rocks, the anisotropy of mechanical properties is mainly related to their structure. The analysis of scientific works studies [4-6] has allowed to establish the following aspects of anisotropic clay environments.

Quantitative assessment of the anisotropy is based on the coefficient of anisotropy, which is the ratio of the maximum to the minimum values obtained by measuring a certain parameter of the mechanical behavior of rocks in different directions. There are two types of anisotropy. In the first case the assessment is based on mean anisotropy, due to the nature of the formation of soil mass and so-called natural (inherent) anisotropy. In the second case, the scientists consider the change of the natural anisotropy and the development of so-called deformation or induced anisotropy associated with irreversible deformations. This latter is mainly due to the change of the stress state and deformation of rocks that leads to some change in its structure. Such structure of rocks formed by with an increase of the gravitational load, which is expressed in the change of its stress state and restructuring of initially isotropic structures into the anisotropic one.

The anisotropy of mechanical properties has been studied in detail by such scientists as J. E. Rogatkin, Lukinskaya I. G., Lushnikov, V. V. Fursa, V. M. [7, 8]. An example of layered soils is sediments, particularly the clay belt of the Gulf of Finland in St. Petersburg. In the works of M. V. Fursa, A. A. Hagan [7-10], R. E. Dashko [11], V. D. Lomtadze [12], T. G. Poliscuka [13] the deformation and strength anisotropy were reported for these soils. The results of the compression test of lake-ice deposits on concrete base of complex flood-protecting structures of St. Petersburg revealed very noticeable deformation and strength anisotropy [12]. The most common example of natural anisotropy in soils of high to medium lithification is a transversal isotropy [14-16]. In [17-25] it was also noted that clay soils exhibited deformation anisotropy. The results of these works are summarized in table 1.

Table 1 Deformation anisotropy of rocks

Authors, year	Type of clay	Deformation level, %	$E_{\parallel}/E_{\perp}$
Ward, 1959	London clay	0,2; 0,6	1,4; 2,4
Kirkpatric and Rennie, 1972	Reduced caoline	Significant	0,6-0,84
Franklin and Mattson, 1972	“ “	Very small	1,8-4,0
Atkinson, 1975	London clay	1	2
Lo and others, 1977	Leda clay	0,4-0,6	0,55
Saada and others, 1978	Reduced caoline	0,001; 0,007	1,25; 1,35
Yong and Silvestri, 1979	Shampleine clay	0,5-1,0	0,62
Graham and Holsby, 1983	Vinningpeg clay	3	1,78
Kirkgard and Lane, 1991	Mud, San Francisco	Significant	1,2-1,8

Comprehensive study of the mechanical behaviour of lithicated clay rocks presented in works by R. E. Dashko, which are summarized in the monograph [26]. R. E. Dashko noted that the presence of macro-and microcracking is a crucial issue in the study of strength, stability and deformability of lithificated clays.

The results of laboratory experiments of argillit-like clay revealed that its behavior is very complex and depends on the achieved stresses and strains and the direction of load transfer. Thus, the model of the behavior of geomaterial, which allows to describe the deformation of argillit-like clay, needs to take into account the following main features of its behavior:

- change of strain properties in the range of very small - small deformations;
- the dependence of deformation properties from the achieved values of average stresses;
- natural anisotropy of the deformation properties at all stages of deformation;
- natural and induced anisotropy of strength properties;
- Non-linear shape of strength envelope.

In general it can be noted that the results presented in scientific works of investigation of the mechanical characteristics of the Proterozoic clays are very representative [1, 4-26]. At the same time, the influence of mean stresses on its deformation indicators is hardly considered and not concluded at all. Additionally, there is no completed study of the mechanical behavior of Proterozoic clay in the range from very small to small deformations. Thus, it is necessary to conduct additional studies aimed at obtaining new knowledge about the mechanical behavior of Proterozoic clay in regimes that have not been studied yet and it is required to perform a reliable prediction of how the deformations in the area of underground facilities and the prediction of deformations of the earth's surface.

2. METHODS

2.1. Lab experiments

The investigation of physico-mechanical properties of Proterozoic clays, that was performed by Leningrad laboratory of CNIIS in collaboration with the laboratory of mechanical properties of VNIMI indicates a serious difference in properties along and perpendicular to the bedding of the rocks [27, 28]. It was revealed that the value of tensile strength under uniaxial compression perpendicular to the bedding is about 3.2 – 3.6 MPa, and the value under compression parallel to the bedding is around 1.0 – 2.0 MPa [20]. The modulus of deformation when the load is applied perpendicular to the layers is 270 – 280 MPa and when it is parallel to the bedding, the modulus is around 710 – 770 MPa. The coefficient of transverse deformation is 0.09 – 0.2 [27]. The coefficient of transverse deformation v increasing with the stress level. The value obtained under the loading perpendicular to the bedding is slightly higher compared to the value obtained when the direction is parallel to the bedding (Table 2).

Table 2 The coefficient of transverse deformation [28]

The direction of the active load	The level of the stress, MPa							
	0.75	1.125	1.5	1.87	2.25	2.62	3.01	3.38
Perpendicularly to the bedding of the rocks	-	0.095	0.162	0.183	0.190	0.193	0.201	0.204
Parallel to the bedding of the rocks	0.095	-	0.011	-	0.136	-	-	-

The results of laboratory tests allowed to build the passports of strength for Proterozoic clay in the direction parallel to the bedding (Fig. 1) and perpendicular to the bedding (Fig. 2). With taking into account the incomplete range of laboratory tests on the samples taken from the tunnel of the subway station “Bucharestskaya” in Saint-Petersburg, the diagrams of strength has been developed on the basis of results the triaxial tests.

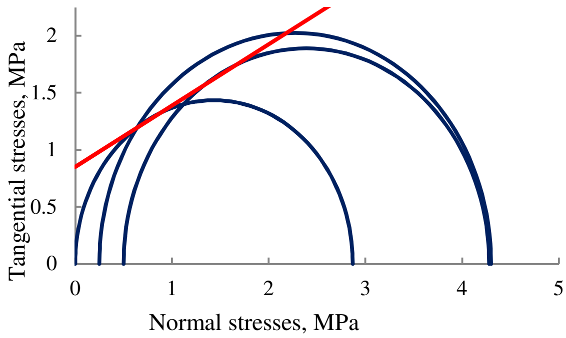

Figure 1 Passport of strength of Proterozoic clays in the direction parallel to the layered structure

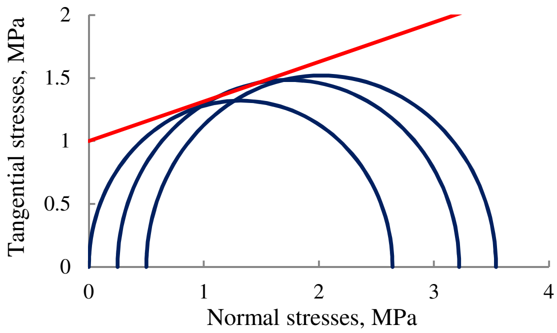

Figure 2 Passport of strength of Proterozoic clays in the direction perpendicular to the layered structure

Hence, the anisotropy of mechanical properties is an inherent property of soils, and their microstructure determines only the degree of influence of this property on the strength and deformability of soils. The work is devoted to study the mechanical behavior of Proterozoic clay like soil, which has a strong anisotropy. In this work we did not consider such an important factor as the structural disturbance of the soil mass, which can make a significant contribution to the change in mechanical properties of soils, and attention is paid only to the study of deformation argilit-like clay soils of undisturbed formation.

2.2. Condition of the transversely isotropic medium

Based on the above experimental data, we must turn to the selection of the conditions of plasticity in plastic deformation of anisotropic rocks. Analysis of experimental data shows that the ultimate strength for uniaxial compression depends on the direction of layers of rocks in relation to the current load:

$$\sigma_{сж} = \sigma_{сж}(\Theta), \quad (1)$$

where Θ is the angle between the bedding of the sample and the direction of action of the load.

The clutch K of rocks is depends on the ultimate strength for uniaxial compression according to the equation:

$$K(\Theta) = (1 - \sin\rho)\sigma_{сж}(\Theta)/(2\cos\rho), \quad (2)$$

where ρ is an angle of internal friction of rock.

We should generalize the most common conditions of plasticity of rocks in an anisotropic medium, given that the tensile strength of them depends on the direction of load application in relation to the bedding of the rocks. In practical calculations the Coulomb condition of plasticity in the form of a sloping linear envelope of circles of main greatest stresses is generally applied. Given the context, (2) we should generalize this condition to anisotropic media, which for plane strain can be written as:

$$(\sigma_\Theta - \sigma_r)^2 + 2\tau_{r\Theta}^2 = \sin^2\rho[\sigma_\Theta + \sigma_r + 2K(\Theta)\mathrm{ctg}\rho]^2, \tag{3}$$

One should consider the angle existing between the principal stresses and the direction of lamination when calculating the plastic deformation of rocks surrounding the tunnels, when the latter is constructed in the massifs with different limits on compression parallel and perpendicular to the bedding. In order to account, it is common to to use the passport of the strength of the tunnel, which takes into account the direction of the principal stresses and the location of the ultimate strength of rocks based on their stratification.

For the approximation of the clutch of rocks near the contour of the tunnel constructed in a layered massif, we will use the following dependency:

$$K(\Theta) = \left[1 + \delta\sum_{n=1}^{m}(a_n \cos n\Theta + b_n \sin n\Theta)\right], \tag{4}$$

where a_n , b_n – approximation coefficients; Θ - angular coordinate; δ – the coefficient characterizing the degree of anisotropy of rocks.

In a first approximation according to (3) it can be limited to two terms

$$K(\Theta) = K(1 + \delta\cos 2\Theta). \tag{5}$$

Let the clutch in rocks under compression perpendicular to the bedding, using $K_\perp$, and in parallel – $K_\parallel$. Then from (4) we have:

$$K(0) = K(1+\delta) = K_\perp,$$
$$K(\pi/2) = K(1-\delta) = K_\parallel, \tag{6}$$

and it leads to the following formula

$$K = (K_\perp + K_\parallel)/2;\ \delta = (K_\perp - K_\parallel)/(K_\perp + K_\parallel). \tag{7}$$

Hence, the K and δ according to (5) have the following physical meaning: the coefficient K characterizes the size of the average clutch of the rock, while δ is the value of the spread induced by anisotropy of the strength characteristics.

2.3. The ultimate state of the soil massif in the area of the tunnel

Let us investigate the influence of anisotropy of the strength characteristics of rocks on the shape and size of the region of plastic deformation around the tunnel. For this investigation, we restrict the tunnel to a circular shape, constructed at a depth H from earth's surface in a plastically anisotropic massif. Let us suppose that an area of limit state is formed near the contour of the tunnel and it fully covers the contour, The solution of this problem will be done in the plane setting, by analyzing the stress field in the plane perpendicular to the axis of the tunnel. Then the problem is simplified to the study of the stress state in an infinite plane with a hole, the contour is given by the boundary conditions σ_r =P, $\tau_{r\Theta}$ =0.. We assume that the plastic anisotropy of rocks is characterized by cohesion, which is described by the dependence (5). Consider the case when the rock in the plastic zone satisfy the condition of plasticity of the pendant.

The solution of the problem will be sought in the form of the decomposition by the small parameter δ

$$\sigma_r = \sum_{n=0}^{\infty} \delta^n \sigma_r^{(n)}; \quad \sigma_\Theta = \sum_{n=0}^{\infty} \delta^n \sigma_\Theta^{(n)}; \quad \tau_{r\Theta} = \sum_{n=0}^{\infty} \delta^n \tau_{r\Theta}^{(n)}, \tag{8}$$

where $\sigma_r^{(n)}, \sigma_\theta^{(n)}, \tau_{r\theta}^{(n)}$–are stress components in the n[th] approximation. By substituting these stress values in (3) and comparing the coefficients of equal powers of δ we derived the plasticity conditions of zero and first approximations:

$$\sigma_\Theta^{(0)} = \left[(1+\sin\rho)\sigma_r^{(0)} + 2K\cos\rho\right]/(1-\sin\rho),$$
$$\sigma_\Theta^{(1)} = \left[(1+\sin\rho)\sigma_r^{(1)} + 2K\cos\rho\cos 2\Theta\right]/(1-\sin\rho). \tag{9}$$

Similarly, the dependencies of the second and subsequent approximations can be found. Let us turn to the determination of the stresses in the zero approximation. For this we are using the first relations of plasticity (9) of plasticity, the equations of equilibrium, the condition of absence of tangential stresses$\tau_{r\theta}^{(n)} = 0$ in the plastic region and the boundary conditions . In the result we obtain:

$$\sigma_r^{(0)} = (P + K\,\mathrm{ctg}\rho)(r/\beta)^\alpha - K\,\mathrm{ctg}\rho; \qquad \tau_{r\Theta}^{(0)} = 0;$$
$$\sigma_\Theta^{(0)} = (1+\sin\rho)(P + K\,\mathrm{ctg}\rho)(r/\beta)^\alpha/(1-\sin\rho) - K\,\mathrm{ctg}\rho, \tag{10}$$

where $r = R/r_s^{(0)}, \beta = R_0/r_s^{(0)}, \alpha = 2\sin\rho/(1-\sin\rho)$; R_0, R are the radius of the underground structures and the radial coordinate, respectively; $r_s^{(0)}$ is the radius of the area of plastic deformations in the zero approximation.

It can be shown that the components of the stresses in the plastic region in the first approximation have the following form:

$$\sigma_r^{(1)} = K\left(r^{\alpha/2-1}/\beta\right)\cos\rho[\cos\chi + (3-2\sin\rho)(\sin\chi)/t]\cos 2\Theta - K\cos\rho\cos 2\Theta;$$
$$\tau_{r\Theta}^{(1)} = K\cos\rho\cos 2\Theta - K\left(r^{\alpha/2-1}/\beta\right)\cos\rho[\cos\chi - (3+2\sin\rho)\sin\chi/(t)]\sin 2\Theta, \tag{11}$$

where $\chi = t(\ln(r/\beta))/(1-\sin\rho)$, $t = \sqrt{3-4\sin^2\rho}$.

Component $\sigma_\Theta^{(1)}$ is determined from the second relation (9). The equation of area of the limit state around the tunnel will be sought in the form of decomposition on small parameter:

$$r_s = r_s^{(0)}\left(1 + \delta r_s^{(1)} + \delta^2 r_s^{(2)} + \ldots\right), \tag{12}$$

where $r_s^{(0)}$, $r_s^{(1)}$, $r_s^{(2)}$ are zero, first, and second approximations.

Let us build the solution in the elastic region. The general form of the elastic solution is determined by the linear combination of solutions from different boundary conditions. The arbitrary constants and the radius of the limit states are determined from the conditions of continuity of stresses at the boundary of elastic and plastic regions. The stress state of rocks in the pristine massif will be assumed equal to:

$$\sigma_y = \gamma H; \quad \sigma_x = \lambda\gamma H; \quad \tau_{xy} = 0, \tag{13}$$

where γH is geostatic vertical pressure of rocks at a depth H from earth's surface; λ – coefficient of lateral pressure. In accordance with the stress (11), the conditions at infinity we write in the form:

$$\left.\begin{matrix}\sigma_r \\ \sigma_\Theta\end{matrix}\right\} = \lambda_1 \gamma H \pm \delta b \gamma H \cos 2\Theta; \qquad \tau_{r\Theta} = \delta b \gamma H \sin 2\Theta, \tag{14}$$

where $\lambda_1 = 0{,}5(1+\lambda)$; $b = (1-\lambda)/(2\delta)$.

In zero approximation the boundary conditions at infinity will as follows:

$$\sigma_r^{(0)} = \sigma_\Theta^{(0)} = \lambda_1 \gamma H; \qquad \tau_{r\Theta}^{(0)} = 0, \tag{15}$$

while in a first approximation it will have the following form:

$$\left.\begin{matrix}\sigma_r^{(1)} \\ \sigma_\Theta^{(1)}\end{matrix}\right\} = \mp b \gamma H \cos 2\Theta; \qquad \tau_{r\Theta}^{(1)} = \delta b \gamma H \sin 2\Theta, \tag{16}$$

In elastic region the stress components in zero approximation will be sought in the form:

$$\left.\begin{matrix}\sigma_r^{(0)} \\ \sigma_\Theta^{(0)}\end{matrix}\right\} = A \mp B/r^2; \qquad \tau_{r\Theta}^{(0)} = 0, \tag{17}$$

Where A, B are arbitrary constants

From the conditions of continuity of breakdown (9) and (21) in the zero approximation, one has the dependence to find the radius of the zone of plastic deformation

$$\left(r_s^{(0)}\right)^\alpha = \frac{(1-\sin\rho)(\lambda_1 \gamma H + K \operatorname{ctg}\rho)}{P + K \operatorname{ctg}\rho} \tag{18}$$

At this leads to the following dependence of the stress components in the elastic zone:

$$\left.\begin{matrix}\sigma_r^{(0)} \\ \sigma_\Theta^{(0)}\end{matrix}\right\} = \lambda_1 \gamma H \mp (\lambda_1 \gamma H + K \operatorname{ctg}\rho)(\sin\rho)/r^2 . \tag{19}$$

Now, we should plot the solution in the elastic zone in the first approximation. From the continuity of strains on the boundary of elastic and plastic medium we can determine the boundary conditions for elastic zone at $r = 1$

$$\sigma_r^{(1)} = K\cos\rho \left(r_s^{(0)}\right)^{1-\alpha/2} \left[\cos\chi_0 + (3-2\sin\rho)(\sin\chi_0)/t\,\right]\cos 2\Theta - K\cos\rho\cos 2\Theta;$$

$$\tau_{r\Theta}^{(1)} = K\cos\rho\sin 2\Theta - K\cos\rho \left(r_s^{(0)}\right)^{1-\alpha/2} \left[\cos\chi_0 - (3+2\sin\rho)\sin\chi_0/(t)\right]\sin 2\Theta, \tag{20}$$

where $\chi_0 = \left(t\ln\left(1/r_s^{(0)}\right)\right)/(1-\sin\rho)$.

From the matching conditions between the different areas for strain dependence obtained in the first approximation:

$$r_s^{(1)} = \frac{\dfrac{4b\gamma H}{K} - \cos\rho\left\{4 + 2\left(r_s^{(0)}\right)^{\frac{\alpha}{2}-1}\left[\left(\dfrac{\alpha}{2}-1\right)\cos\chi_0 - \dfrac{\sin\chi_0}{t}\dfrac{3+2\sin\rho-4\sin^2\rho}{1-\sin\rho}\right]\right.}{\left(\dfrac{\lambda_1\gamma H}{K} + \operatorname{ctg}\rho\right)\sin\rho + 2\sin\rho\left(\dfrac{P}{K} + \operatorname{ctg}\rho\right)\dfrac{1+\sin\rho}{(1-\sin\rho)^2}\left(r_s^{(0)}\right)^{\alpha-1}}\cos 2\Theta. \tag{21}$$

Then, in the first approximation, the radius of plastic deformation zone around the tunnel can found by the following equation

$$r_s = r_s^{(0)}\left(1 + \delta r_s^{(1)}\right). \tag{22}$$

Note, the solution is reliable only if $r \geq 1$. Similarly, the second and the following approximations can be obtained.

3. RESULTS AND DISCUSSION

3.1. Determination of the zone of limiting state of soil massif around the tunnel

Due to the complexity of elasto-plastic problem in this work along with the analytical method we have used a numerical method. To calculate the stress-strain state in the area of a tunnel with a circular shape in the representation of the massif as an isotropic medium accounting the anisotropy strength, we have used the Coulomb condition of plasticity in the software package Abaqus Sumulia. The problem is solved for the plane configuration statement.

Modeling of the soil mass was performed using identical boundary conditions and by varying the selected parameter, namely the lateral pressure coefficient. The tunnel has a circular cross section with diameters of 6 meters and it is at a depth of 50 m. The deformation modulus of the clay in the numerical solution of the problem is taken equal to 500 MPa, Poisson's ratio of 0.24, the angle of internal friction of 28 degrees, the adhesion when tested parallel to the arrangement of layers 0,735 MPa perpendicular to the bedding, – 1 MPa. Modeling of the soil massif is made with laminations arranged in a horizontal direction.

In our modeling we are calculating the stress-strain state in the soil mass around the tunnel, the distribution of shear stresses and the magnitude of plastic deformations into the massif from the contour of the tunnel and by contour of the tunnel. The results of calculation of the limit state around the tunnel of circular cross-section shown in Fig. 3, which displayed diagrams of the distribution of plastic deformations in the area of the mining tunnel at a depth of 50 m for different ratios of principal stresses. Fig. 3a shows the area of the limit state around a tunnel under hydrostatic stress field. In this case, it has an oval shape and it is elongated in the vertical direction. At the lateral pressure coefficient equal to 0.75 (Fig. 3b) the oval area of the limiting state is elongating relatively to the horizontal axis. The analytical solution of the problem by using the method of small parameter leads to the resulting distribution zone of plastic deformations around a tunnel under hydrostatic stress distribution around the tunnel. Thus, when comparing the results of analytical and numerical solutions, the qualitative and quantitative convergence has been accomplished. As a result of the analytical solution by using the dependences (17) and (21) the calculated radius of the limit state around the tunnel in a hydrostatic stress field is of 0.64 and 0.55 m in the roof and in the side of the tunnel, respectively. The difference in the results obtained using these approaches is about 2-3% and it indicates a high reproducibility of the results.

a b

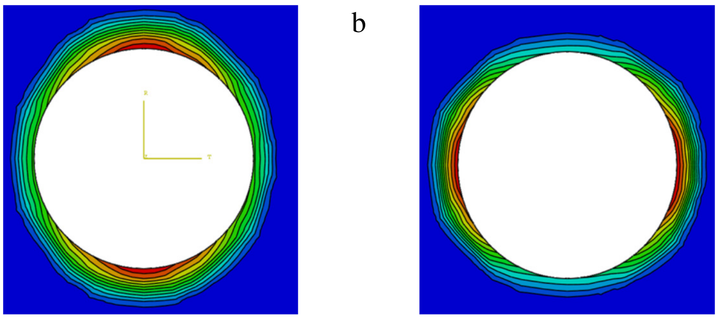

Figure 3 Diagrams of the distribution of the limit state zones in the soil mass around the tunnel with a diameter of 6 m: a - coefficient of lateral pressure 1, b - coefficient of lateral pressure of 0.75.

The analysis of the results of the numerical solution led to following statements: the area of the limit state around the tunnel depends on the direction of the applied load relative to the bedding of the clay. Anisotropy of strength and deformation characteristics, formed by lithification of pelitic rocks results in a nonuniform distribution of stresses and strains in the soil mass (Fig. 4) leading to the occurrence of zones with high plastic deformation. If one increases the vertical component of the stresses it leads to the growth of zones of plastic deformation in the sides of the tunnel.

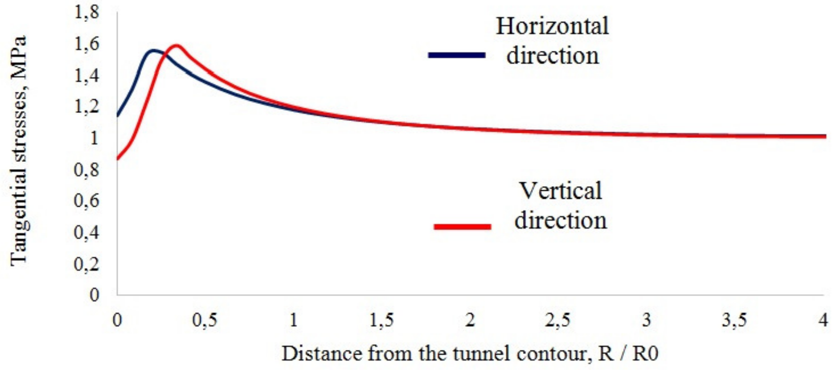

Figure 4 Tangential stresses, 1 MPa, hydrostatic stress field, the depth of the tunnel 50 m

With a decrease of the vertical / horizontal stress ratio to the higher values, the zone of concentrated plastic deformations is moving from the arch of the tunnel into its sides. The Fig. 5 shows the dependence of the propagation of plastic deformations into the massif at the vertical and horizontal directions around the tunnel located in a hydrostatic field stress of 1 MPa in the vertical direction from the roof of the tunnel the magnitude of plastic deformation is higher than in the horizontal direction at 1.37 times. Fig. 7 shows similar characteristics of stress-strain state around the tunnel when the ratio of horizontal stresses to the vertical equals to 0.7. In result of redistribution of stresses, a radical change in dependency of plastic deformation occurs: the plastic deformations on the contour of the tunnel in the horizontal direction exceeds plastic deformations in the vertical direction by a factor of 3. However, this big difference is decreasing with the depth of penetration into the soil till the end of limiting the deformations zone. The above-mentioned mirror difference of character of distribution of plastic deformations can be easily seen in Fig. 7, which presents the dependence of plastic deformations on the contour of the tunnel under hydrostatic stress field and the lateral pressure coefficient of 0.75. The x-axis is located the angular coordinates, where 0 and 360 degrees are the coordinate of the arch of the tunnel.

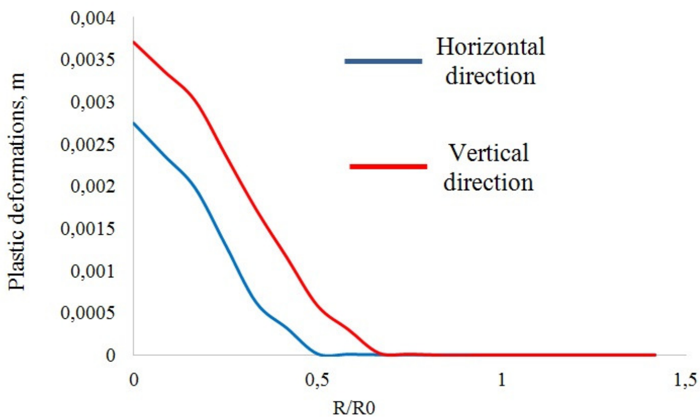

Figure 5 The quantity of plastic deformation as a function of the distance into the massif from the contour of the tunnel at a fixed lateral pressure coefficient equal to 1 and the depth of the tunnel of 50 m

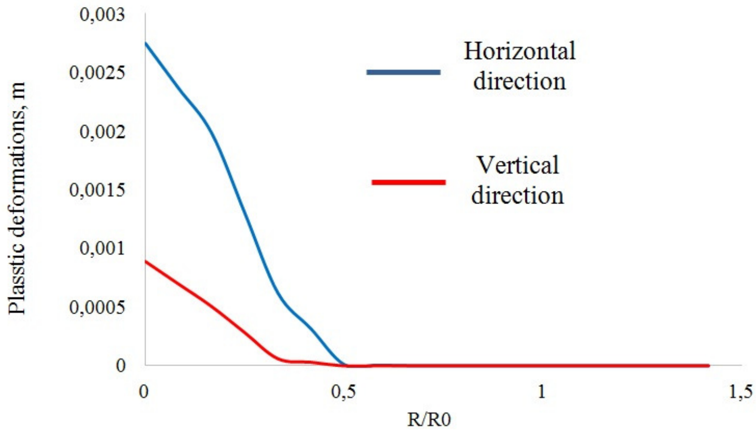

Figure 6 The quantity of plastic deformation as a function of the distance into the massif from the contour of the tunnel at a fixed lateral pressure coefficient equal to 0.75 and the depth of the tunnel of 50 m

When modeling the soil massif with identical boundary conditions and using a constant value of the adhesion of clay 1 MPa, the zone of plastic deformation is not formed if the value of the hydrostatic stress field equals to 1 MPa,. The creation of the zone occurs at a hydrostatic field stress of 2 MPa. For comparison of approaches for estimating the stress-strain state in soil with taking into account the anisotropy of the strength characteristics and without considering them, the results are shown in Fig. 8. The magnitude of plastic deformations on the contour of the tunnel taking into account the anisotropy of the strength characteristics is higher by 4.5 times (at least) compared to the value obtained in the calculation result without taking into account the anisotropy, where the clutch Proterozoic clay is taken as a constant value.

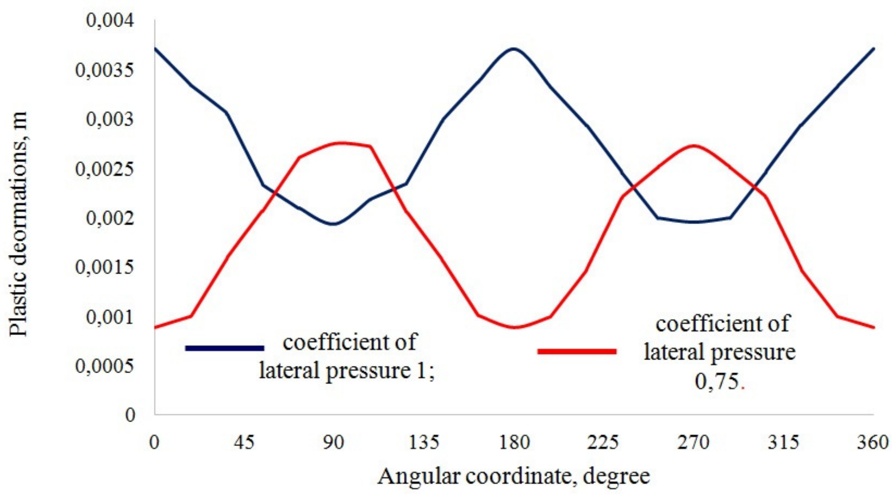

Figure 7 The quantity of plastic deformation as a function of the angular coordinate at different lateral pressure coefficient and at a fixed depth of the tunnel of 50 m

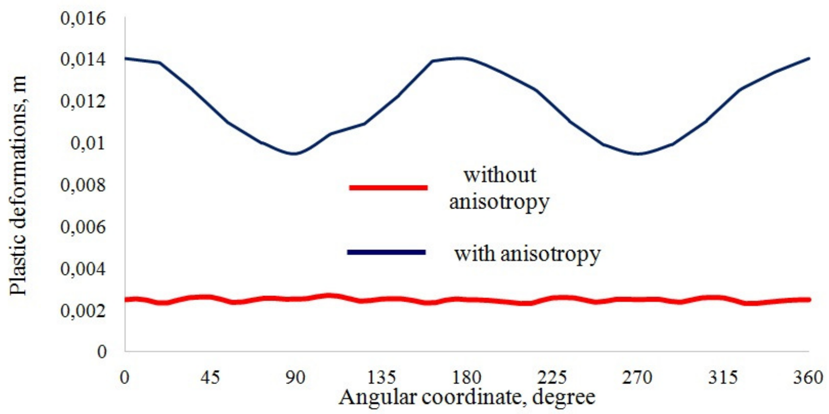

Figure 8 The quantity of plastic deformation as a function of the angular coordinate with taking into account (or nor) of the anisotropy of strength characteristics the distance into the massif from the contour of the tunnel at a fixed lateral pressure coefficient equal to 2 MPa

4. CONCLUSION

In general, it should be noted that the layered structure of Proterozoic clay has an impact on its all mechanical characteristics. The highest anisotropy as the strength and deformation characteristics is evident for small values of minimum principal stresses but with increasing stress values the impact of the structure is markedly reduced. The coefficient of anisotropy of the deformation characteristics is higher than the one of the strength, i.e., the influence of Proterozoic structures of the clay in the stage before the limit of deformation is higher than the its influence in the limit or beyond limit stage.

The anisotropy of the mechanical characteristics of the clay leads to the appearance in the soils around the constructed underground facilities uneven nature of distribution of stresses and irreversible deformations. The appearance of uneven areas limit state contributes to the formation of fall-outs in the construction of underground structures. It is revealed that the zone of concentration of stresses and strains depends on the natural stress field in the soil mass and slope stratification of the soil massif to the horizontal plane. The same account stratification modifies the representation of the distribution of the stress-strain state around the tunnel.

The reported solution allows to reveal the featured properties of geomechanical processes in the area of underground constructions, the sizes of the limit state zones and the coefficients of stress concentrations. The obtained results demonstrate both scientific and practical importance for the selection of design parameters of excavation and support. As a conclusion it should be noted that when using the approach of continuum medium mechanics in numerical simulation without taking into account the anisotropy of the strength of the soil mass, there is no convergence of the results with analytical and numerical solutions. In contrast, when the anisotropy of strength is taken into account a goog convergence of the results has been obtained. The solution obtained using the analytical dependences has a quantitative difference with the results of the calculation of the limit state zone around the tunnel by numerical simulation (the model is taking into account the strength anisotropy of the soil mass), is not more than 2-3 %, depending on the considered part of the contour of the tunnel. This indicates a high reliability of the results.

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