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THE SHEAR STRENGTH OF INFILLED ROCK JOINTS

According to the Jaeger's theory, the minimum possible rock mass shear strength as a discontinuum actually corresponds to the shear strength of rock joints. Since failures in rock masses due to loads caused by civil structure and/or civil works occur mainly by exceeding their shear strength, the shear strength of rock joints has huge practical significance in rock engineering. For this reason, in this paper it was decided to analyse one of the factors which have very important influence on the shear strength of rock joints. Namely, natural rock joints are often filled with soft soil material and this infill material may have a significant and often decisive influence on the shear strength of rock joints. As a basis for the conducted analyses were used the results of direct shear tests under constant normal stress which was performed on natural or artificial specimens with horizontal infilled joints by various researchers around the word. Analyses have shown that some basic principles of mechanical behaviour of infilled rock joints during shearing can be reached. The thickness *t* and mechanical characteristic of the infill material have decisive influence on the peak and residual shear strength of infilled rock joints.

Keywords: infilled rock joints, shear strength, direct shear test, infill material thickness

1. INTRODUCTION

The space between the walls of open rock joints is often not empty or filled only with water and/or air. This space is often filled with material which can be primary or secondary origin i.e. it can be a product of physical and chemical degradation of adjacent rock blocks or it can transported material. The infill material can be such a mineral composition and mechanical characteristics that its shear strength is approximately equal to the shear strength of the intact rock. In this situation actually we talk about healed rock joints. However, from the aspect of shear strength of rock joints, special attention is required by geological situations when the space between the walls of joints is filled with soft non-cohesive (sand) or cohesive (clay) infill material. In the following text, more will be said about these situations.

Factors that affect the shear strength of natural unfilled rock joints (joint surface roughness, compressive strength at the joint surface, normal stress during shearing, scale effect, etc.) also affect the shear strength of natural infilled rock joints. However, there are two additional extremely important factors in the case of infilled rock joints. These are the infill material thickness *t* and the mechanical characteristics of the infill material. In general, the infill material thickness has a decisive or dominant influence on mechanical behaviour of infilled rock joints during shearing. However, this thickness is a relative category because it must be observed in relation to the height (amplitude) of the joint surface asperities *a*. It is concluded that in fact the relative infill material thickness i.e. the ratio *t/a* is the one that has a dominant influence on the mechanical behaviour of the infilled rock joints during shearing. Many experimental, laboratory research have proven that with increasing values of the ratio *t/a* peak and residual shear strength of the infilled rock joints decrease. The main reason for this drop in shear strength is the decreasing direct contact between the joint walls i.e. between irregularities on the joint walls (rock-rock asperity contact) with increasing infill material thickness. This actually changes the geometry of the shear plane, prevents the development of dilation and interlocking effects. Also, the presence of the infill material reduces the basic friction angle ϕ_b i.e. reduces the coefficient of friction of joint surface. This is very important for value of residual shear strength.

2. DETERMINATION OF THE SHEAR STRENGTH OF ROCK JOINTS

Determination of shear strength of rock joints is generally performed in displacement controlled direct shear test along joint under constant normal stress or constant normal stiffness. In accordance with ISRM (2013), rock specimens with a regular (rectangular or elliptical) cross-section are preferred. The length of the tested rock joint i.e. tested rock specimen (measured along the shear direction) should be at least 10 times the maximum joint wall asperity height and sufficient to encapsulate the specimen in the specimen holder. Of course, this length must be significantly greater than maximum shear displacement during test. The width of the tested rock joint i.e. tested rock specimen (measured perpendicularly to the shear direction) should have at least 48mm and this width should not change significantly over the shearing length. Minimum width of tested rock joint should be greater than 75% of its maximum width.

In accordance with ISRM (2013), in the first phase of direct shear test along joint normal stress should be applied on the rock specimen continuously at selected rate of normal stress. The rates of 0.01 MPa/s or less are recommended. In the second phase of test, after the normal displacements stabilize under the applied normal stress, shear displacement should be applied on the rock specimen continuously at selected rate of shear displacement until ultimate or residual shear stress is reached. Shear displacement rates around 0.1–0.2 mm/min are usually suitable for the whole test, although it can be slightly increased up to values around 0.5 mm/min after peak shear strength. The normal and shear forces are measured with accuracy better than ±2.0% directly by load cells, or indirectly by pressure gauges, transducers, or proving rings. Displacement transducers are used to measure the displacements. A minimum of two displacement transducers are required: one mounted parallel with the rock joint to measure the shear displacement and one mounted vertically at the centre of the specimen to measure normal displacement.

3. ANALYSIS RESULTS OF SAME EXPERIMENTAL RESEARCH

Papaliangas et al. (1993) is examined in shear displacement controlled direct shear test along joint under constant normal stress a large number of artificial sandstone prismatic specimens of dimensions 12cm/25cm/12cm. All specimens contained one horizontal infilled joint with smooth and undulated walls (average asperities height of 7 mm) and different infill material thicknesses. The specimens were formed by hardening a mixture of silver sand, dental plaster, water and additives (calcined alumina and mineral barite). In this way was obtained a material which had a density of 1.85mg/m3, uniaxial compressive strength of 3.50MPa, point load strength of 0.45MPa and Young's modulus of elasticity of 0.60GPa. Dry, non-cohesive pulverised fuel ash with almost spherical particles of glass, specific gravity of 2.39mg/m3, mean particle size of 0.001mm and shear resistance angle of 33º was used as the infill material. In this way, the obtained specimens with infilled joint which are represent prototypes of real rock block of dimensions 15 times those of the specimen and strength parameters 20 times those of the specimen. The pulverised fuel ash simulated a silt or silty-sandy infill material of natural rock joints. The results of performed direct shear tests along joint under constant normal stress are shown below.

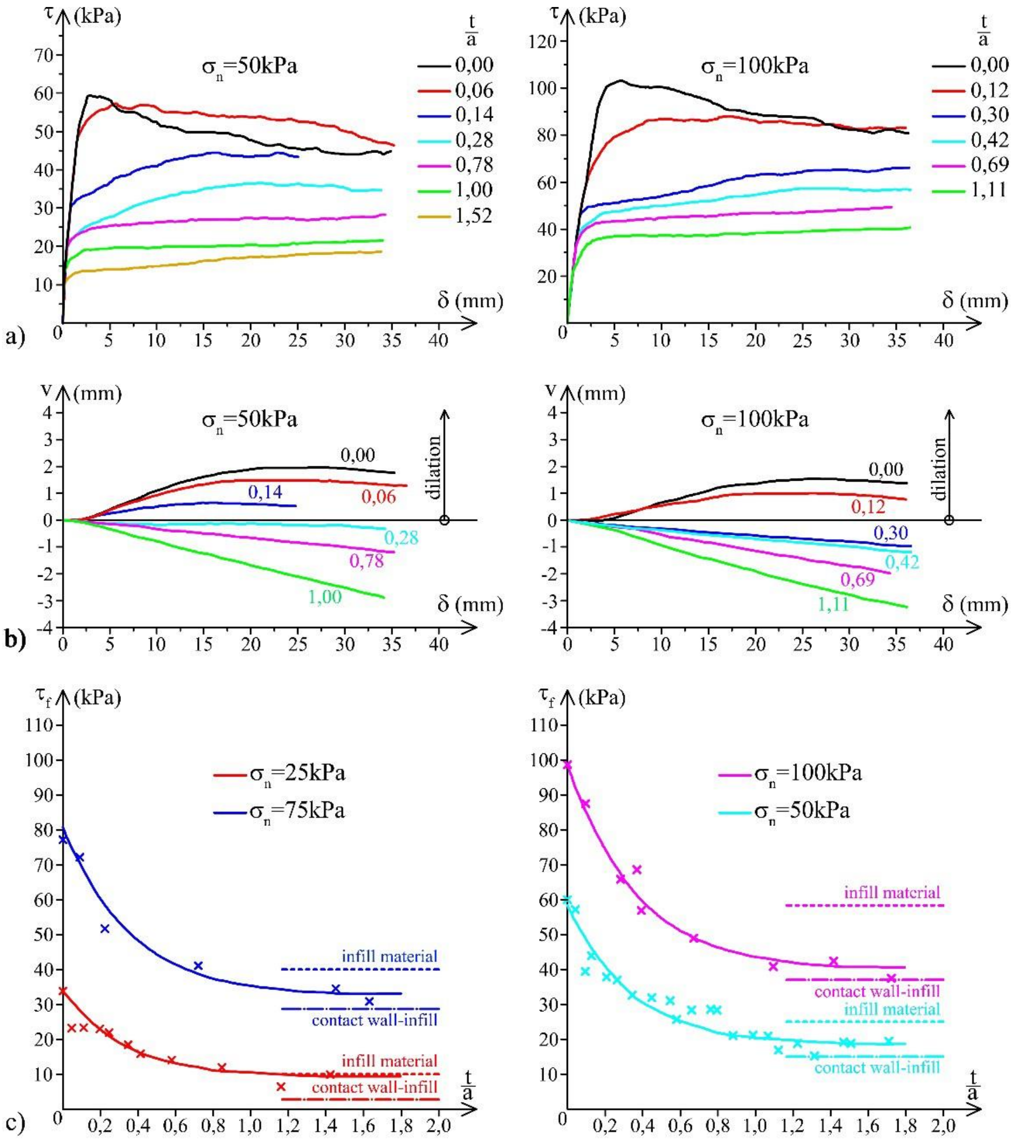

Figure 1. Results of direct shear tests along joint under constant normal stress for infilled artificial rock joints with smooth and undulated walls a) Shear stress τ vs shear displacement δ curves for different value of ratio t/a b) Vertical displacement v vs shear displacement δ curves for different value of ratio t/a c) Peak shear strength τ_f vs ratio t/a curves for different value of normal stress during shearing (Papaleangas et al., 1993)

The presented results indicate the expected decrease in the peak and residual shear strength of the examined infilled rock joints as well as a gradual change in their mechanical behaviour during shearing under constant normal stress with increasing their ratio *t/a* i.e. with increasing their relative infill material thickness. Also, it can be seen that instead of dilation a contraction of examined specimen is registered when its value of the ratio t/a ratio is greater than approximately 20%.

Special attention should be paid to the recorded values of the peak shear strength of the infilled joints at a relatively large infill material thickness i.e. in situations when the ratio *t/a*>100%. Figure 1c shows that these minimum values of the peak shear strength of the tested infilled joints were often significantly lower than the shear strength of the infill material itself at the same value of normal stress during shearing. In Figure 1c, the colored dashed lines (the color indicates the value of the normal stress during shearing) define the shear strength of the infill material itself at the corresponding value of normal stress during shearing. So, Papaliangas et al. (1993) unequivocally proved that the shear plane of does not always pass through the infill material. Actually, the contact between joint walls and infill material (contact "joint wall-infill") is often the weakest part of the infilled joint.

In order to complete the obtained results, the same author and his collaborators formed and tested several specimens of the same rock-like material (artificial sandstone) with planar and smooth horizontal joints and relatively large infill material thickness (pulverised fuel ash) in the direct shear test under constant normal stress. In this way, they experimentally defined the shear strength of the contact "joint wall-infill" for some values of normal stress during shearing. Since the joints walls of these additionally tested specimens were planar and smooth, their measured shear strengths at the same time represent the minimum possible shear strengths of the all analyzed infilled joints for the corresponding values of normal stresses during shearing. Of course, this minimum values of shear strength correspond to the dimensions of the tested specimens, physical and mechanical characteristics of rock-like material and machanical characteristics of infill material. In Figure 1c, the colored dash-dotted lines (the color indicates the value of the normal stress during shearing) define values of these minimum possible shear strengths of all tested infilled joints (shear strength of contact "planar and smooth joint wall-infill") for corresponding values of normal stresses during shearing.

Based on the obtained results of the direct shear test along joint, idealized curves of the change in the normalized shear strength τ/σ_n of the infilled rock joint with increasing relative infill material thickness i.e. with increasing values of the ratio *t/a* were formed (Figure 2). Two types of infilled joints that differ from each other only in terms of the joint surface roughness (the same type of rock and the same type of infill material) were analyzed. The first type, marked I, represents an infilled joint with very rough surface. The second type, marked II, represents an infilled joint with planar surface.

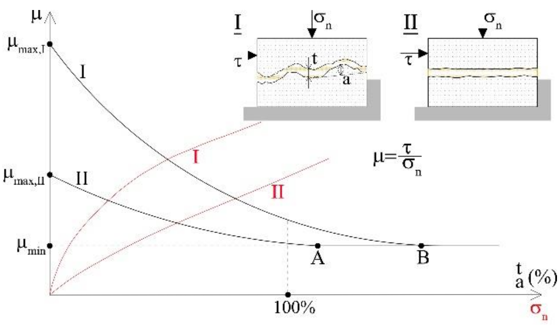

Figure 2. Idealized curve of the change in the normalized shear strength of infilled rock joint with increasing relative infill material thickness (modified after Papalianges et al., 1993)

At very small values of the ratio *t/a* (a few percent), i.e. at very small relative infill material thickness, the presence of infill material can be completely neglected. For both treated types of rock joints the shear strength is approximately equal to its maximum possible value (τ_{max} or μ_{max}), which actually corresponds to the shear strength of the identical unfilled rock joints for the same value of normal stress. In the figure above, red lines represent normalized shear strength envelopes for the identical unfilled rock joints I and II. However, with increasing infill material thickness, the shear strength of the rock joints decrease. This decrease is very pronounced in joint type I because with increasing infill material thickness dilation is becoming less pronounced. In these situations, the actual shear strength of the infilled rock joint depends on the geometric and mechanical characteristics of its surface (walls) as wall as the mechanical characteristics of the infill material.

With a further increase in the relative infill material thickness the shear strength of the joints decreases but at a decreasing speed. So, the shear strength of the joints asymptotically tends some of its final minimum value (τ_{min} or μ_{min}). It is important to note two facts that have been experimentally confirmed several times. The first fact refers to the moment of reaching the minimum shear strength of the rock joints. In rock joint type I (joint with very rough surface) at the moment when the ratio *t/a*=100%, its shear strength is still approximately 10% to 50% higher than the minimum value, depending on the level of normal stress during shearing (Goodman, 1970; Ladanyi & Archambault, 1977). This fact can be considered as the influence of the joint surface roughness and the compaction (consolidation) of the infill material during shearing. In other words, with joint type I, the minimum shear strength is reached when the infill material thickness is significantly greater than the height (amplitude) of joint surface asperities (point B). The described excess of the height of the asperities can be approximately 25% to 50% (Papaliangas et al., 1993) or even more than 100% (Barton, 1973). For joint type II at the moment when the ratio *t/a*=100% shear strength of rock joint is approximately equal to the minimum shear strength.

Second fact to note relates to the value of the minimum shear strength of the infilled rock joint. In general, this value corresponds to the shear strength of the infill material itself. Civil enginners in practice usually think this way in situations when the ratio is *t/a*≥100%. However, experimental research has shown that in these situations the shear strength of the infilled rock joint may be less than the shear strength of the infill material itself. These are usually situations with non-cohesive, fine-grained infill material with low water content and/or situations with planar and smooth joint surface. In these situations, in fact the shear plane does not pass through the infill material but passes completely or for the most part through the contact of the joint surface (joint wall) and the infill material.

Figure 3 shows the typical shear stress τ vs shear displacement δ curves for rock joint type I with different infill material thickness in displacement controlled direct shear test along joint under constant normal stress (σ_n=50kPa).

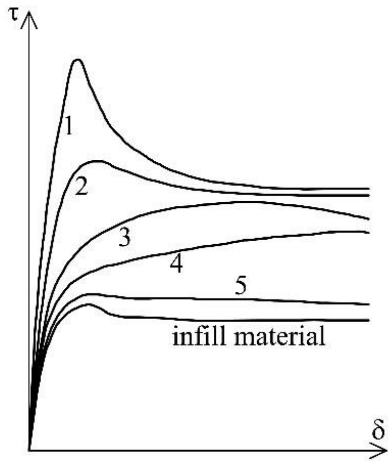

Figure 3. Typical shear stress τ vs shear displacement δ curves for joint type I with different infill material thickness in direct shear test under constant normal stress (Papalianges et al., 1993)

As previously established, when the value of the ratio *t*/*a* is very small (a few percent), then the mechanical behaviour of the rock joint type I during shearing is actually the same as the behaviour of the identical unfilled rock joint during shearing. The initial stiffness of the joint is large and its peak shear strength is reached at a relatively small shear displacement δ (curve 1).

With increasing infill material thickness *t* the contribution of dilation to the shear strength of rock joint type I is smaller, which leads to a significant decrease in its peak shear strength. Residual shear strength of also decreases but to a much lesser extent than peak shear strength. The initial stiffness of the rock joint type I decreases but the value of the shear displacement at failure increases (curve 2, *t*/*a*≈10%). Soon, with a further increase in the infill material thickness instead of dilation of the analyzed infilled rock joint type I during the shear process, a contraction is registered. This negatively affects primarily the value of its peak shear strength which is registered at a relatively large shear displacement. The initial stiffness of the rock joint type I is decreasing, as well as its residual shear strength (curve 3, *t*/*a*≈20%).

With a further increase in the infill material thickness, plastic stress-strain behaviour with obvious strain hardening is registered. The maximum value of shear stress is registered at the end of the test, i.e. at the maximum value of the applied shear displacement. Residual shear strength can not be reached (curve 4, quite approximately 30%≤*t*/*a*≤75%).

For the case when the infill material thickness is approximately equal to the height (amplitude) of joint surface asperities *a*, the mechanical behaviour of analysed infilled rock joint type I during shearing corresponds to the mechanical behaviour of the infill material. Due to the influence of joint surface roughness, the peak and residual shear strength of the analyzed rock joint may be slightly higher than the peak and residual shear strength of the infill material. In this general description of the mechanical behaviour of infilled rock joints in the direct shear test under constant normal stress, the mechanical behaviour of the infill material during shearing corresponds to the mechanical behaviour of normally consolidated clays and loose sands during shearing in direct shear test under constant normal stress.

Masoud (2015) performed a very interesting study of the shear strength of natural infilled rock joints. With standard geotechnical field works (drilling and sampling) but very careful, from sandstone rock mass he extracted quality prismatic specimens with orientation dimensions of B/L/H=7.0cm/7.0cm/15cm. All specimens were intersected in the middle by a natural horizontal unfilled joint with smooth and undulated surfece. For all specimens joint roughness coefficient JRC was around 7. After that, in the laboratory, the specimens were separated and then a layer of infill material of a certain thickness over the joint walls was carefully added (Figure 4). Three types of materials for the infill of the rock joints regarding their graining including were used: sand, clay and sandy-clay. Then, all natural specimens were reassembled and tested these specimens in a direct shear test under constant normal stress. Nine different values of ratio *t*/*a* in the range of 0.0 to 1.6 was considered. Four different values of normal stress during shearing of 0.25MPa, 0.50MPa, 0.75MPa and 1.0MPa was considered. Figure 5 shows only part of the results of the laboratory stady carried out by Masoud (2015).

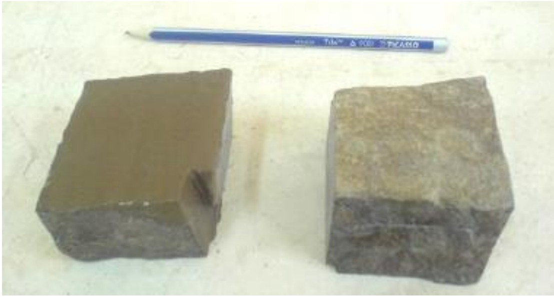

Figure 4. Addition of the clayey infill material over the rock joint (Masoud, 2015)

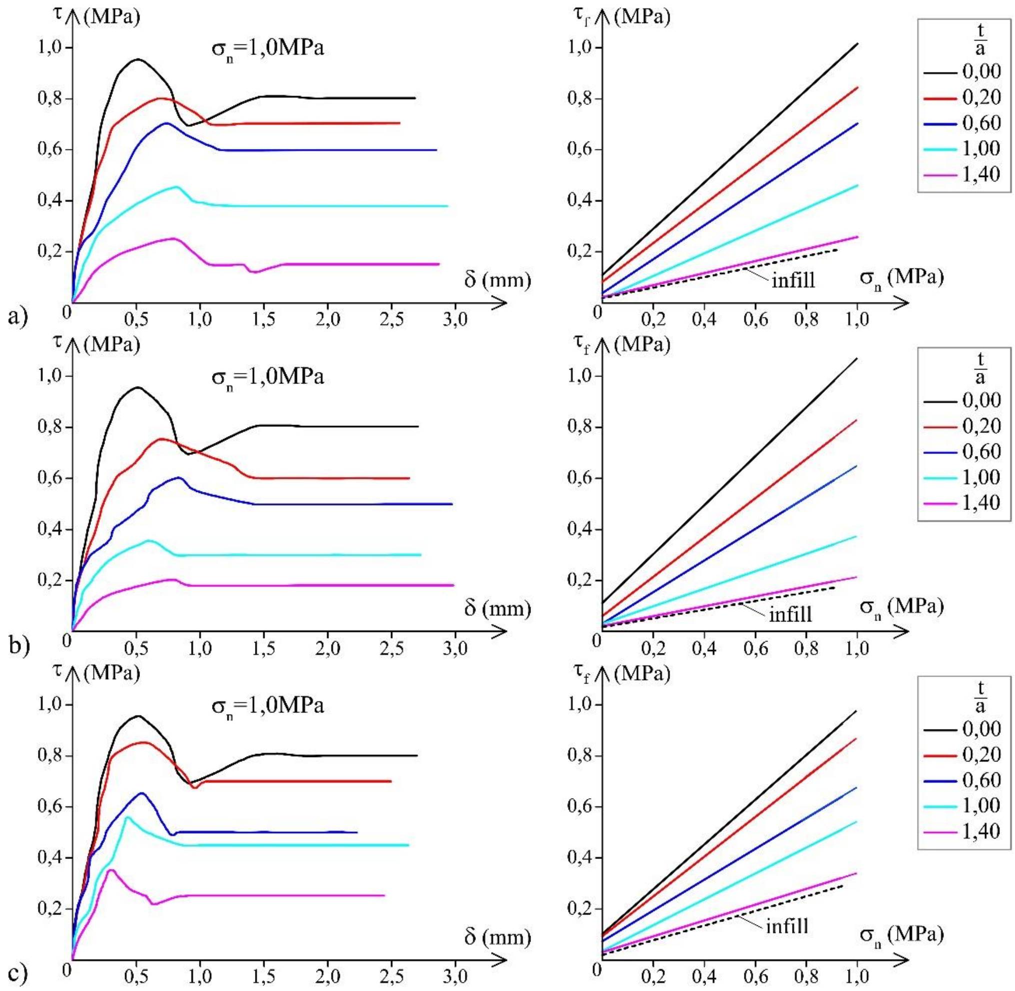

Figure 5. Shear stress τ vs shear displacement δ curves and linear shear strength envelopes for natural infilled rock joints with different values of ratio *t/a* in direct shear test along joint under constant normal stress
a) Clayey infill material b) Sandy infill material c) Sandy-clayey infill material (Masoud, 2015)

The presented results confirm the fact that the initial stiffness, peak and residual shear strength of the infilled rock joints decrease with increasing relative infill material thickness i.e. with increasing ratio *t/a*. The values of shear displacement at the failure of these rock joints generally increase with increasing infill material thickness. There is a certain deviation regarding the shape of the recored shear stress τ vs shear displacement δ curves in relation to the shape of these curves which was declared as typical by Papaleangas et al. (1993). This deviation primarily refers to the occurrence of strain hardening that Masoud (2015) did not record. The slope of linear shear strength envelope of infilled rock joints decreases with increasing value of the ratio *t/a*. This slope actually tends the shear resistance angle of the infill material itself. Also, by increasing the value of the ratio *t/a*, the cohesion of the infilled joints i.e. the section of their shear strength envelope on the vertical axis decreases. This is due to a decrease in dilation with increasing infill material thickness, resulting in a smaller curvature of the shear strength envelope in the zone of small normal stresses.

4. CONCLUSION

The presence of a infill material between the walls of a rock joint negatively affects its shear strength. As the infill material thickness increases, the shear strength of the infill rock joint decreases and asymptotically tends to same minimum value. This minimum value of shear strength of infilled rock joint with rough surface is actually equal to the shear strength of the infill material itself. However, for infilled rock joint with planar and smooth surface, the minimum shear strength may be even lower and actually correspond to the shear strength of the contact "planar and smooth joint wall-infill". Of corse, mechanical behaviour of infill rock joints during shearing depends on mechanical characteristic of infill material.

In this paper, the results of displacement controlled direct shear tests along joint in which the rock specimens (joints) were loaded with shear load in one direction to failure are analyzed. However, since we are in the area with high degree of the seismic hazard, it is very important to know the mechanical behavior of rock joints during cyclic shearing. This is one of the tasks of future researches. Also, it is interesting to analyse mechanical behavior of intermittent rock joints.

When studying the shear strength of rock joints, it is always a challenge to conduct direct shear tests along joint on natural rock specimens. One such research is planned to be conducted in the laboratory of the Faculty of Civil Engineering in Podgorica. Marl specimens with natural joints will be used. In addition to standard displacement controlled direct shear tests, it is planned to load rock joints with long-term shear load.

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