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ENGINEERING AND GEOTECHNICAL SURVEYS AT THE SITE OF THE MAIN STRUCTURES OF THE PSKEM HYDROELECTRIC POWER STATION ON THE PSKEM RIVER

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ABSTRACT
This paper presents the results of geomechanical studies of the siltstone massif carried out in the right-bank experimental adit at the site of the Pskemskaya HPP dam. Primary shear experiments were carried out at the following 6 normal pressures on the stamp σ: 0.5 MPa; 1.0 MPa; 1.5 MPa; 2.0 MPa, 2.5 MPa and 3.0 MPa.

Keywords: stamp, strength, ultimate strength, siltstone, shear, shear angle, cracks.

Introduction

The Pskem hydroelectric power station is planned for construction in the Bostanlyk district of the Tashkent region, 120 km northeast of Tashkent, in the middle reaches of the Pskem River (see Fig. 1.4-1). The nearest settlement is the village of Kyrdaptyr, located on the right bank of the Pskem River, 5 km below the target area.

The 73 km long Pskem River is the largest of the unused rivers of Uzbekistan, completely located on the territory of the republic.

The construction site stretches along the river valley from the left-bank tributary of the Ispysay stream upstream to the village of Tepar. The dam section is located 1.2 km above the mouth of the right-bank tributary of the Pskem River - the Oromzadasay stream. The reservoir will spread from the dam 19 km upstream to the village of Pskem.

Engineering-geological characteristics of experimental sites

In accordance with the engineering-geological description of the massif, set forth in the report of JSC "Hydroproject" (Tashkent), the experimental sites are located in rocks of the Neogene age, represented by massive-layered siltstones.

The layered stratum of Neogene rocks, lying monoclinally, structurally forms the northern wing of the Pskem graben-syncline. The rock layers extend at an acute angle to the riverbed and fall on the NW-300-3300 at an angle of 50-55° from the left side of the canyon to the right.

Stratum cracks in the thickness of the Neogene sediments are found everywhere and are recorded mainly with a sharp change in the composition of the rocks.

The documentation of the experimental (sub-stamping) sites included fixing and sketching all cracks with a length of more than 10 cm, measuring their azimuths and angles of incidence, identifying the presence and type of aggregate. Based on the results of the documentation, the crack voidness coefficient (KTP) was calculated for each site.

The documentation of the experimental sites showed that at the base of the dies there are cracks with a width of mainly no more than 0.1-0.2 mm, and only on sites No. 8, No. 11 and No. 12, the width of individual cracks reached 3 mm.

Shear experiments on dies

After performing deformation tests on the same concrete dies, shear experiments were carried out. On each die, 2 primary shear experiments were performed at different normal loads, in which the ultimate shear strength (breakdown) was determined at the concrete-rock contactτ_{etc}.

The first shear experiment stopped with a noticeable deviation of the horizontal displacement of the stamp "u" from the initial linear dependence u = f (τ), which indicated the proximity of the shear strength. At the same time, observations of the vertical displacements of the stamp v in most cases showed its rise. Second The experiment, under a different normal load, continued until both the limit τ_{pr} and the residual τ ost shear strength were reached. The estimation of the τ_{ost} value was also carried out based on the results of repeated shifts along the resulting displacement surface (3 repeated shifts were performed on each die). Thus, 5 shear experiments (tests) were carried out on each stamp - 2 primary and 3 repeated. All experiments were carried out according to a torqueless scheme.

Primary shear experiments were performed at the following 6-normal pressures on the σ die: 0.5 MPa; 1.0 MPa; 1.5 MPa; 2.0 MPa, 2.5 MPa and 3.0 MPa. As noted above, on each stamp, primary experiments were carried out at two fixed values of σ.

Repeated shifts on each die were carried out at 3 normal pressures on the dies: 1.0 MPa, 2.0 MPa and 3.0 MPa.As noted above, the near-surface region of the massif during all shear experiments was in a water-saturated state.

In the photo.1. concrete dies BSh-7 prepared for shear tests are shown.

When performing primary shear experiments, the magnitude of the shear stress τ applied to the die and the horizontal u and vertical v displacements of the die caused by this stress were recorded.

The output data on the results of the experiments were the ultimate shear strength τ pr, the horizontal movement of the die u_{pr}, at which the ultimate strength and residual shear strength τ_{ost} (at the second shift) were achieved.

Attention is drawn to the large value of the shear strength (τ_{pr} = 3.38 MPa) obtained during the second shear on the BSh-8 die at a low value of the normal load σ = 1.0 MPa. Quite clearly noticeable deviation in the value of the ultimate shear strength, obtained on the stamp BSh-8, can be seen in Fig.2.3. This figure shows the values of the shear coefficients K sdv = τ_{sdv} /σ determined by the results of the first and second shifts on all dies. As follows from the figure, the value of the shear coefficient obtained in the experiment under consideration (3.38) is almost 2 times higher than the average value of this coefficient determined by the results of all experiments (1.77). Therefore, this value can be considered a random ejection and it was not taken into account when determining the calculated shear parameters (coefficient of friction and adhesion).

The corresponding Coulomb dependence of the ultimate shear strength on the value of the normal pressure applied to the die is as follows:

$$\tau_{пр} = 1{,}40\ \sigma + 0{,}47,\ МПа$$

To obtain the calculated shear parameters, it is necessary to obtain a soil γ g.>1.25, the value $\gamma_{g.}$ = 1.25 is taken, we obtain the following values of the design characteristics: tgφ p = 1.12 and C_p = 0.38 MPa.

In addition to the reliability factor for the soil γg, the recommendations of SNiP2.02.02-85 and SP 23.13330.2011 [4] provide for the introduction of an additional safety factor for a possible discrepancy between the test conditions and full-scale conditions. Taking this into account, as well as the sufficiently large spread of experimental data obtained, which caused an increased value of the γg coefficient, the following values of the shear parameters can be finally recommended as calculated:

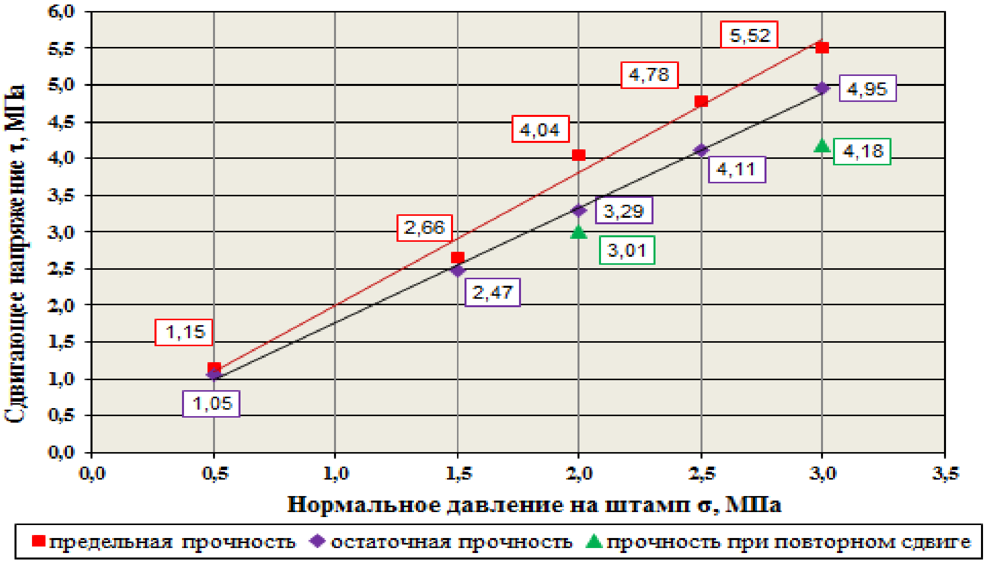
$$tg\varphi\ p = 0.90 \text{ and } C_p = 0.30 \text{ MPa}$$

The value of the residual strength τ ost can be estimated by the results of primary experiments in which the displacements of the dies were recorded after they reached the ultimate strength (second shear experiments on each die). The values of τ ost, determined in the process of primary shear experiments, are shown in Table 2.4. Also, the value of τ_{ost} can be estimated from the data of repeated experiments performed on the same dies. Figure 2.5 shows a comparison of the values of the limit τ_{pr} and residual τ ost of shear strengths obtained in the experiments under consideration (respectively, without taking into account and taking into account the data of the second shift on the BSh-8 die), performed on the same stamps. Fig.2.6 compares the data obtained in the primary experiments (second shift) with the data of all repeated experiments.

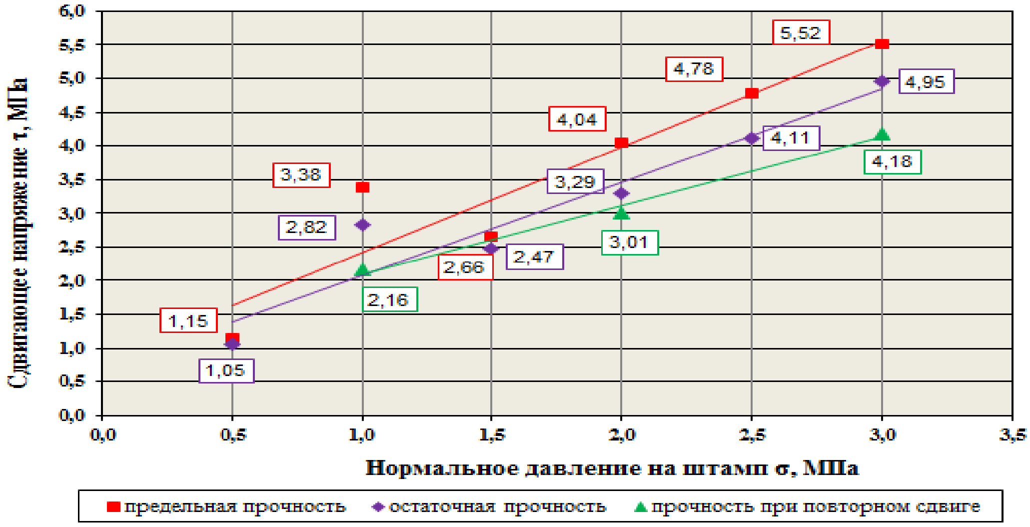
Analysis of the above data shows that the residual shear strength, determined in the process of primary experiments, is less than the ultimate strength by 7.1÷18.6% with an average value of about 12%. According to the results of repeated shifts performed in the range of normal stresses of 1÷3MPa, the residual shear strength was obtained less than the ultimate strength by 14.2% -36.1% with an average value of about 25%.

It should be noted that the need to use residual shear strength in calculations may arise after strong earthquakes or other extreme force effects on the rock massif, which divide the standard values tgφ_n and C_n by the reliability coefficient on the ground γ_g.

In accordance with the recommendations of SP 23.13330.2011 [4], the value of γ g should be determined according to GOST 20522 with a one-sided confidence probability α = 0.95. The calculations showed that the reliability coefficient for the soil in this case is γ_g = 1.33. Taking into account the provision of SP23.13330.2011 (clause 5.16), according to which, with the reliability factor, even small displacements were caused.



(a)



b)

Fig.1. The values of the limiting τ_{pr} and the residual τ_{ost} of the shear strength obtained in the primary shear experiments (second shift) and in repeated experiments without taking into account (a) and taking into account (b) the data on the BSh-8 stamp.

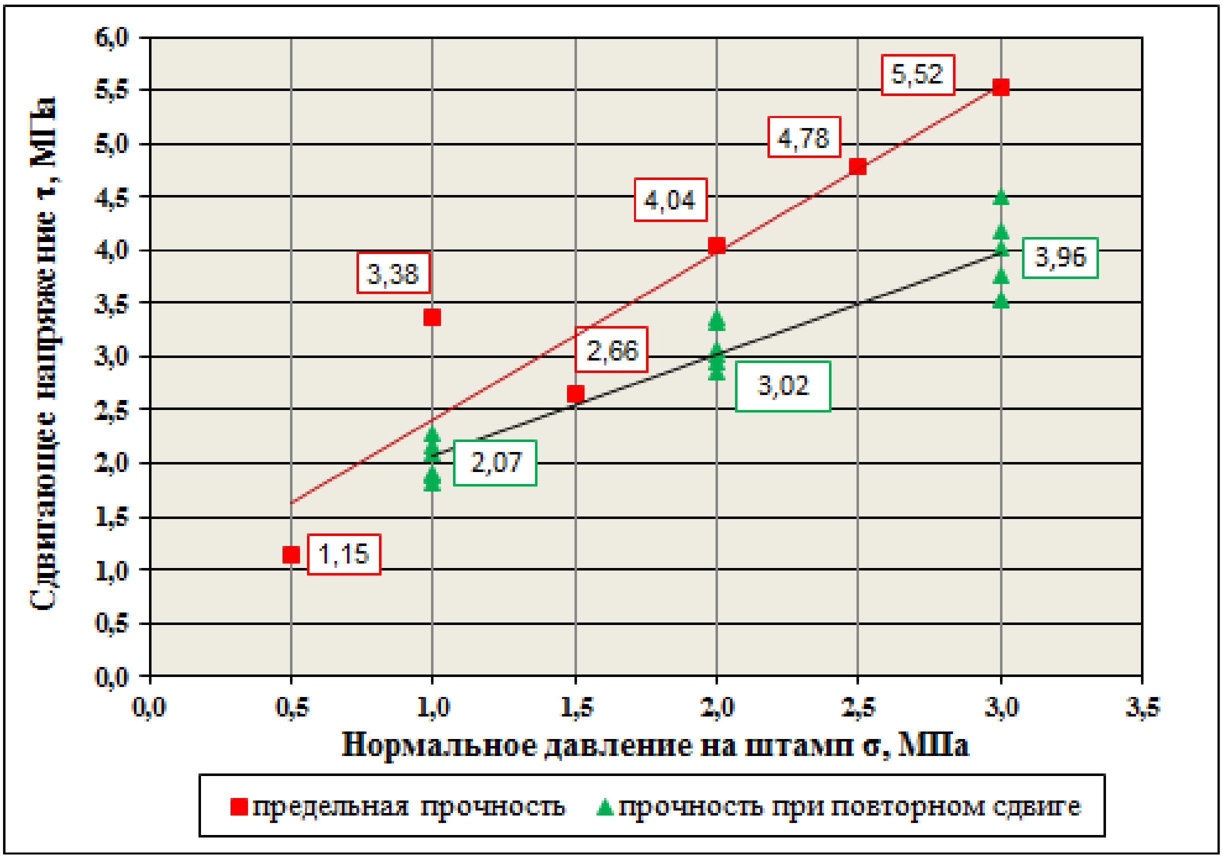


Fig.2. Comparison of the ultimate shear strength τ_{pr} obtained in the primary shear experiments (second shift) with the shear strength obtained in all repeated experiments.

After the shear tests were completed, the dies were overturned and a sketch of the resulting shear surface (chip) was performed. Such a survey made it possible to identify the nature of the destruction that occurred anddetermine the areas through which the shear surface passed. The results of the survey of the base of all tested dies with the percentage determination of the shear areas passing through the massif, contact and concrete.

KEY FINDINGS

1. This report presents the results of geomechanical studies of the siltstone massif performed in the right-bank experimental tunnel at the site of the dam section of the Pskem hydroelectric power station.

2. Field geomechanical studies included stamp experiments to determine the deformation properties of the array and the strength (shear) characteristics of the concrete-rock contact. These studies were carried out at 6 sites located in the experimental tunnel. The experiments were carried out on concrete dies with a base size of 0.7 m x 0.7 m. A total of 6 deformation and 12 shear tests were performed. The dies were shifted in the direction from the upstream to the downstream, which corresponded to the direction of the main loads that would act on the dam.

3. The value of the coefficient of fracture voidness (KTP) of rocks at the experimental sites is insignificant and averages 0.23%. In accordance with the existing classification, the tested rocks were mainly slightly fractured and very slightly fractured.In the engineering and

geological documentation of the experimental sites, no subhorizontal cracks were found, oriented towards the upper or lower headwaters, along which the displacements of the dies could occur.

4. In accordance with the results of deformation tests, the modulus deformations E of the rock massif in the experimental areas vary in the range from 2100 to 9200 MPa with an average value of 5200 MPa.

5. Based on the results of shear experiments, the following design parameters of shear strength along the massif and along the concrete-rock contact can be recommended: tgφ p = 0.90 and C_p = 0.30 MPa.

6. The obtained deformation and strength characteristics of the massif-siltstones significantly exceed the corresponding indicators previously accepted for performing computational studies at the Pskem hydroelectric power station.

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