Specific of stress behavior in enclosing rock mass around underground structures during construction of waterworks

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Abstract. By the numerical stress–strain analysis using the method of boundary integral equations and the model of linearly deformable quasi-isotropic model of a layered rock mass has found the nature, mechanisms and specifics of stress state distribution in enclosing rock mass around the powerhouse hall and transformer room of the Rogunskaya Hydroelectric Plant. The influence of stage-wise face tunneling of the powerhouse hall on the elastic convergence between the bottom and roof of the hall as well as on the inelastic deformation of adjacent rock mass is assessed.

1. Introduction
One of the major influences on stability of underground structure of waterworks is exerted by the sequence of mined-out void formation, which governs the stress–strain behavior in enclosing rock mass. The role of the stress state of rock mass is high when underground structures are arranged at great depths where stress are comparable with the strength of rocks, or exceed it. Under such conditions, the classification systems [1–3] based on the mechanical properties of rocks may appear inefficient. In this connection, stability of underground structures is widely assessed using numerical and experimental–analytical methods [3–5]. These methods allow tracking the stress–strain behavior of rock mass around various structures by stages of construction. A more reliable estimate and prediction of structural stability is possible from combination of the numerical modeling and in-situ observations over deformation of adjacent rock mass [5].

2. Results of a full-scale experiment
This paper describes the analytical data on variation in the geomechanical behavior in rock mass around the powerhouse hall (length 220 m, width 22 m, height 78 m) in course of the stage-wise face tunneling at the Rogunskaya Hydroelectric Plant. The powerhouse hall is constructed at a depth of 350 m in the left-hand bank of the Vakhsh River [6, 7]. The first stage includes construction of the arch and 1/3 face tunneling of the powerhouse hall, the second stage—2/3 face tunneling and the third stage is the full face opening. It is assumed that the transformer room tunneling, which has influence on the stress–strain behavior of the enclosing rock mass, is finished to the full face.

Rock mass in the hydroelectric plant site are composed of alternating sandstone and siltstone. The bedding is inclined at 70–75 deg relative to the horizon [7]. Sandstone and siltstone are hard rocks having the uniaxial compression strength of 100–200 MPa and 60–80 MPa, respectively [6].

The model of the linearly deformable bedded medium assumes the rock mass quasi-isotropic; the interfaces of the beds are free from sliding and feature stiff cohesion. It is shown in [8] that the
distribution of the shear stresses calculated at the boundary of the powerhouse hall opening under assumption of the quasi-isotropic rock mass agrees with the results obtained using the transversely isotropic model [6].

The problem was solved using the method of boundary integral equation [4, 8]. The ratio of geometrical sizes of the powerhouse hall allows 2D problem solution of the stress–strain behavior of rocks.

For the numerical stress–strain analysis of the structural elements of the powerhouse hall, it is assumed that: elasticity modulus $E=3.5–4.5\cdot10^4$ MPa; Poisson’s ratio $\nu=0.26$; cohesion $C=0.41–0.7$ MPa, and internal friction angle $\varphi=35^\circ$ [7]. The in-situ stress state is accepted as per [6]: $\sigma_0^= -35$ MPa, $\sigma_y^0 = -26$ MPa. The calculation results are interpreted by the numerical values of the stress tensor components ($\sigma_x$, $\sigma_y$, $\tau_{xy}$) and stresses $\sigma_x$ allowing determination of potential post-limit deformation zones in rock mass using the Mohr–Coulomb criterion [3, 4]:

$$\sigma_x = \frac{\sigma_1 - \sigma_3}{2\cos\varphi} + \frac{\sigma_1 + \sigma_3}{2}\tan\varphi,$$

where $\sigma_1 > \sigma_2 > \sigma_3$ are the principal stresses; $\varphi$ is the internal friction angle.

Figure 1 reflects formation of the horizontal $\sigma_x$ (Figure 1a) and vertical stresses $\sigma_y$ (Figure 1b) at the first stage of face tunneling in the powerhouse hall. The table 1 gives the maximum and minimum values of $\sigma_x$ and $\sigma_y$ in the roof, floor and sidewalls of the powerhouse hall at three stages of its construction. The nature of the elastic displacements at the boundary of the full face opening is depicted in Figure 2. Figure 3 shows development of the inelastic strain zones in the enclosing rock mass: black color—in hard rocks; grey color—in weakly jointed rocks.

(a)  
(b)

**Figure 1.** Horizontal $\sigma_x$ (a) and vertical $\sigma_y$ (b) stresses at the first stage of face tunneling in the powerhouse hall

**Table 1.** Stress state of enclosing rock mass during construction of powerhouse hall.

| Stage | Structural element   | $\sigma_x$, MPa | $\sigma_y$, MPa |
|-------|----------------------|-----------------|-----------------|
| I     | Bottom               | -40             | 0               |
|       | Right-hand sidewall  | 0               | -25             |
|       | Arch                 | -88             | 0               |
|       | Left-hand sidewall   | 0               | -27             |
| II    | Bottom               | -62             | 0               |
|       | Right-hand sidewall  | 0               | -16             |
|       | Arch                 | -108            | 0               |
|       | Left-hand sidewall   | 0               | -12             |
| III   | Bottom               | -105            | 0               |
|       | Right-hand sidewall  | 0               | 3,5             |
|       | Arch                 | -126            | 0               |
|       | Left-hand sidewall   | 0               | -5              |
Figure 2. Elastic displacements (mm) in horizontal (a) and vertical (b) directions at the boundary of the full face opening

Figure 3. Inelastic strain zones in the enclosing rock mass: black color—in hard rocks; grey color—in weakly jointed rocks

The presented results imply that:
— the stress state of the enclosing rock mass around the powerhouse hall is characterized by the concentrated compressive horizontal stresses \( \sigma_x \) in the roof and bottom (Figure 2a), as well as by the concentrated vertical stresses \( \sigma_y \) at the corners. At stage I of the face tunneling in the powerhouse hall, the stress concentrations are commensurable with the uniaxial compression strength of rocks (Figure 1); at stage III \( \sigma_x \) and \( \sigma_y \) exceed \( \sigma_{UCS} \) by 30–50% (table 1);
— the sidewalls of the powerhouse hall are free from the in-situ stresses (Figure 1, table 1): in the right-hand sidewall (from the side of the transformer room) at stage III, the vertical stresses change to the tensile stresses (to 3.5 MPa). The compressive stresses on the left-hand sidewall change from -27 (stage I) to -5 MPa at stage III (table 1);
— the elastic convergence of the sidewalls in the powerhouse hall increases in the course of the full face opening from 280 mm to 543 mm (Figure 2), while the bottom–roof convergence is no more than 50 mm;
— the inelastic strain zones at the first stage of the full face opening arise at a distance of 1–2.5 m from the cross-section boundary in hard rocks and at a distance of 5–6 m in jointed rock mass (Figure 3). The further tunneling results in the formation of the inelastic strain zone in the pillar between the powerhouse hall and the transformer room. These strain zones in the upper part of the enclosing rock mass around the powerhouse hall almost merge with the zones of \( \sigma_s \) forming in the lower area of the...
enclosing rock mass around the transformer room. At the sidewalls the inelastic strain zones occur at a distance to 1.5–3 m from the boundary in hard rocks; in the roof in the course of the full face tunneling $\sigma_s$ vary insignificantly.

3. Conclusions

The stress–strain behavior of the enclosing rock mass around the powerhouse hall and transformer room is determined: at the sidewalls of the powerhouse hall, the enclosing rock mass is relaxed from the in-situ stresses. At the final stage of the full face opening, in the concentration zones, the stresses $\sigma_x$ and $\sigma_y$ exceed $\sigma_{UCS}$ by 30–50%. Using the Mohr–Coulomb criterion, the parameters of the inelastic strain zones are estimated at three stages of tunneling in the powerhouse hall. The maximal elastic displacements of the powerhouse hall boundary take place at the sidewalls—(the convergence reaches 543 mm at stage III).

The calculation results using the method of boundary integral equations for piecewise–homogeneous domains prove the method efficiency in the stress–strain analysis of the enclosing rock mass around the underground structures of the Rogunskaya Hydroelectric Plant.

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