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ASSESSMENT OF THE STRESS-STRAIN STATES OF THE HOST MASS AND LINING OF THE ROGUN HYDROELECTRIC POWER PLANT ROOM

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In this study, the numerical modeling of the stress-strain state of the host rock mass around underground hydraulic structures located at great depths in a mountain massif is carried out. According to the results, the regularities of changes in the stress-strain state of the host mass and the lining of the underground structure during stage-wise construction were established. Moreover, the considered approach enables to estimate the bearing capacity of the lining and the residual life of the structure. This is done using the results of the field data on the change in the convergence of the underground structure with the lining and the numerical analysis of the structural elements stress state.

Keywords: Rogun hydroelectric power plant; host mass; stress-strain state.

In hydraulic engineering, large-scale underground chambers are generally required for the placement of technological objects designed for a long period of operation. The placement of underground structures in a rock mass leads to the deformation and destruction of rocks near the outcrops. The stability of underground structures is maintained by lining (barring), which can prevent the deformation and displacement of rocks on outcrops, thereby contributing to the safe operation of the structures.

To meet the requirements of the stable state of complex hydraulic structures in underground conditions, assessing the stress-strain states (SSSs) of the lining elements and rock masses when placing the structures is essential.

The numerical modeling of the SSS of the host mass around the turbine room of the Rogun hydroelectric power plant is carried out. The results analysis indicates the formation of zones of inelastic deformations near the lateral surfaces of the host mass; these zones are deep into the mass even at the initial stage of opening the turbine room cross section, to a depth of 1 – 2.5 m in hard rocks and up to 5 – 6 m in disturbed rock masses [1]. The bearing capacity of the lining (i.e., its ability to prevent the collapse of rocks) should be preserved not only at the structure construction stage but throughout the operation of the structure. Considering this, the change in the bearing capacity of the lining over time needs to be analyzed, which is crucial in assessing the residual life of the structure.

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The most complete idea of the interaction of the lining with a rock mass is reflected in [2] and has been presented in several other sources [3 – 8]. Generally, the host mass and the lining material are represented by elastic isotropic materials, and the magnitudes of the stresses and deformations arising at the contact of the lining and the host mass depend on the initial stress state of the rocks, the physical and mechanical properties of the rock mass and the lining, and the technology of the lining construction [5, 8].

Most works assume that the opening of the chamber cross section and its barring are implemented in one step. Meanwhile, in [8], researchers showed that the schemes for opening the cross section of the underground structure and erecting the lining in it significantly affect the SSS formation of the entire structure and the individual structural elements.

The current research aims to analyze the interaction between the lining and the host mass, the procedure and the possibility of assessing the lining bearing capacity, and the residual life of construction considering the conditions of the Rogun hydroelectric power plant underground structures.

The Stress State of the Lining Elements and the Host Mass During the Stage-Wise Opening of the Turbine Room Cross Section

We present the analysis results of the SSSs of the host mass and lining elements of the Rogun hydroelectric power plant turbine room, located in the rock mass at a depth of 400 m. Due to the lack of an exact scheme for opening the cross section of the turbine room (length 220 m, width 22 m,

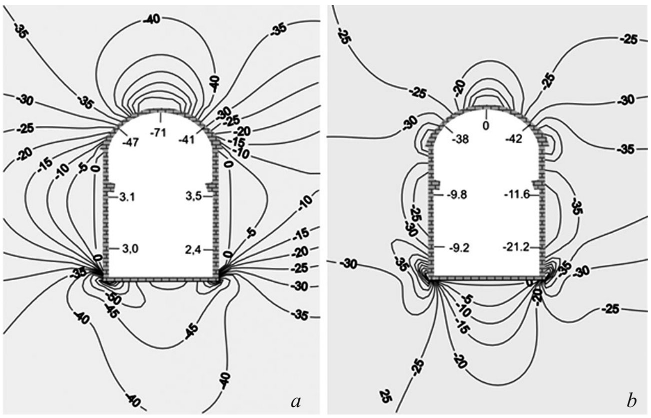

Fig. 1. Horizontal σ_x (*a*) and vertical σ_y (*b*) components of stresses, MPa, host mass, and lining after the first stage of opening the turbine room cross section.

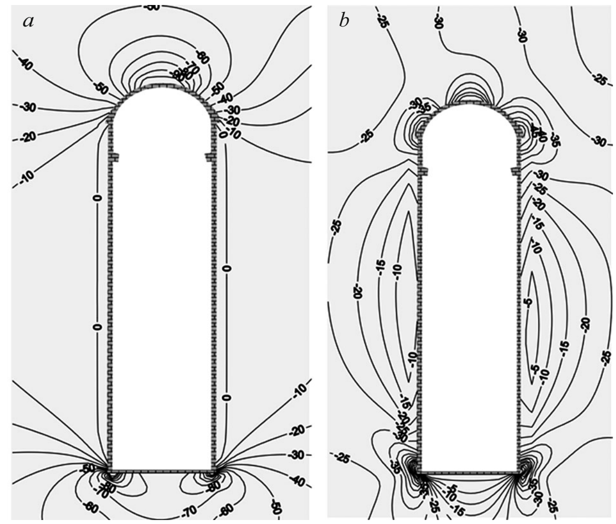

Fig. 2. Horizontal σ_x (*a*) and vertical σ_y (*b*) components of stresses, MPa, enclosing array, and lining after full closure of the turbine room cross section.

maximum height 78 m) and its lining, the section is assumed to be opened in three stages. At the first stage, the arch is formed, and 1/3 of the cross section opens; at the second stage, 2/3 of the cross section opens; and at the third stage, the section is fully opened [1]. Each stage ends with the construction of the lining. When performing the calculations, the presence of the transformer room was considered, since its placement at a distance of 30 m from the turbine room has a significant impact on the SSS of its structural elements. The cross section of the transformer room and the room lining are assumed to be fully formed.

The turbine room is located in a rock mass composed of alternating sandstones and siltstones of the Obigarm suites of the Lower Cretaceous (K1, L1, and K1ob). According to [9], the rock mass consists of layers inclined at an angle of 70 – 75° to the horizontal. Sandstones and siltstones are strong rocks with a high ultimate resistance for uniaxial compressions (where the σ_{com} is 100 – 200 MPa for sandstone and 60 – 80 MPa for siltstone) [10].

In the calculation model of a linearly deformable medium, the following are assumed: the host layered rock mass is quasi-isotropic, slippage is absent at the boundaries of the layers, and there is rigid adhesion [1]. Moreover, the lining is assumed to be in full contact with the host mass along the entire external contour (Fig. 1) and is deformed together with the host mass, i.e., on the contact line; the continuity condition of the vectors of total stresses and displacements is fulfilled.

In previous studies, the problems have been solved using the boundary integral equation method for piecewise-homogeneous media [11, 12]. The ratio of the geometrical dimensions of the turbine room enables to solve the problem of the host mass SSS in a plane arrangement (under conditions of plane deformation).

In [10], the elastic, deformation, and strength properties of the host mass were studied. According to the results, for numerical calculations of the SSS of the elements of hydroelectric power stations underground structures, the following are recommended: a Poisson's ratio of 0.26, deformation modulus of 3.5 – 4.5 × 10^4 MPa, array adhesion of 0.41 – 0.7 MPa, and angle of internal friction of 35°.

Due to the lack of information on the parameters and structure of the lining (exact geometric dimensions and the physical and mechanical properties of the reinforcing materials), the thickness along the entire surface of the underground structure was taken as 1 m; concrete was used as the lining material, with Poisson's ratio $\nu = 0.25$; and deformation modulus $E = 3 \times 10^4$ MPa. The parameters of the natural field of stresses in [9] were adopted: $\sigma_x^0 = -35$ MPa and $\sigma_y^0 = -26$ MPa. Thus, the model problem was solved, and its results are required to clarify the initial information.

The calculation results are analyzed according to the numerical values of the tensor components of the full stress (σ_x, σ_y, σ_{xy}) in areas S_0 (undisturbed rock mass) and S_1 (area of concrete lining), which can be represented as equations [2]:

$$\sigma_x = \sigma_x^{(0)} + \sigma_x^{(1)};$$

$$\sigma_y = \sigma_y^{(0)} + \sigma_y^{(1)};$$

$$\sigma_{xy} = \sigma_{xy}^{(0)} + \sigma_{xy}^{(1)},$$

where $\sigma_x^{(0)}$, $\sigma_y^{(0)}$, and $\sigma_{xy}^{(0)}$ are components of the initial stresses of the undisturbed rock mass; $\sigma_x^{(1)}$, $\sigma_y^{(1)}$, and $\sigma_{xy}^{(1)}$ are components of the additional stresses caused by the creation of a reinforced room in the turbine room.

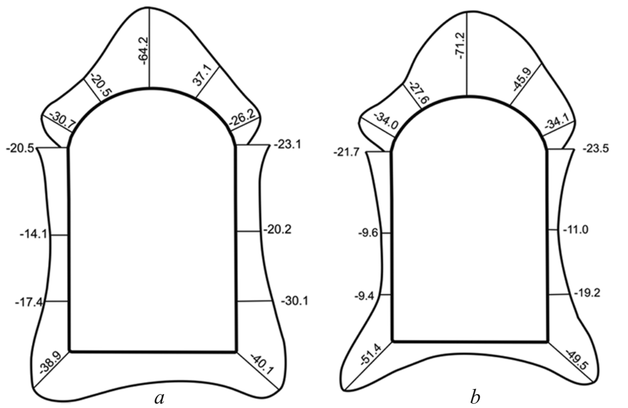

Fig. 3. Stress diagrams of normal tangential stresses σ_θ, MPa, on the outer (*a*) and inner (*b*) surfaces of the lining at the initial stage of opening the turbine room cross section.

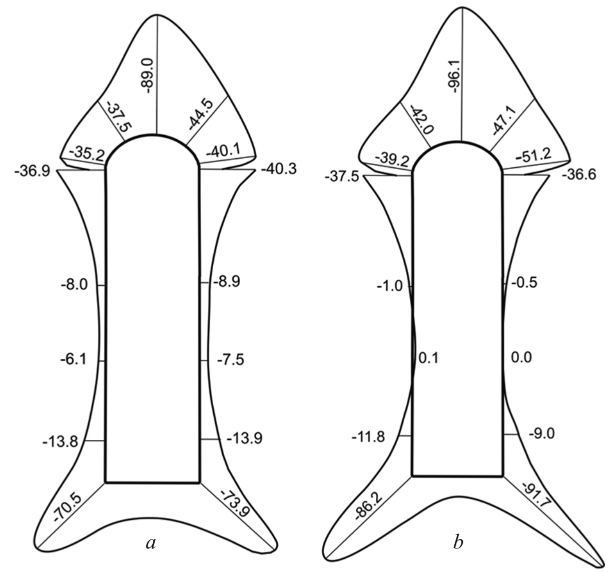

Fig. 4. Stress diagrams of normal tangential stresses σ_θ, MPa, on the outer (*a*) and inner (*b*) surfaces of the lining after completely opening the turbine room cross section.

For the convenience of analyzing the SSS of the lining, the calculation results are also presented in the polar coordinate system by the normal tangential stresses σ_θ.

The results presented in Figs. 1 and 2 show the nature of the formation of the SSSs of the host mass around the turbine room and the lining. The values of the stress components σ_x, σ_y on the lining surface are shown along its inner contour.

In the roof and soil of the host mass around the turbine room, concentration zones σ_x and σ_y are formed. At the first stage of the opening of the turbine room cross section in these zones, the stress is comparable with the compressive strength of the rocks. However, at the final stage, σ_x and σ_y exceed σ_{com} by 30 – 50% (Figs. 1 and 2). The horizontal stresses in the chamber arch and the vertical stresses in the chamber sides determine the high stresses in the roof lining (up to 100 MPa at the final stage of construction). The lateral surfaces of the host mass around the turbine room are unloaded from the stresses of the intact array. Moreover, horizontal tensile stresses act in the lining in the middle part of the lateral surfaces at the final stage of the turbine room construction (Fig. 2*a*). The asymmetry of the SSSs of the host mass and lining in the left and right sides of the chamber is determined by the influence of the transformer room, which is considered in the calculations.

Figures 3 and 4 show the stress diagrams of the tangential normal stresses in the structural elements of the lining after the first stage of the structure cross section opening and at the final stage of the structure construction.

As in the host mass, the largest compressive stress σ_θ is in the roof of the lining; moreover, the compressive stress σ_θ on the inner surface of the lining in the arch is greater than that at the external surface, both at the initial stage of the turbine room cross section opening (Fig. 3) and the final construction stages (Fig. 4). Since the lining concrete is assumed to be unreinforced, the maximum values of the compressive stress σ_θ significantly exceed the concrete resistance to compression. The lateral internal surfaces of the lining are unloaded from the action of σ_θ; when the cross section is fully opened, they become tensile (Fig. 4*b*).

The elastic convergence of the side surfaces of the host mass around the turbine room with a width of 22 m was 543 mm [1, 9]; i.e., 24.6 mm per 1 m; 412 mm with 20 m. With a lined turbine room width of 20 m, the elastic convergence was 448 mm.

Assessment of the Residual Life of the Elements of a Lined Underground Structure

The SSS observations of the underground structure elements indicate a continuing development of stress and strain in their lining over time. The deformations development in the lining could be because of the manifestation of the rheological properties of rocks and the lining material. Regarding this, in many problems on the mechanics of underground structures, methods of variable modules [3, 5, 13] are used, which involve the use of time functions for E and ν. Regarding the lining in the calculations, the creep of the concrete can be considered using the temporary elastic modulus:

$$E_c^t = \frac{E_c^0}{1+\Phi_c^n}, \tag{1}$$

where E_c^0 is the initial modulus of the elasticity of the concrete, and Φ_c^n is the creep coefficient of the concrete [4], corresponding to its class and the relative humidity of the environment.

To consider the rheological properties of the host mass of rocks, the method of variable modules for the deformation characteristics of a rock mass according to linear hereditary creep is used [5, 13]:

$$E^t_{\mathrm{m}} = \frac{E}{1+\Phi(t)};\ \nu^t_{\mathrm{m}} = 0.5 - \frac{0.5-\nu}{1+\Phi(t)}, \tag{2}$$

where $\Phi(t) = \dfrac{\delta t^{1-\alpha}}{1-\alpha}$ is the creep core, and α and δ are creep parameters obtained experimentally.

In the absence of information on the rheological properties of the rocks, structural stresses at a given time can be determined based on information on the measured convergence of the lining by solving the inverse problem [5]. In this case, at the first stage, the deformation modulus of the host mass is taken to be equal to the initial (undisturbed array) E_0, and the lining deformation modulus is calculated using Eq. (1). The stresses and convergence calculated while considering these values are $\sigma^{(n)}$ and $U^{(n)}$, respectively. Next, we find E_1 using Eq. (2), corresponding to a given time, which we consider to calculate $\sigma^{(t)}$ and $U^{(t)}$. Then, we obtain the strain in the lining at a given time using the equation [4]:

$$\sigma = \sigma^1 + (\sigma^{(t)} - \sigma^{(n)}), \tag{3}$$

where σ^1 are stresses calculated without considering the rheological properties of the host mass and concrete. The corresponding convergence values are

$$U = U^1 + (U^{(t)} - U^{(n)}), \tag{4}$$

where U^1 is the convergence calculated without considering the rheological properties of the host mass and concrete.

The stresses obtained according to Eq. (3) enable to estimate the load-bearing capacity of the lining, and Eq. (4) enables to evaluate the change in the convergence over time.

As an example of the application of the technique [5], we evaluate the stress state of the concrete lining of the turbine room after 20 years of operation assuming that at time t_0 the SSS of the structure corresponds to the calculation results obtained above.

We suppose that in accordance with the field data, the graph of convergence changes represents a straight line with increments of 0.2 mm per year; that is, for over 20 years of the facility operation, the convergence increment will be 4 mm (excluding the lining deformation in the initial period). Having performed the corresponding calculations, by the selection method, we determine the modulus of deformation of the medium for a convergence of 1 mm: $E_t = 5 \times 10^3$ MPa. The corresponding value of the lining deformation modulus for calculating stresses in 20 years will be 10,000 MPa ($3 \times 10^4 - 4 \times 5 \times 10^3$ MPa); the modulus of deformation of the concrete brand used in the SSS calculations (see source information) is 3×10^4 MPa; the increment of convergence is 4; and $E_t = 5 \times 10^3$ MPa.

Stress diagrams of stresses σ_θ in the lining of an underground structure (turbine room) after 20 years of operation

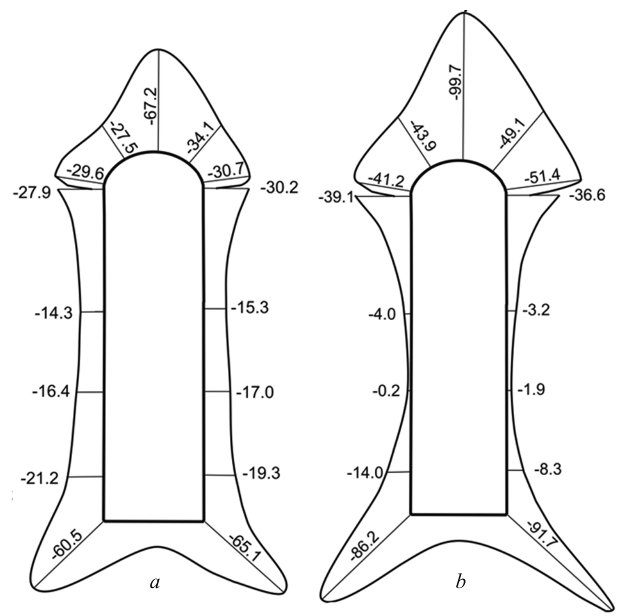

Fig. 5. Stress diagrams of normal tangential stresses σ_θ, MPa, on the outer (*a*) and inner (*b*) surfaces of the lining 20 years after the turbine room operation.

are presented in Fig. 5. The calculation results indicate that the compressive stress σ_θ on contact with the rocks (the lining outer surface) changes more significantly (up to 40%) than that on the inner surface (Figs. 4*a* and 5*a*), whereby the increase in the maximum compressive stress σ_θ does not exceed 8 – 10%. Along the lateral internal surfaces, tensile stresses become compressive (Figs. 4*b* and 5*b*).

The values of the stress distribution in the lining, considering the law of its linear distribution over the lining thickness, determine the forces acting in a dangerous normal section; the compressive stresses exceed the calculated concrete resistance: the moment of resistance (M, MN · m) and the longitudinal force (N, MN), which is compared with the ultimate longitudinal force calculated by SNiP [4] N_{ult}. If $N < N_{\mathrm{ult}}$, then the concrete strength in a dangerous section will be ensured.

By knowing the ultimate value of the convergence of Uult and the increment of the measured convergence values for the year, determined by the method proposed in [5], the period of trouble-free operation of the underground structures can be calculated.

CONCLUSIONS

1. In this study, the particularities of the SSS distribution in the host mass and the lining of the turbine room for the conditions of the Rogun hydroelectric power plant underground structures were established, assuming unreinforced concrete as the lining material, and the model problem was solved.

It is established that horizontal stresses in the host mass in the chamber arch and the vertical stresses in the chamber sides result in high stresses in the arch lining (~100 MPa). The lateral surfaces of the host mass around the turbine room are unloaded from the stresses of the intact array, and horizontal tensile stresses act in the lining in the middle of the lateral surfaces at the final stage of the turbine room construction.

2. A procedure is proposed for assessing the residual life of the structural elements of the Rogun hydroelectric power plant lined turbine room.

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