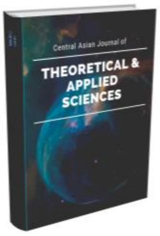

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Article

Field Deformation Tests at the Construction Site of HPP Structures in Uzbekistan

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Abstract: This article presents an analysis of the conducted studies aimed at determining the properties of rock materials used in the construction of the Pskem HPP dam. Engineering-geological surveys were carried out at the dam construction site. Deformation tests were conducted using the static loading method on the rock surface through a rigid concrete stamp.

The fracture void coefficient was determined at the test stamp sites. The deformation modulus (E) was calculated based on the loading branch by varying the normal pressure from zero to the maximum value (σmax) within the studied cycle. The calculated deformation and elasticity moduli were derived from load-unload cycle tests, in which specific normal loads reached levels corresponding to the maximum operational loads on the dam foundation.

Following the deformation tests, shear tests were performed on the same concrete stamps. The article presents the results of geomechanical studies of the aleurolite rock massif, conducted in the right-bank experimental adit at the Pskem HPP dam construction site.

Keywords

Geology, rock, stamps, fractures, stamp, internal friction angle, cohesion, density, tests, Poisson's ratio. Geological Characteristics of the Test Sites

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Introduction

According to engineering-geological surveys carried out by JSC "Hydroproject" (Tashkent) [2], the test sites are located in Neogene-age rock formations, predominantly consisting of massive-layered aleurolites.

The layered thickness of Neogene deposits, lying monoclinically, structurally forms the northern wing of the Pskem graben-syncline. The rock layers stretch at a sharp angle to the riverbed, dipping to the northwest (300°-330°) at angles of 50°-55° from the left canyon wall toward the right.

Fractures within the Neogene deposits do not appear uniformly and are mainly observed where there is a sharp change in lithological composition.

Documentation and Analysis of Test Sites

The documentation of the test (stamp) sites included:

- Recording and sketching all fractures longer than 10 cm;
- Measuring azimuths and dip angles of fractures;
- Identifying the presence and type of fracture fillers.

Based on the documentation results, the fracture void coefficient (FVC) was calculated for each test site.

Results

The summarized data on the number of recorded fractures and KTP values for each test site are presented in Table 1.

From Table 1, it is evident that the fracture void coefficient (FVC) at the test sites is insignificant, ranging from 0.04% to 0.83%, with an average value of 0.23%.

Table 1

Number of Fractures and KTP Values at Test Sites

Site number	7	8	9	10	11	12
Number of cracks	10	10	9	6	14	8
FVC, %	0,05	0,18	0,05	0,04	0,26	0,83

According to the classification of SP 23.13330.2011 [3], rock masses with the specified KTP values are classified as very weakly fractured and weakly fractured masses. The only exception is the foundation of stamp BS-12, which belongs to moderately fractured masses.

Deformation Testing Methodology

Deformation tests were conducted using the static loading method, where the surface of the rock was loaded through a rigid concrete stamp.

Each test consisted of five loading-unloading cycles with maximum specific loads in the cycles:

$\sigma max=1.0;2.0;3.0;4.0;$and 5.0 MPa.\sigma_{max} = 1.0; 2.0; 3.0; 4.0; \text{and } 5.0 \text{ MPa}.

In each cycle, the normal load on the stamp was increased and decreased in stages (5-7 stages during loading and 4 stages during unloading).

The processing of test results for a given stamp involved calculating the average settlement values of its foundation at each loading stage and constructing pressure-settlement dependence curves "pressure on the stamp σ\sigma – foundation settlement SS" for all five test cycles.

Calculation of Deformation and Elasticity Moduli

Based on the test results, for each loading-unloading cycle, the deformation modulus EE and elastic modulus EuE_u of the rock mass were determined using the following formula [1]:

$E(Eu)=b\cdot w\cdot(1-\nu 2)\cdot\Delta\sigma\Delta sE$ (E_u) = b \cdot w \cdot (1 - \nu^2) \cdot \frac{\Delta\sigma}{\Delta s}

where:

- bb – width of the stamp;
- ν\nu – Poisson's ratio of the rock (assumed to be 0.25);
- $\Delta\sigma$\Delta\sigma – change in normal pressure (stress) applied to the stamp;
- Δs\Delta s – change in foundation settlement caused by the corresponding change in pressure $\Delta\sigma$\Delta\sigma;
- ww – constant coefficient considering the shape and stiffness of the stamp (for the given tests, w=0.88w = 0.88 [1]).

It should be noted that the Poisson's ratio within its possible range has a very small effect on the deformation modulus values.

The deformation modulus EE was determined along the loading branch, where the normal pressure increased from zero to the maximum value σmax\sigma_{max} in the respective cycle.

Meanwhile, the elastic modulus EuE_u was determined along the unloading branch, where the normal pressure decreased from σmax\sigma_{max} to zero in the same cycle.

An example of the dependency graphs s=f(σ)s = f(\sigma) is shown in Figure 1.

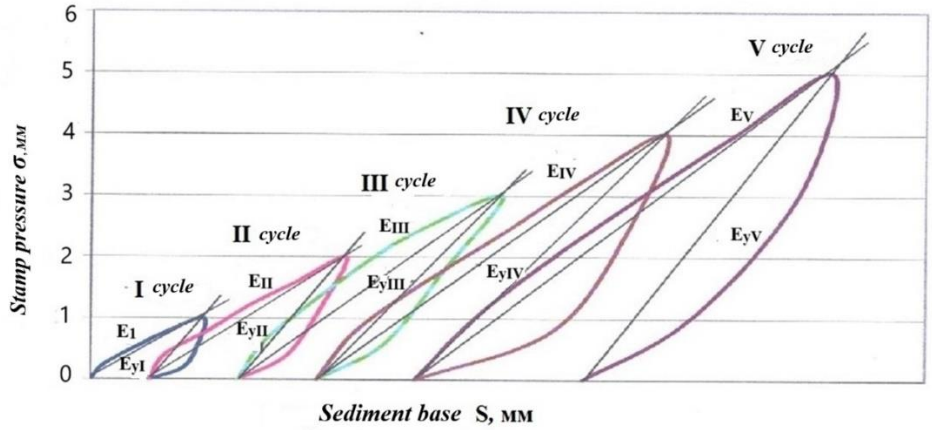


Figure 1. Example of Graphs s=f(σ)s = f(\sigma) from Stamp Load Tests

EI,…EVE_I, … E_V – Deformation moduli at different loading cycles, EyE_y – Elasticity moduli.

The calculated deformation and elasticity moduli were determined based on the results obtained in the 4th and 5th loading-unloading cycles, where the specific normal loads were increased to values corresponding to the maximum operational loads on the dam foundation (4-5 MPa).

Additionally, during the first and second loading cycles, the rock mass, which had been loosened due to tunneling excavation, was compacted back to its natural state.

The obtained values of the deformation modulus EE and elastic modulus EyE_y from the fourth and fifth loading cycles are presented in Table 2.

For consistency, in further discussions, the terms deformation and elasticity moduli will refer to their average values, obtained from the 4th and 5th test cycles, which are provided in columns 4 and 7 of Table 2.

Table 2

Values of Deformation Moduli EE and Elasticity Moduli EyE_y of the Rock Mass Obtained from Stamp Load Tests

Stamp Number	Values of modulus of deformation and elasticity, MPa						(Ey)a/(E)a
	E			Ey			
	4-й cycle	5-й cycle	Average	4-й cycle	5-й cycle	Average	
1	2	3	4	5	6	7	8
BS-7	1925	2188	2057	2357	2977	2667	1,30
BS -8	5022	4894	4958	6243	6144	6194	1,25
BS -9	9625	8750	9188	10500	10694	10597	1,15
BS -10	6243	7219	6731	6794	8493	7644	1,14
BS -11	3039	3438	3238	4053	4310	4181	1,29
BS -12	4915	5156	5036	5634	5662	5648	1,12

As can be seen from Table 2, the magnitude of the deformation modules of the E array in the experimental sections varies from 2060 to 9190 MPa with an average value of 5200 MPa. The minimum modulus of deformation for the experimental sections was obtained on concrete die No. 7 (BS-7), the maximum on die No. 9 (BS-9). The ratio of the modulus of elasticity to the modulus of deformation of the array varies from 1.12 to 1.30, averaging 1.21.

Table 3

Recommended calculated values of the deformation modulus E and elastic modulus E of the stamping pads

The number of the site	7	8	9	10	11	12	Average
(stamp)	2100	5000	9200	6700	3200	5000	5200
E, MPa	2700	6200	10600	7600	4200	5600	6200

Recommended Calculated Deformation and Elasticity Moduli

The recommended calculated deformation and elasticity moduli for each site are presented in Table 3. The calculated moduli were determined as the average values between the 4th and 5th cycle moduli, rounded to 100 MPa.

Unfortunately, the analysis performed did not reveal any correlation between the obtained deformation moduli and the fracture porosity coefficients of the test sites.

Shear Tests on Stamps

After completing the deformation tests, shear tests were conducted on the same concrete stamps.

Each stamp underwent two primary shear tests under different normal loads, where the ultimate shear strength at the concrete-rock contact (τpr\tau_{pr}) was determined.

- The first shear test was stopped when a noticeable deviation of the horizontal displacement uu of the stamp from the initial linear relationship u=f(τ)u = f(\tau) was observed, indicating that the shear strength limit was approaching. In most cases, observations of the vertical displacement vv of the stamp showed an upward movement.
- The second shear test, under a different normal load, continued until both the ultimate shear strength τpr\tau_{pr} and residual shear strength τost\tau_{ost} were reached. The residual shear strength τost\tau_{ost} was also assessed based on repeat shear tests along the formed shear surface, with three repeated shear tests performed on each stamp.

Thus, a total of five shear tests were conducted on each stamp:

- Two primary shear tests
- Three repeated shear tests

All tests were conducted using a moment-free scheme.

Table 4: Values of Normal Stresses σ\sigma in Primary Shear Tests on Stamps

Stamp Number		7	8	9	10	11	12
Magnitude of σ\sigmaσ, MPa	First shift	0,5	2,0	1,5	1,0	2,5	3,0
	The second shift	3,0	1,0	2,5	2,0	1,5	0,5

Methodology

The primary shear tests were conducted at six different normal pressures applied to the stamp: 0.5 MPa, 1.0 MPa, 1.5 MPa, 2.0 MPa, 2.5 MPa, and 3.0 MPa.

As mentioned earlier, each stamp underwent two primary tests at two fixed values of normal pressure (σ\sigma). The values of normal stresses used in primary tests for different stamps are presented in Table 4.

The repeated shear tests on each stamp were conducted at three normal pressures: 1.0 MPa, 2.0 MPa, and 3.0 MPa.

Recorded Data During Shear Tests

During the primary shear tests, the following parameters were recorded:

- The shear stress (τ\tau) applied to the stamp.
- The horizontal displacement (uu) and vertical displacement (vv) of the stamp caused by this stress.
- The normal stress on the stamp was maintained constant throughout the test.

The output data from the tests included:

1. The ultimate shear strength (τpr\tau_{pr}).
2. The horizontal displacement of the stamp (upru_{pr}) at which the ultimate shear strength was reached.
3. The residual shear strength (τost\tau_{ost}) obtained from the second shear test.

The results of each completed shear test were represented as graphs of the relationship between horizontal displacement uu and shear stress τ\tau.

A characteristic example of such a u=f(τ)u = f(\tau) dependency graph for stamp BS-7, with the parameters mentioned above, is shown in Figure 2.

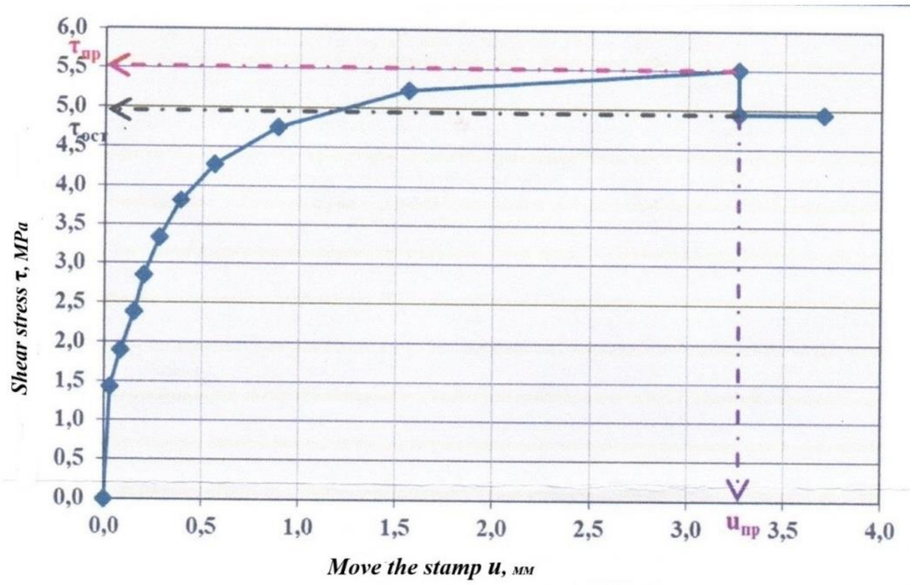


Figure 2. Dependence of Horizontal Displacements uu of Stamp BS-7 on Applied Shear Stresses τ\tau at σ=3.0\sigma = 3.0 MPa

(τpr=5.52\tau_{pr} = 5.52 MPa, upr=3.26u_{pr} = 3.26 mm, τost=4.95\tau_{ost} = 4.95 MPa)

A particularly notable result is the high shear strength value (τpr=3.38\tau_{pr} = 3.38 MPa) obtained during the second shear test on stamp BS-8 under a low normal load of σ=1.0\sigma = 1.0 MPa.

A significant deviation in the ultimate shear strength recorded on stamp BS-8 can be clearly seen in Figure 3, which presents the shear coefficient values

Kshear=τshearσK_{shear} = \frac{\tau_{shear}}{\sigma}

determined from the first and second shear tests on all stamps.

As illustrated in the figure, the shear coefficient obtained in this test (3.38) is nearly twice the average shear coefficient (1.77) calculated across all tests.

Therefore, this value is considered a random outlier and was excluded from the determination of the design shear parameters (friction coefficient and cohesion).

Table 5: Shear Strength Values τ_{pr}, τ_{ost} and Corresponding Displacements u_{pr} Obtained in Shear Tests

Stamp number Normal pressure σ,	Stamp number Normal pressure σ,	Stamp number Normal pressure σ,	Stamp number Normal pressure σ,	Stamp number Normal pressure σ,
1	2	3	4	5
BS-7	0,5	1,24	-	0,52
	3,0	5,52	4,95	3,26
BS-8	2,0	2,44	-	0,46
	1,0	3,38	2,82	2,53
BS -9	1,5	3,15	-	0,97
	2,5	4,78	4,11	1,78
BS -10	1,0	1,41	-	0,45
	2,0	4,04	3,29	4,52
BS -11	2,5	2,85	-	2,03
	1,5	2,66	2,47	1,85
BS -12	3,0	3,92	-	2,78
	0,5	1,15	1,03	0,99

The results of the remaining 11 primary shear tests are presented in Figure 4.

Through statistical processing of the obtained results, the following standard values of shear parameters—friction coefficient ($\tan \phi$) and cohesion (C)—were determined:

$\tan \phi_n = 1.40, \quad C_n = 0.47 \text{ MPa}.$

This yields the following equation for ultimate shear strength:

$$\tau_{pr} = 1.40 \sigma + 0.47, \quad \text{MPa} \quad (2)$$

To obtain design shear parameters, the standard values $\tan \phi_n$ and C_n must be divided by the soil reliability coefficient γ_g.

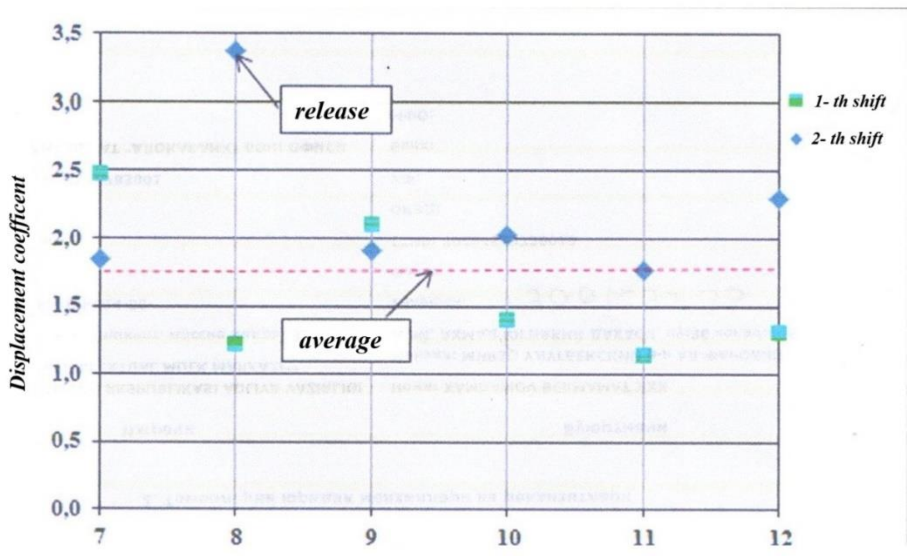


Figure 3. Shear Coefficients Obtained from Tests on Stamps BS-7 to BS-12

This figure presents the shear coefficients ($K_{shear} = \tau_{shear} / \sigma$) determined from the shear tests conducted on stamps BS-7 to BS-12. The data illustrate variations in shear strength relative to normal stress across different test conditions.

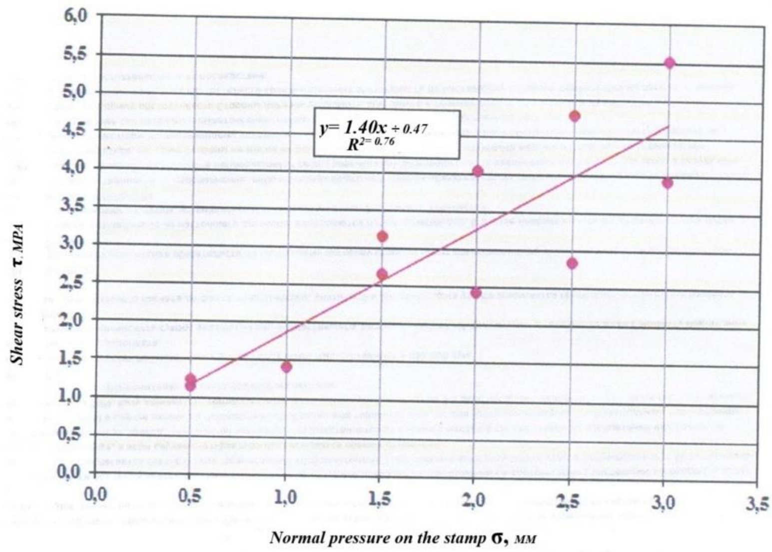


Figure 4. Dependence of Ultimate Shear Strength on the Magnitude of Specific Normal Load on Concrete Stamps BS-7 to BS-12

The corresponding Coulombian dependence of ultimate shear strength on the applied normal pressure follows the equation:

$\tau_{pr} = \tan \phi \cdot \sigma + C$

Reliability Coefficient and Design Shear Parameters

According to the recommendations of SP 23.13330.2011 [3], the reliability coefficient for soil (γ_g) should be determined following GOST 20522 [4] with a one-sided confidence probability of $\alpha = 0.95$.

The calculations showed that the reliability coefficient for soil in this case is:

$\gamma_g = 1.33$

Considering SP 23.13330.2011 (Clause 5.16), which states that if $\gamma_g > 1.25$, the value should be taken as $\gamma_g = 1.25$, the design shear characteristics are obtained as follows:

$\tan \phi_r = 1.12, \quad C_r = 0.38 \text{ MPa}.$

In addition to the reliability coefficient (γ_g), SNiP 2.02.02-85 [5] and SP 23.13330.2011 [3] recommend introducing an additional safety factor to account for possible discrepancies between test conditions and real-world conditions.

Taking this into account, along with the significant variation in experimental data that resulted in a high value of γ_g, the final recommended design shear parameters are:

$\tan \phi_r = 0.90, \quad C_r = 0.30 \text{ MPa}.$

Residual Shear Strength Analysis

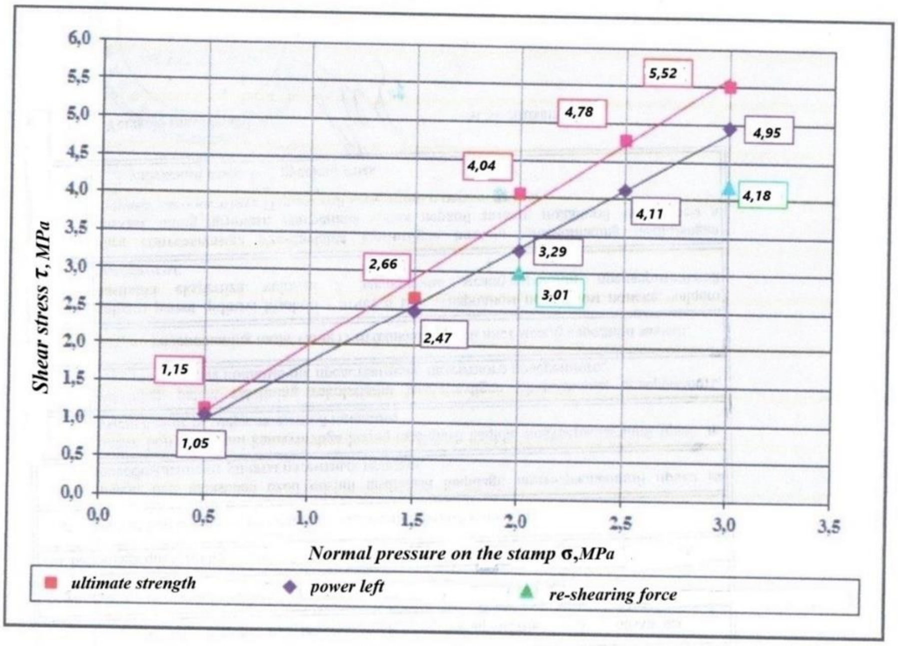
The residual shear strength (τ_{ost}) can be evaluated based on:

- Primary tests, where stamp displacements were recorded after reaching ultimate shear strength (second shear tests on each stamp).
- Repeated shear tests, conducted on the same stamps.

Figure 5 presents a comparison of ultimate shear strength (τ_{pr}) and residual shear strength (τ_{ost}) obtained from these tests (excluding and including the second shear test on stamp BS-8).

The data analysis shows that the residual shear strength determined during primary tests is 7.1% to 18.6% lower than the ultimate shear strength, with an average reduction of approximately 12%.

Further results from repeated shear tests, performed within the range of normal stress values, confirm these findings.



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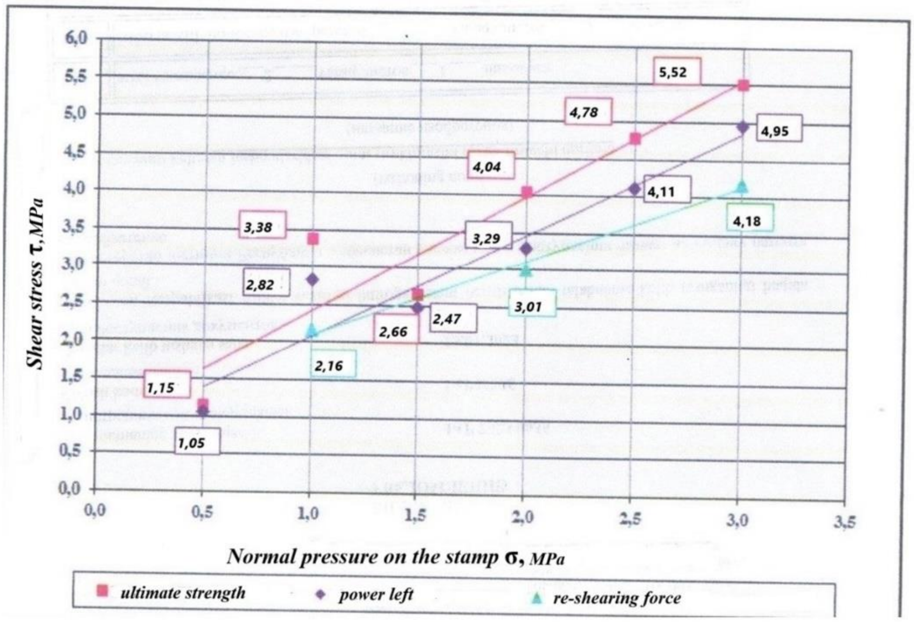

Figure 5. Values of Ultimate (τpr\tau_{pr}) and Residual (τost\tau_{ost}) Shear Strengths Obtained in Primary (Second Shear) and Repeated Shear Tests, Excluding (a) and Including (b) Data from Stamp BS-8

In shear tests conducted at normal stresses ranging from 1 to 3 MPa, the residual shear strength (τost\tau_{ost}) was found to be 14.2% to 36.1% lower than the ultimate shear strength, with an average reduction of approximately 25%.

It is important to note that the necessity of considering residual shear strength in calculations may arise after strong earthquakes or other extreme force impacts on the rock mass, which can cause even minor displacements.

CONCLUSIONS

1. This study presents the results of geomechanical investigations of aleurolite rock masses, conducted in the right-bank experimental tunnel at the dam site of the Pskem Hydropower Plant.

2. The fracture porosity coefficient (KTP) of the rock at the test sites is insignificant, averaging 0.23%. According to the existing classification, the tested rocks are mainly weakly fractured or very weakly fractured.
3. Based on deformation test results, the deformation modulus (EE) of the rock mass at the test sites ranges from 2,100 to 9,200 MPa, with an average value of 5,200 MPa.
4. Based on the shear test results, the following recommended design shear strength parameters for the rock mass and the concrete-rock interface are: $\tan\varphi_r=0.90, C_r=0.30$ MPa.\tan \phi_r = 0.90, \quad C_r = 0.30 \text{ MPa}.

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