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Review paper

Influence of material bridges on shear strength of unfilled intermittent rock joints

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ABSTRACT

According to the well-known 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 structures or civil works occur mainly by exceeding their shear strength, the shear strength of rock joints has huge practical significance. Therefore, in this paper it was decided to analyse a factor which can have a very important influence on the shear strength of unfilled rock joints. Namely, the influence of the presence of material bridges and initial joints between them, their number, size, mutual distance and orientation relative to the shearing direction were analyzed using results of laboratory or numerical CNS direct shear tests which were carried out by different researchers around the world. Except for increasing its peak and residual value, the professional public is currently unaware of the impact of this factor on the shear strength of intermittent rock joints. Based on the carried out analysis, appropriate conclusions have been made.

1 Introduction

The single plane of weakness theory proposed by Jaeger is the most widely known in Rock Mechanics. According to Jaeger's theory, the minimum possible rock mass shear strength as a discontinuum actually corresponds to the shear strength of rock joints (discontinuities). Since failures in rock masses due to loads caused by civil structures or civil works occur mainly by exceeding their shear strength, the shear strength of rock joints has huge practical significance in Rock engineering, especially in Rockslide and slope engineering. Therefore, in this paper it was decided to analyse the factor which has a very important influence on the shear strength of unfilled rock joints and their mechanical behaviour during shearing. Namely, after a short analysis of the influence of well-known factors such as joint surface roughness and rock compressive strength at the joint surface, special attention is paid to the influence of the presence of material bridges and initial joints between them, their number, size, mutual distance, and orientation relative to the shearing direction.

2 Mechanical behaviour of unfilled rock joints during shearing

A number of factors influence the mechanical behaviour of natural unfilled rock joints during shearing, including joint surface roughness (roughness of the joint walls), the presence of material bridges and their configuration, rock compressive strength at the joint surface (compressive strength of the joint walls), the level of normal stress during shearing σ_n, and the scale effect. These factors will be analyzed in more detail below.

2.1 Joint surface roughness

A planar, smooth, and clean natural rock joint is very rare. All natural rock joint surfaces exhibit some degree of roughness, varying from smoothed and polished sheared rock joints with very low surface roughness to very rough tension rock joints. This degree of roughness has a significant effect on the shear strength of unfilled rock joints. This effect can be quantitative or qualitative in nature. Quantitative means that joint surface roughness often increases the peak shear strength of rock joint τ_p well above the minimum value of shear strenght [1, 2]. This minimum shear strenght is defined by the basic friction angle ϕ_b and its characteristic for planar, smooth, and clean unfilled rock joints (Fig. 1). Qualitative means that joint surface roughness can cause a change in the type of mechanical behaviour of unfilled joints during shearing. It is quite clear if we compare the shear stress τ vs. shear displacement δ curves (hereinafter shear curves) for planar, smooth and unfilled rock joints in granite Hwangdeung recorded by Jang et al. [3] in the CNS direct shear test (Fig. 2) with normalized shear stress τ/σ_n vs. shear displacement δ curves (hereinafter normalized shear curves) for natural joints in different rocks recorded by Grasselli [1] also in the CNS direct shear test (Fig. 3).

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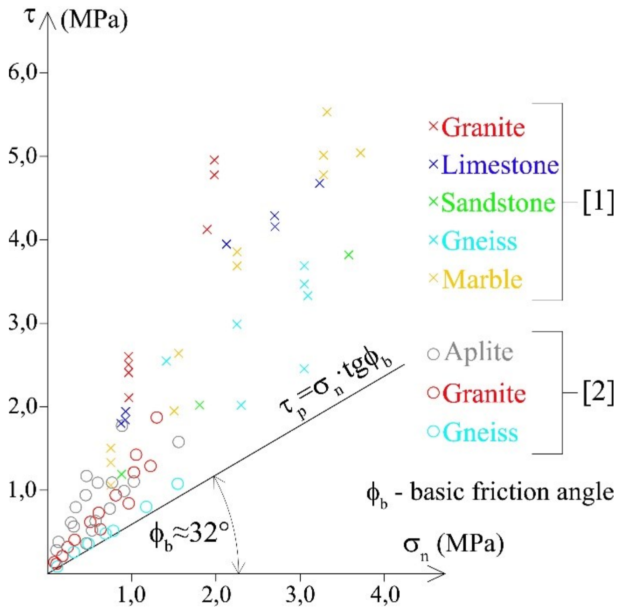

Figure 1. Values of peak shear strength for natural unfilled rock joints in different rocks with ϕ_b≈32° which are measured in CNS direct shear test. Adapted from [1, 2]

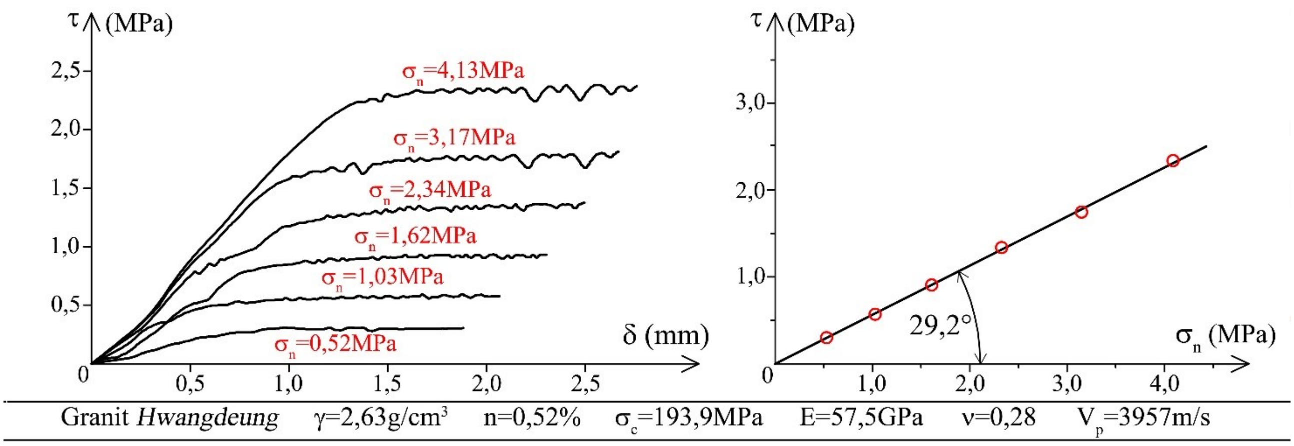

Figure 2. Results of CNS direct shear tests with different values of normal stress during shearing for planar, smooth and unfilled rock joints in granite Hwangdeung. Left: shear curves. Right: linear MC failure envelope. Adapted from [3]

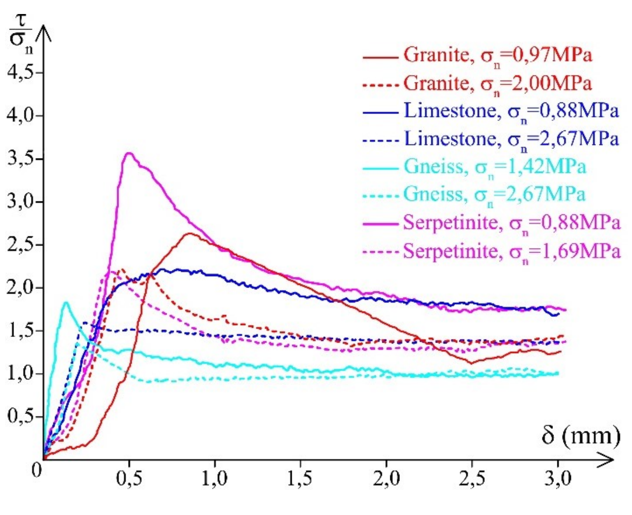

Figure 3. Normalized shear curves for nature unfilled rock joints in different rocks recorded in CNS direct shear tests with different values of normal stress during shearing. Adapted from [1]

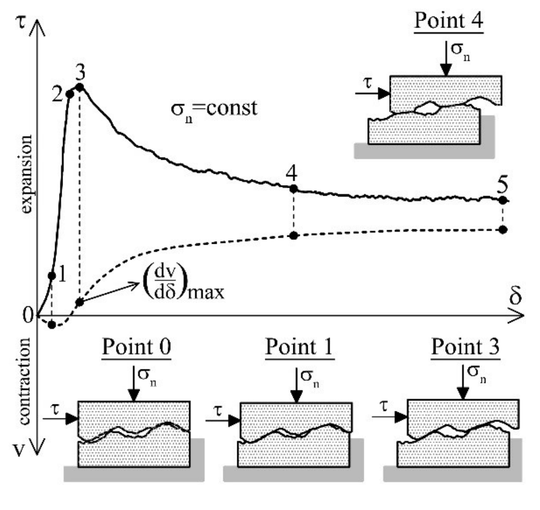

Figure 4. Typical shear curves and dilation curves for nature unfilled rock joints recorded in CNS direct shear test. Adapted from [2]

Fig. 4 shows a typical shear stress τ vs. shear displacement δ and vertical displacement v vs. shear displacement δ curves (dilation curves) for natural unfilled rock joints recored in CNS direct shear tests. In general, it can be concluded that the peak shear strength of natural unfilled rock joints (unfilled rock joints with a greater or lesser degree of surface roughness) can be considered a function of three variables: normal stress σ_n, basic friction angle ϕ_b and peak dilation angle ψ_{max}. Unlike the basic friction angle, which can be considered constant for the analyzed rock type, the peak dilation angle ψ_{max} is not constant. The value of this angle depends on the level of normal stress σ_n during shearing as well as the joint surface roughness, the presence of material bridges and their configuration, and rock compressive strength at the joint surface. Also, the peak dilation angle is scale dependent, i.e., the peak dilation angle, as well as the peak shear strength of natural unfilled rock joints, depends on the scale effect.

2.2 Rock compressive strength of the joint surface

Numerous experimental studies of natural unfilled rock joints have clearly shown that their shear strength depends on the rock compressive strength at the joint surface, i.e., the compressive strength of the bumps, ripples, and undulations on the rock joint surface (joint walls). These surface irregularities are given the general term asperities. The strength of the asperities is extremely important because they represent the sites (locations, points) of a stress concentration during shearing. The well-known effects of dilation and the effect of interlocking, which increase the shear strength of rock joints, occur (are pronounced) when asperities have more significant strength. In this case, the shearing process takes place over the asperity surface. However, if the strength of the asperities is lower, during shearing the asperities progressively shear off. In that case, the shearing process takes place through the asperities. For that reason, previously mentioned effects either do not exist or are very weakly expressed, which negatively affects the shear strength of rock joints.

The rock compressive strength of the joint surface can be equal to the compressive strength of intact rock. This may be the case with completely closed rock joints, and then one can talk about a fresh joint surface. However, due to the effects of weathering rock, the compressive strength of the joint surface is usually significantly less than the compressive strength of intact rock. Then one can talk about the weathered joint surface.

2.3 Level of normal stress during shearing

It is totally clear that the increase in normal stress σ_n causes an increase in the shear strength of rock joints. Shear tests carried out on planar, smooth, and clean unfilled rock joints under constant normal stress generally give a constant value of the ratio τ_p/σ_n, which is actually equal to $tg\phi_b$ at any level of normal stress. However, for naturally unfilled rock joints with a greater or lesser degree of joint surface roughness, the ratio τ_p/σ_n is not constant. The value of the peak friction angle of the joint surface ϕ_p, which actually represents the sum of the basic friction angle ϕ_b and peak dilation angle ψ_{max}, depends on the level of normal stress during shearing. The value of the peak dilation angle ψ_{max}, as well as the value of the peak friction angle of the joint surface ϕ_p, decreases with the increasing value of normal stress σ_n. Thus, with increasing normal stress during shearing, the peak friction angle of the joint surface ϕ_p tends to the value of the basic friction angle ϕ_b. Since the dilation is a direct consequence of the joint surface roughness, it means that the influence of this roughness on the shear strength of the rock joint is significant but at lower values of the normal stress σ_n. At higher values of normal stress, degradation or complete crushing of asperities occurs, which negatively affects the value of the peak dilation angle. All of the above directly leads to the conclusion that the failure envelope for natural, unfilled rock joints (to a varying degrees of joint surface roughness) is nonlinear.

When analyzing the influence of normal stress on the shear strength of natural unfilled rock joints (to a greater or lesser degree of joint surface roughness), it is useful to mention the effect of "pre-stress". Actually, this effect is identical to the well-known effect of soil pre-consolidation. The shear strength of any natural unfilled rock joints depends on the value of the normal stress σ_n during shearing. However, the value of the shear strength of the joint can also be significantly affected by whether the normal stress σ_n is at the same time the highest normal stress that has ever acted on this rock joint. In general, an increase in the value of the ''pre-stress'' coefficient leads to an increase in the value of the shear strength of rock joints.

2.4 Scale effect

The value of the peak friction angle of the joint surface ϕ_p represents the sum of the basic friction angle ϕ_b and the peak dilation angle ψ_{max}. Bandis [4] divided the peak dilation angle into two components. The first component, known as the geometrical asperity component, represents the average slope of the dominant (main) asperities on the joint walls relative to the shear direction. The second component, known as the mechanical asperity component, represents the strength of the dominant (main) asperities on the joint walls. According to the results of the carried out CNS direct shear tests, Bandis [4] concluded that the values of both components of the peak dilation angle decrease with the increasing dimensions of the examined rock joint, i.e., dimensions of the examined rock specimen with joint (scale effect). It is important to note that the basic friction angle ϕ_b is one of the few parameters in Rock Mechanics that does not depend on the scale effect. Shear tests carried out on planar, smooth, and clean unfilled rock joints of different dimensions under constant normal stress generally give a constant value of the ratio τ_p/σ_n, which is actually equal to $tg\phi_b$.

3 Mechanical behaviour of unfilled intermittent rock joints during shearing

The discontinuity of natural unfilled rock joints by material bridges or some healed sections may have a significant influence on their shear strength. It is totally clear that the presence of material bridges increases the shear strength of rock joints in proportion to the material bridge ratio ξ_b (material bridge area divided by total joint surface area). Except for the ratio ξ_b and configuration of material bridges also may have very significant influence on the shear strength of intermittent rock joints. The configuration of material bridges implies their shape, number, size, mutual distance, and orientation relative to the shear direction.

The mechanical behaviour of intermittent rock joints during shearing is very complicated and insufficiently investigated. So far, this type of mechanical behaviour has been commonly investigated in direct shear tests carried out on specimens of rock-like material (for example, gypsum) with intermittent joints which are easily formed during the pouring of the specimens. If specimens of intact rock material are used in direct shear tests, the intermittent joints are formed by cutting the specimens in a controlled manner (in certain zones and at a certain angle) with a diamond tile saw. Also, for the analysis of the mechanical behaviour of intermittent rock joints during shearing, numerical simulations of direct shear tests were performed by different researchers using sophisticated software.

Probably one of the most extensive and significant experimental investigations of the mechanical behaviour of unfilled intermittent rock joints during shearing was performed by German engineers Gehle and Kutter [5] from Rohr-University. They prepared more than 130 prism-shaped specimens with intermittent joints and tested them in the CNS displacement-controlled direct shear test (σ_n=1.0MPa, v=2mm/s). They analyzed different configurations of material bridges and their influences on the mechanical behaviour of joints during shearing (Fig. 5). The majority of the specimens tested were made from a gypsum + water mixture that was poured into a steel mould with internal dimensions of B/L/H=5cm/25cm/15cm. The rest of the tested specimens consisted of natural specimens of limestone with the same dimensions as gypsum (artificial) specimens. Shear (horizontal) displacement δ was applied to all tested specimens at a same controlled speed, and the values of generated shear stress τ and vertical displacement v were recorded. Fig. 6 shows results which Gehle and Kutter [5] recorded in two selected direct shear tests, i.e., for two selected artificial specimens.

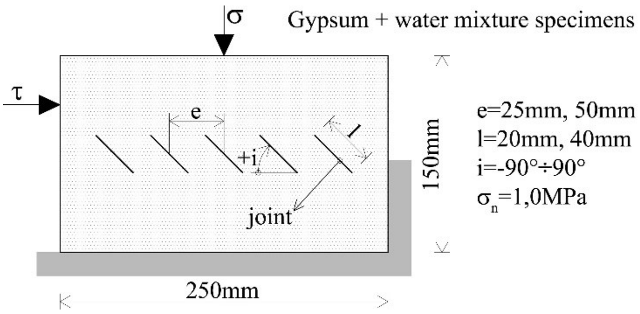

Figure 5. Geometrical characteristics of used specimens with intermittent joints. Adapted from [5]

First of all, according to the presented results, it can be concluded that the mechanical behaviour of an unfilled intermittent rock joint during shearing is very complicated. In the initial phase of the test, the increase in shear displacement δ is followed by an intense increase in shear stress τ. With some minor deviations, it can be considered that the shear stress is linearly related to shear displacement. The proportionality of this relationship corresponds to the stiffness of material bridges. At one moment, it comes to a sudden drop in shear stress (shear resistance). At that time, the initial joints are completely interconnected by cracks formed through material bridges. Actually, at that time, continuous (non-intermittent) joints were formed in the tested specimens. The geometric characteristics of the formed continuous joints depend on the configuration of the initial joints and material bridges. With a further increase in the shear displacement, the mechanical behaviour of the tested joints corresponds to the mechanical behaviour of rock joints with a greater or lesser degree of surface roughness during shearing. In this phase of the test, dilation is very pronounced.

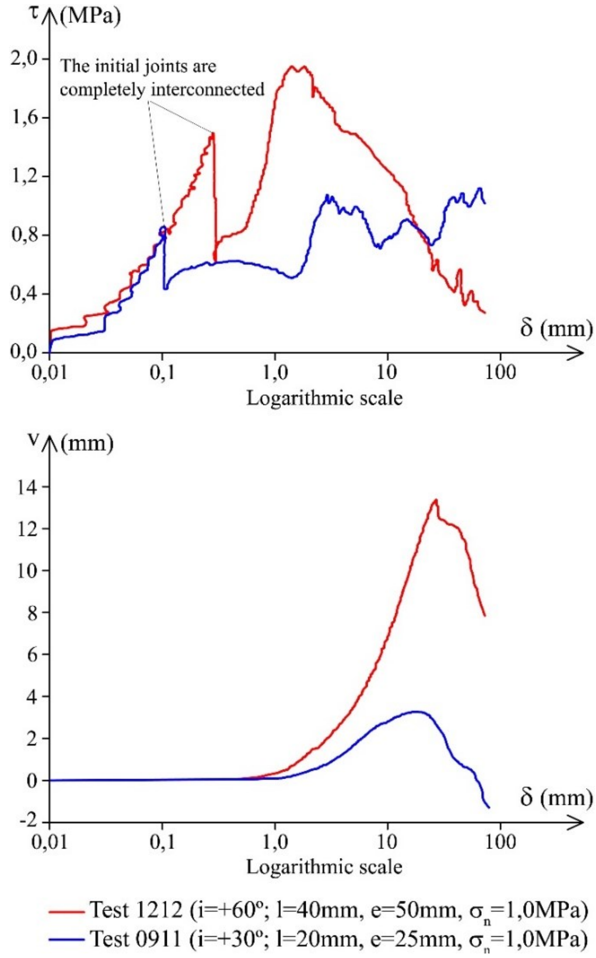

Figure 6. Examples of results of two different tests (σ_n=1MPa). Above: shear. Below: dilation curves. Adapted from [5]

It can be seen from Fig. 6 that a large horizontal displacement u_{max}=65mm is applied to the tested specimens. This displacement is actually 26% of the length of the shear plane and it is significantly larger than the initial distance between the individual joints. However, in both cases, the residual shear strength of the intermittent joints was not reached. For example, Grasselli [1] recorded that the shear displacements of approximately 3% of the shear plane length were sufficient to generate residual shear strength of the unfilled rock joints with pronounced joint surface roughness. Gehle and Kutter [5] claim that the shear displacement of 65mm is sufficient to record all of the significant shear behaviour characteristics for tested specimens.

Summarizing the results of the performed investigations, Gehle and Kutter [5] defined some main principles of the mechanical behaviour of unfilled intermittent rock joints during shearing. As a final result of the research, regardless of the numerous and significant differences in the recorded results of individual experiments, the authors proposed an idealized shear curve that describes the mechanical behaviour of unfilled intermittent rock joints during shearing under constant normal stress (Fig. 7). As can be seen, Gehle and Kutter [5] divide the mechanical behaviour of this type of rock joint during shearing into three phases.

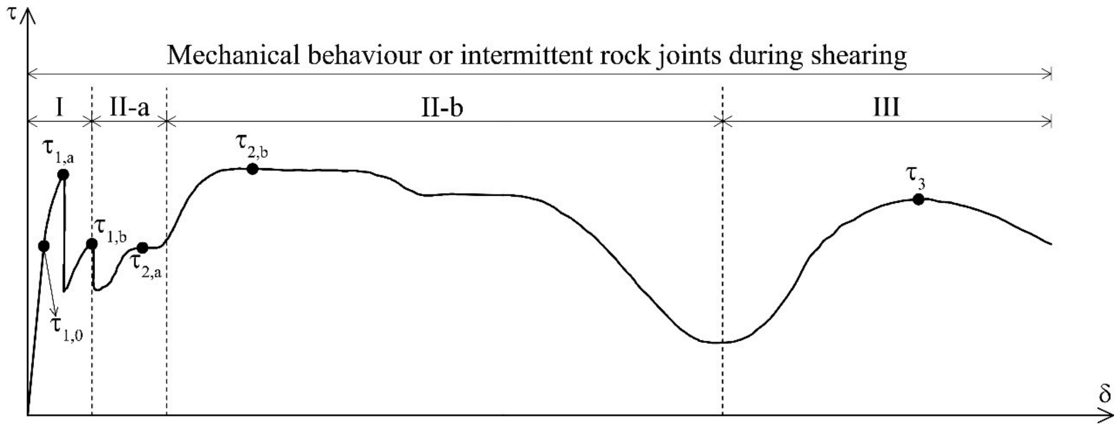

Figure 7. Idealized shear curve for unfilled intermittent joints during shearing under constant normal stress. Adapted from [5]

Since the stiffness of material bridges is many times greater than the stiffness of individual joints, in the first phase the increase in shear displacement causes a concentration of the shear stress on these bridges. Elastic deformations are only recorded, and it can be considered that the measured shear stress τ is linearly related to the applied shear displacement δ. However, fractures of material bridges occur when the concentrated shear stresses reach the value of the shear strength of material bridges (shear strength of intact rock). Formed cracks pass through the material bridges and connect the initial joints. It is very important to note that these fractures occur successively. The appearance of the first fracture or the first crack passing through any material bridge actually represents the beginning of the nonlinear mechanical behaviour of unfilled intermittent joints during shearing. At that time, the registered value of the shear stress is $\tau_{1.0}$. In some specimens at that time, but in most specimens at a slightly higher value of the applied shear displacement (measured shear stress is $\tau_{1,a}$), several cracks have already formed through the material bridges. These cracks lead to a sudden drop in the shear strength of intermittent joints. After that, the shear strength of the joints usually slightly increases until the shear stress $\tau_{1,b}$ is reached. Then it usually comes to a sudden drop in the shear strength of the joints. This drop actually means that all the initial joints are finally connected.

During the first phase of the shearing, continuous (non-intermittent) joints with more or less a "saw-tooth" surface were formed in the tested (sheared) specimens with $i\neq0$. Between "saw-tooth" joint walls, relatively large cut-out fragments of rock-like (rock) material were trapped. In the tested specimens with i=0 during the first phase of the shearing, continuous joints with more or less planar and smooth surfaces were formed, which primarily depends on the length and mutual spacing of the initial joints.

After the first phase, in the shearing process of unfilled intermittent joints, the second phase occurs. It lasts much longer, i.e., it covers a significantly larger shear displacement interval compared to the first phase. Generally, the second phase of the shearing process can be divided into two sub-phases. During the first sub-phase (II-a), an intensive increase in dilation angle is recorded. However, it is not followed by an equally intensive and pronounced increase in the shear strength of the tested joints. The reason for this is the small contribution of pure (physical) friction between the ''saw-tooth'' joint walls and their shear resistance angle. The initial joints are open, so the contact between their walls is realized only indirectly through the cut-out fragments of rock-like (rock) material. As the shear displacement increases, these cut-out fragments rotate. It is known that the friction during rotation is many times less than during sliding. The maximum measured value of the shear stress in this sub-phase is $\tau_{2,a}$.

In the second sub-phase (II-b) of the shearing process, with the increase of shear displacement δ direct contact between the "saw-tooth" joint walls is gradually realized. For that reason, the shear strength of tested (sheared) joints usually increases. At the moment of closing the initial joints, the measured value of the shear stress is $\tau_{2,b}$. As the shear displacement increases, the dilation stops and the shear resistance is solely based on the friction between the joint walls. The end of the second sub-phase was marked by a sudden drop in the shear strength of tested joints followed by intense contraction of the tested specimens. The contraction is the result of the degradation and crushing of cut-out fragments of rock-like (rock) material that are "trapped" between the joint walls.

Finally, at relatively large values of the applied shear displacement, the third phase of the shearing process occurs. In this phase, the peak shear strength of tested (sheared) joints τ_3 is predominantly conditioned by the shear resistance angle of degraded and crushed material that fills the space between joint walls, i.e., fills the space of formed shear zones in the sheared specimens. The width of the shear zone primarily depends on the geometric characteristics of the initial joints. It is useful to note that for some sheared specimens, this shear strength was the maximum shear strength that was recorded during the whole shearing process. At this stage, small or negligible changes in the volume of the sheared specimens were registered.

Each of the previously described phases or sub-phases of the mechanical behaviour of unfilled intermittent joints during shearing has its characteristic peak shear strength. It is important to note that each of these so-called phase peak shear strengths (except $\tau_{1.0}$) in some sheared specimens actually represented the maximum measured value of shear strength during shearing under constant normal stress. For example, in all sheared specimens with horizontal initial joints (i=0), the highest measured value of shear strength in the whole shearing process was $\tau_{1,a}$. However, in all sheared specimens with vertical initial joints (i=90º), the highest measured value of shear strength in whole shearing process was $\tau_{2,b}$. Generally, in the case of sheared specimens with a negative initial joint inclination (-90º<i<0º), the highest measured value of shear strength in the whole shearing process was $\tau_{2,a}$. In sheared specimens with positive initial joint inclination (0<i<90º) and depending on their mutual distance, the highest measured value of shear strength in whole shearing process was often $\tau_{2,b}$, then τ_3 and rarely $\tau_{1,a}$. In general, in the performed direct shear tests under constant normal stress, the highest shear strengths were measured on the specimens with a negative initial joint inclination, which is a consequence of very pronounced dilation in these specimens.

After Gehle and Kutter [5], Zhang et al. [6] carried out significant research on the shear strength of intermittent rock joints. Numerically, using the FE method with the help of appropriate software, he simulated the shear of rock-like specimens with intermittent joints (jointed rock-like specimens) in a CNS displacement-controlled direct shear test (σ_n=0,15MPa, v=2mm/s). The stress-strain behaviour of used rock-like material is defined by appropriate constitutive relations for a finite element under uniaxial compressive stress and tensile stress (Fig. 8.). The values of the used mechanical parameters for these constitutive relations are also given in Fig. 8. Two groups of jointed rock-like specimens were tested. In the first group of specimens, all initial joints are horizontal i=0 (Fig. 8). Their width b was varied, i.e., the value of material bridge ratio ξ_b was varied. In the second group of specimens, the total number, width, and mutual distance of initial joints were constant but their slope relative to the shear direction was varied (Fig. 8).

The peak shear strength of the first group of jointed rock-like specimens was found to increase almost linearly with increasing width of initial joints, i.e., with increasing value of material bridge ratio ξ_b. However, for the second group of jointed rock specimens, measured values of the shear strength are highly non-linearly related to the values of angle i (Fig. 9). Generally, the peak shear strength of intermittent rock joints in the case of a negative value of angle i is higher than those in the case of a positive value of angle i.

Sarfazi and Schubert [7] also carried out experimental research on the mechanical behaviour of unfilled intermittent rock joints during shearing. They prepared two groups of nine prism-shaped rock-like specimens with intermittent joints and tested them in the CNS displacement-controlled direct shear test (σ_n=1.0MPa, v=2mm/s). All specimens tested were made from a gypsum + water mixture that was poured into a steel mould with internal dimensions of B/L/H=15cm/15cm/15cm. They analyzed different configurations of material bridges and their influences on the mechanical behaviour of joints during shearing. All jointed rock-like specimens had only horizontal initial joints i=0°. All jointed rock-like specimens from the same group had an identical value of material bridge ratio ξ_b. The shape, size, and mutual distance of the horizontal initial joints were varied. Research carried out by Sarfazi and Schubert [7] showed that from the aspect of the shear strength of intermittent rock joints, the best configuration of material bridges is a configuration with a small number of larger material bridges that are elongated in the shear direction.

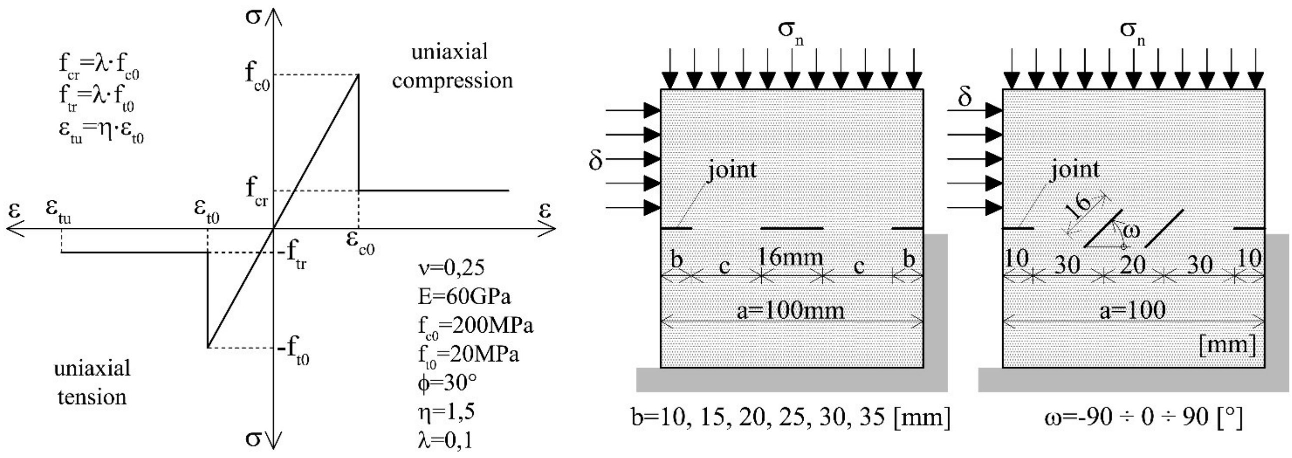

Figure 8. Left: the stress-strain behaviour of used rock-like material. Middle: first group of tested jointed rock-like specimens. Right: second group of tested jointed rock-like specimens. Adapted from [6]

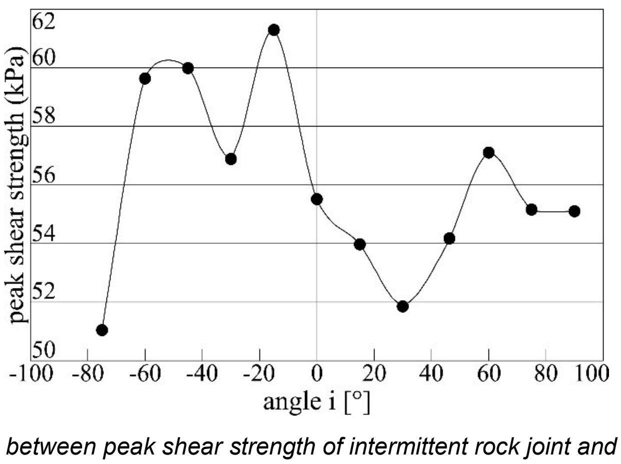

Figure 9. The relationship between peak shear strength of intermittent rock joint and angle I. Adapted from [6]

Fan et al. [8] also carried out experimental research on the mechanical behaviour of unfilled intermittent rock joints during shearing. They prepared six prism-shaped rock-like specimens with intermittent joints and tested them in the CNS displacement-controlled direct shear test (σ_n=2.5MPa, v=2mm/s). All specimens tested were made from a gypsum + water mixture that was poured into a steel mould with internal dimensions of B/L/H=15cm/15cm/3cm. They analyzed different configurations of material bridges and their influences on the mechanical behaviour of intermittent joints during shearing. Namely, all jointed rock-like specimens had three initial joints of the same length (L=20mm) and the same value of angle *i*. This angle was set from 0° to 150° with an interval of 30°. For tested rock-like specimens with horizontal initial joints, a brittle-plastic mechanical behaviour was obtained with a clearly expressed value of the peak and residual shear strength. However, for tested rock-like specimens with inclined initial joints, very complicated mechanical behaviour was obtained (Fig. 10). It goes through three phases. Three characteristic values of the shear strength of these jointed rock-like specimens are recorded: first peak shear strength, second peak shear strength, and residual shear strength. Between the first and second peak shear strength, a sudden drop in shear strength was registered. This drop occurs when the cracks that pass through the material bridges connect all initial joints, i.e., when a continuous (non-intermittent) joint with more or less "saw-tooth" surface is formed. Further, the shear strength increases from the first to the second peak value because the shear takes place more or less "saw-tooth" joint surface. A new sudden drop in shear strength occurs when the generated contact stresses cause degradation and crushing of main joint asperities. After that, with greater or lesser fluctuations, the shear strength of the rock joint tends to a residual value. To reach this value of shear strength, it was necessary to apply a shear displacement to the tested rock-like specimens, which is approximately 30% of the specimen width.

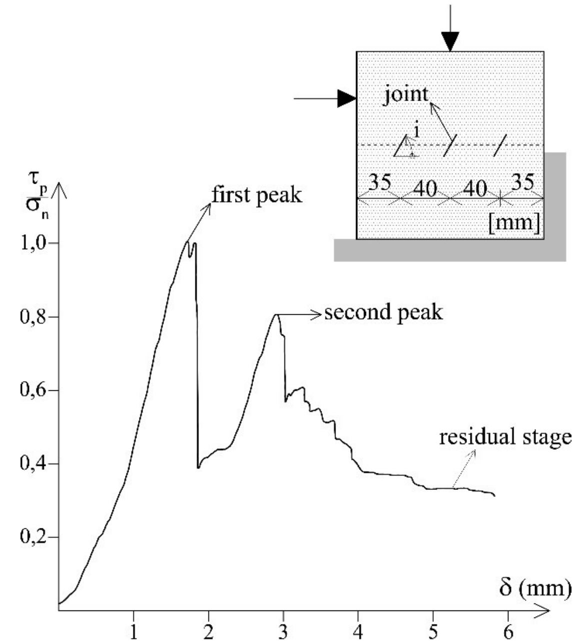

Figure 10. Typical shear curve for jointed rock-like specimens with inclined initial joints. Adapted from [8]

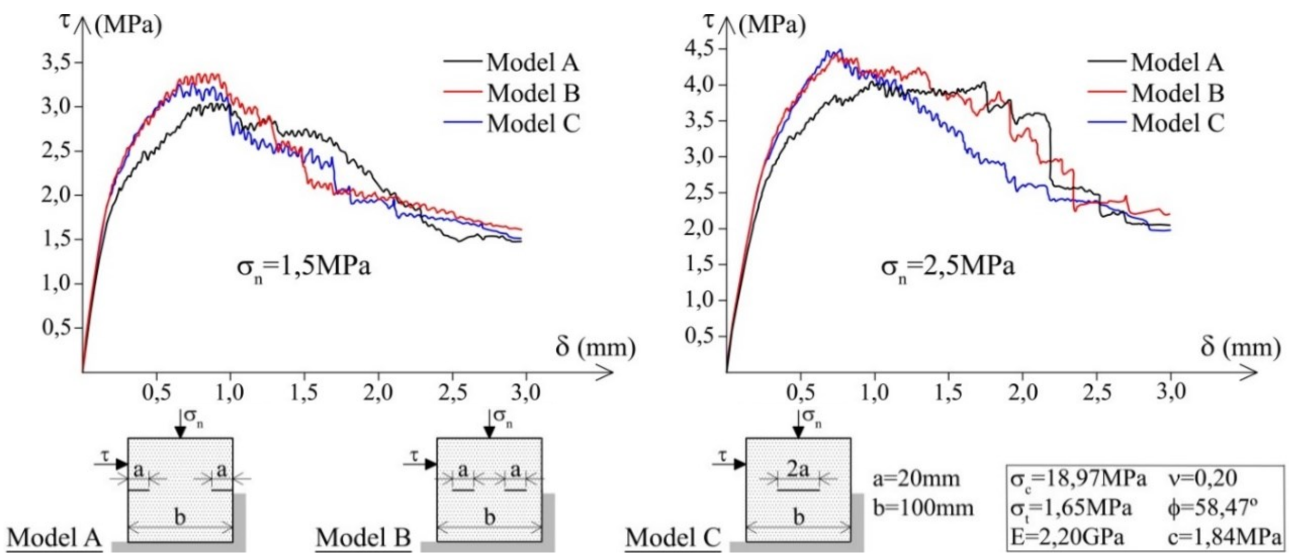

Figure 11. Shear curves for unfilled intermittent rock joints with different configurations of horizontal initial joints. Adapted from [8]

Lin et al. [9] performed a numerical analysis of the mechanical behaviour of unfilled intermittent rock joints during shearing under constant normal stress using the software PFC2D (Particle Flow Code-2D). Intermittent joints with three different configurations of horizontal initial joints (three different models) but with the same value of the material bridge ratio ξ_b were analyzed. It can be said that for all tested models, it is typically plastic mechanical behaviour with linear-elastic, nonlinear-plastic, peak, strain softening and residual stress phase (Fig. 11). The shear displacement that corresponds to the beginning of the nonlinear-plastic phase increases with an increase in the normal stress. For model C, i.e., for a model with both-side-end initial joints, the lowest value of peak shear strength was recorded. The residual shear strength is achieved at a shear displacement that is approximately 3% of the shear plane length. So, to achieve residual shear strength in the case of intermittent rock joints with horizontal initial joints, it is necessary to apply significantly less shear displacement compared to the case of intermittent rock joints with inclined initial joints.

4 Conclusion

Previously presented research has shown that the mechanical behaviour of unfilled intermittent rock joints during shearing is very complicated. As assumed, it is conditioned by the number, size, shape, mutual distance, and orientation of the material bridges. From the aspect of the shear strength of intermittent rock joints, the best configuration of material bridges is a configuration with a small number of larger material bridges that are elongated in the shear direction. However, the analysis of the results of the previous research came to the conclusion that the slope of the initial joints in relation to the shear direction has a particularly significant influence on the mechanical behavior of the intermittent rock joints.

If the initial joints are horizontal, i.e., parallel to the shear direction, the mechanical behaviour of unfilled intermittent joints during shearing is plastic with linear-elastic, nonlinear-plastic, peak, strain softening and residual stress phase. If the initial joints are inclined, their mechanical behaviour during shearing is very complicated.

This behaviour goes through three phases, and each phase has its own peak shear strength. Which of these three shear strengths will be the largest depends on the orientation and slope of the initial joints in relation to the shear direction. The lowest values of the peak shear strength are obtained for intermittent rock joints with both-side-end initial joints. To achieve residual shear strength in the case of intermittent rock joints with horizontal initial joints, it is necessary to apply significantly less shear displacement compared to the case of intermittent rock joints with inclined initial joints.

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