The importance of accounting for matedness when predicting the peak shear strength of rock joints

F Ríos-Bayona¹, E Andersson¹, F Johansson¹, D Mas Ivars¹,²
¹ KTH Royal Institute of Technology, Department of Civil and Architectural Engineering, Division of Soil and Rock Mechanics, Stockholm, Sweden
² SKB Swedish Nuclear Fuel and Waste Management Co, Solna, Sweden
frrb@kth.se

Abstract. The contribution from both surface roughness and matedness in the peak shear strength of rock joints is not yet well understood. To be able to account for the influence of matedness on the peak shear strength of rock joints, both surface roughness and aperture need to be considered. Technical developments over the past few decades have shown that both surface roughness and aperture can be accurately measured using optical scanning. This technique has been utilized to account for surface roughness parameters in various shear strength criteria that assume a perfect match between joint surfaces. This paper investigates and compares the capabilities of two shear strength criteria to predict the peak shear strength of rock joints with different matedness. The analysis performed shows that both approaches have their strengths and limitations. For instance, accounting for the matedness of unmated rock joints based on their surface aperture gives better predictions of the peak shear strength. On the other hand, accounting for shearing failure mode becomes relevant at high normal loads. A possible way forward to reduce the limitations of these criteria could be to combine their strengths.

1. Introduction
In recent decades, various empirical and analytical peak shear strength criteria have been developed to increase our understanding of how the matedness of a rock joint interacts with its surface roughness and contributes to the peak shear strength [1-5]. Nevertheless, the uncertainties associated with this parameter are still significant. To be able to explicitly account for the influence of matedness on the peak shear strength of rock joints, both surface roughness and aperture need to be considered. This is possible in Barton and Choubey’s [3] empirical peak shear strength criterion when the $J_{RC}$ parameter is estimated based on tilt tests. However, the influence of roughness and matedness cannot be separated when using only $J_{RC}$. Zhao [4, 5] revised Barton and Choubey’s criterion and separated the influence of matedness from the roughness of a rock joint by introducing the $J_{MC}$. One limitation of Zhao’s approach is that the estimation of the $J_{MC}$ parameter is mainly performed by visual inspections of the percentage of the rock joint surface in contact.

Over the past few years, technical developments have shown that both surface roughness and aperture can be accurately measured by surveying the rock joint surfaces with high-resolution optical scanning [6-9]. For instance, this technique has been used to obtain surface roughness parameters in three dimensions, which have been integrated in various peak shear strength criteria [10-14]. More recently, Casagrande et al. [15] tackled the question of how to predict the shear strength of rock joints from a semi-analytical perspective. They presented a promising approach that can predict the peak and residual shear strength of a rock joint without having full access to its surface. For instance, their
strategy uses the available information from a visible trace of a rock joint in two dimensions and a random field model to synthetically generate a large number of joint surfaces in three dimensions as a step in the prediction of its shear strength. Furthermore, Casagrande and co-workers themselves believe that this approach can potentially be applied to large scale rock joints in the field [15, 16]. However, a major limitation with the criterion developed by Casagrande et al. [15], and the aforementioned criteria based on optical scanning of the joint surfaces roughness [10-14], is that rock joint surfaces are assumed to be perfectly mated, which may not always be the case for natural, unfilled rock joints in the field.

Taking up the challenge of accounting for both the three-dimensional characteristics of surface roughness and the influence of matedness, Ríos-Bayona et al. [17] proposed a methodology that uses objective measurements of the average aperture between the joint surfaces of natural, unfilled rock joints to predict their matedness. The measured average aperture presented in their study was based on the high-resolution optical scanning of the joint surfaces. Furthermore, they integrated the proposed relationship between measured average aperture and matedness of natural, unfilled rock joints in a revised version of the peak shear strength criterion developed by Johansson and Stille [11, 17]. The main benefit of their approach is that aperture measurements can be directly obtained from both visible traces and core-drillings. Consequently, accounting for the matedness of natural, unfilled rock joints when predicting their peak shear strength can be possible under conditions with difficult access, such as the rock foundation under an existing dam. However, a major drawback of their revised criterion is that it assumes sliding along the active asperities as the predominant failure mode, which is not always the case.

This study investigates and compares the capabilities of the criteria developed by Casagrande et al. [15] and Ríos-Bayona et al. [17] to predict the peak shear strength of (1) a group of six tensile-induced perfectly mated rock joints, and (2) a group of ten natural, unfilled rock joints taken from existing rock joints adjacent to the foundation of two dams in Sweden. The calculated peak shear strength of the sixteen analysed rock joints with the applied criteria is compared with the measured peak shear strength obtained in the laboratory investigations.

2. Peak shear strength criteria

2.1. Peak shear strength criterion by Casagrande et al.

The criterion developed by Casagrande and co-workers [15] tackles the prediction of the shear strength of rock joints from a semi-analytical stochastic perspective. Their approach uses the accessible information from rock joint traces in 2D to create a large number of synthetic joint surfaces in 3D supported by a random field model. The output of this criterion is a shear strength distribution obtained by calculating the shear strength of all the generated synthetic joint surfaces in 3D. Furthermore, the shear strength of each joint surface is calculated by accounting for the contribution of all the active asperities that are mobilised during the shearing process. According to Casagrande et al. [15], rock joint dilation occurs along the steepest asperities in contact. These asperities are then sheared off and the applied load is redistributed to less steep asperities that will be sheared in a later step. This process continues until no more asperities are sheared. The main inputs needed to run the model are the scanned rock joint surface at a resolution that captures grain size [10, 18], the applied normal stress (σ_n), the values of internal cohesion (c_i) and friction angle (ϕ_i) of the intact rock material, the basic friction angle calculated for a dry and sawn surface (ϕ_h) and the applied shear direction (t).

In a first step, the re-generated rock joint surface is analysed and the values of apparent dip angle (θ') of all the asperities facing t are calculated using

\[ \cos(90° - \theta_i') = \frac{|n_i \cdot t|}{|n_i| \cdot |t|} \]  \hspace{1cm} (1)

where n_i is the unit normal vector of each element on the re-generated surface.

Taking the calculated values of θ' for the analysed rock joint surface, the model developed by Casagrande et al. [15] identifies the active asperities in contact by defining a loop through the variable θ. The loop starts from the maximum apparent dip angle in the shear direction measured on the joint surface (i.e., θ = θ_{max}) and progressively decreases towards zero with decrements of 0.1°. For each
defined value of $\theta$ in the loop, all asperities having a $\theta^* \geq \theta$ are considered active and can potentially be sheared. The loop on the variable $\theta$ stops when no more asperities are sheared. As a clarification, the criterion by Casagrande et al. [15] assumes that all asperities in the upper and lower surfaces are in contact during the computation (i.e., perfectly matched).

For each identified active asperity, the local horizontal forces required to slide on the asperity ($f_{\text{sliding},i}$) and to shear the asperity along the horizontal plane ($f_{\text{shear},i}$) are calculated. The expressions to obtain $f_{\text{sliding},i}$ and $f_{\text{shear},i}$ are given by

$$f_{\text{sliding},i} = f_{\text{local},i} \cdot \tan(\phi_b \cdot \theta_i^*),$$

and

$$f_{\text{shear},i} = A_{ip} \cdot (c_i + \sigma_{\text{local},i} \cdot \tan(\phi_i)),$$

where $f_{\text{local},i}$ is the vertical force acting on each active asperity and is calculated by dividing the total applied vertical force by the total number of active asperities ($N_{cf}$) and $\sigma_{\text{local},i}$ is the normal stress acting on each active asperity, which is calculated by dividing $f_{\text{local},i}$ by the area of the active asperity projected on the horizontal plane ($A_{ip}$).

Local shearing of an active asperity occurs if $f_{\text{shear},i} \leq f_{\text{sliding},i}$. When shearing occurs, a surface modification along a plane with an inclination of $\theta$ is imposed in the model. The number of iterations continues until sliding is the governing failure mode along the active asperities. Consequently, the loop stops the iteration at a certain value of $\theta$ and calculates the peak shear force ($f_{\text{peak}}$) and peak shear stress ($\tau_p$) as

$$f_{\text{peak}} = \sum_{i=1}^{N_{cf}} f_{\text{sliding},i} = \sum_{i=1}^{N_{cf}} f_{\text{local},i} \cdot \tan(\phi_b \cdot \theta_i^*),$$

and

$$\tau_p = \frac{f_{\text{peak}}}{A}.$$

where $A$ is the total area of the sample.

2.2. Peak shear strength criterion by Ríos-Bayona et al.

The criterion developed by Ríos-Bayona and co-workers [17] is a revised version of the criterion originally developed by Johansson and Stille [11]. The main difference between the original and revised versions is that the revised criterion uses objective measurements of the average aperture of natural, unfilled rock joints based on high-resolution optical scanning to account for matedness as a step in the calculation of their peak shear strength.

The revised criterion by Ríos-Bayona et al. is based on the adhesion theory of friction and assumes that sliding is the governing failure mode along the active asperities in contact. Furthermore, the criterion assumes that surface roughness can be described with self-affine fractal theory as a superposition of different asperities at different scales [19-21]. The criterion accounts also for the variation of the number and size of the active asperities in contact during the shearing process due to scale and matedness. The criterion can be expressed as

$$\phi_p = \phi_b + i_n,$$

in which

$$i_n = \tan\left(\frac{L_n}{L_g} \right) \left(\frac{L_n}{L_g}\right)^{k(h-1)}$$

and

$$i_g = \theta_{\text{max}} - 10 \left(\frac{\log_{10} \sigma_{\text{ci}} - \log_{10} A_0}{C}\right) \theta_{\text{max}},$$

where $\phi_p$ is the peak friction angle, $i_n$ and $i_g$ are the dilation angle at sample and grain scale, respectively. $\sigma_{\text{ci}}$ is the uniaxial compressive strength of the intact rock, $A_0$ and $C$ are the maximum possible contact area ratio and a roughness parameter, respectively, calculated by best-fit regression
analysis [7]. \( L_n \) is the length of the sample, \( L_g \) is the scale of the asperities associated with grain size, \( H \) is the Hurst exponent and \( k \) is the matedness constant.

The parameter \( k \) describes how the number and inclination of the active asperities in contact during the shearing process (i.e., \( \iota_n \)) vary proportionally with the measured average aperture (\( \alpha \)) between the joint surfaces of natural, unfilled rock joints. The value of \( k \) varies between 0 for a perfectly mated rock joint (i.e., \( \alpha = 0 \)) and 1 for a totally mismatched rock joint (i.e., maximum possible measured \( \alpha \) due to dilation along the active asperities). The equation for \( k \) is given by

\[
k = \frac{\log_{10}(\frac{2a}{\tan(\iota_n)}) - \frac{\log_{10}L_{asp,g}}{2}}{\frac{\log_{10}L_n}{2} - \frac{\log_{10}L_g}{2}}, \tag{9}
\]

where \( L_{asp,g} \) is the average length of the asperities in contact at grain size.

The value of \( \alpha \) can be obtained by superposing the lower and upper digitised surfaces and calculating the difference in elevation between points with same \( x \)- and \( y \)-coordinates. Lower and upper digitised surfaces need to be scanned in the same global reference system in order for them to be compatible [17]. The calculation of \( \alpha \) can be expressed as

\[
\alpha = \frac{\sum_{i=1}^{N_x} \sum_{j=1}^{N_y} (z_{i,j}^{upper} - z_{i,j}^{lower})}{N_x \cdot N_y}, \tag{10}
\]

where \( z_{i,j}^{upper} \) and \( z_{i,j}^{lower} \) are the elevation of two points with same \( x \)- and \( y \)-coordinates situated in the upper and lower digitised surfaces, respectively. \( N_x \) and \( N_y \) are the number of coordinate points over a digitised rock joint surface in the \( x \)- and \( y \)-directions, respectively.

Therefore, the criterion developed by Ríos-Bayona et al. [17] uses the scanned surfaces and equation (10) to objectively calculate \( \alpha \) and determines \( \iota_n \) and \( k \) by performing an iterative process with equations (7), (8) and (9) to calculate \( \phi_p \) with equation (6).

3. Methodology

3.1. General methodology

The aim of the study presented in this paper is to investigate the capability of the criteria by Casagrande et al. [15] and Ríos-Bayona et al. [17] to predict the peak shear strength of rock joints with different surface matedness. First, these two criteria were applied to calculate the peak shear strength of a group of six tensile-induced perfectly mated rock joints taken from the Flivik quarry in Sweden. These rock joint samples consisted of grey coarse-grained granite and were scanned and tested in the laboratory by Johansson [22]. Furthermore, a second group of eight natural, unfilled rock joints obtained by over-drilling through an existing rock joint adjacent to the foundation of the Storfinnforsen buttress dam were analysed in this study. These rock joint samples consisted also of grey coarse-grained granite and were scanned and tested in the laboratory by Ríos-Bayona et al. [17]. Finally, two rock joint samples obtained by over-drilling through an existing rock joint adjacent to the foundation of the Långbjörn concrete dam were analysed. The intact rock beneath the dam’s foundation consists of grey coarse-grained granite. These two samples were scanned and tested in the laboratory by Johansson [23]. All samples were tested in the laboratory under constant normal load (CNL) conditions [17, 22, 23].

3.2. Input data

The joint surfaces of all analysed rock joint samples were scanned with high-resolution optical scanning to capture the surface roughness prior to the conducted direct shear tests. Lower and upper parts were first scanned separately keeping the same global reference system. Furthermore, lower and upper parts were then put together and scanned again in order to capture the degree of contact between the parts. All joint surfaces were re-generated with a resolution of 0.3 by 0.3 mm, which is assumed to be appropriate to capture \( L_g \) [10, 18]. The values of \( \Theta^* \) of all asperities facing \( t \) were determined using equation (1). This information was used to calculate the parameters \( \Theta_{max}^* \), \( C \) and \( A_g \) by best-fit regression analysis according to the procedure described by Grasselli [7].
samples was analysed after superposing lower and upper digitized surfaces. The values of \( \alpha \) were calculated using equation (10). Table 1 presents the size of the analysed rock joint samples from Flivik (F1 to F6), Storfinnforsen (S1 to S8) and Långbjörn (L1 and L2) together with the applied \( \sigma_n \) in the shear tests, roughness parameters \( (A_0, C, \theta_{\text{max}}) \), measured values of \( H \) and \( \alpha \) and estimated values of \( k \) with equation (9) [17, 22, 23]. The values of \( k \) for the samples from Flivik in table 1 were taken from Johansson [22]. These values were based on the assumption that tensile-induced samples F1 to F6 had a perfect match between lower and upper surfaces and therefore \( k = 0 \).

The values of \( \phi_i \) obtained by tilt tests, values of \( \sigma_{ci} \), estimated using the Schmidt Hammer Index as suggested by Barton and Choubey [3] and estimated values of \( c_i \) and \( \phi_i \) are presented in table 2.

### Table 1. Size and values of applied \( \sigma_n, A_0, C, \theta_{\text{max}}, H, \alpha \) and \( k \) for the rock joint samples from Flivik [22], Storfinnforsen [17] and Långbjörn [23].

| \( L_n \) (mm) | Width (mm) | \( \sigma_n \) (MPa) | \( A_0 \) (-) | \( C \) (-) | \( \theta_{\text{max}} \) (°) | \( H \) (-) | \( \alpha \) (mm) | \( k \) (-) |
|-----------------|-------------|----------------------|--------------|-------|------------------|--------|----------------|---------|
| F1              | 61          | 61                   | 1.0          | 0.50  | 3.58             | 50.94  | 0.74           | 0.0     |
| F2              | 61          | 60                   | 1.0          | 0.55  | 4.04             | 53.95  | 0.79           | 0.0     |
| F3              | 61          | 59                   | 1.0          | 0.39  | 3.19             | 47.33  | 0.82           | 0.0     |
| F4              | 200         | 200                  | 1.0          | 0.55  | 4.29             | 58.80  | 0.81           | 0.0     |
| F5              | 200         | 200                  | 1.0          | 0.52  | 3.92             | 54.43  | 0.82           | 0.0     |
| F6              | 200         | 201                  | 1.0          | 0.58  | 4.23             | 54.19  | 0.83           | 0.0     |
| S1              | 137         | 190                  | 1.0          | 0.58  | 5.15             | 76.22  | 0.85           | 0.79    | 0.38  |
| S2              | 202.5       | 190                  | 1.0          | 0.50  | 6.15             | 67.50  | 0.82           | 0.87    | 0.47  |
| S3              | 213.0       | 189                  | 1.0          | 0.53  | 5.63             | 78.34  | 0.85           | 0.77    | 0.37  |
| S4              | 188.5       | 162.5                | 1.0          | 0.59  | 5.17             | 62.33  | 0.84           | 0.47    | 0.31  |
| S5              | 135         | 191                  | 1.0          | 0.60  | 5.63             | 74.67  | 0.79           | 0.65    | 0.39  |
| S6              | 185         | 177.5                | 1.0          | 0.60  | 6.60             | 76.41  | 0.79           | 0.81    | 0.47  |
| S7              | 195         | 191.2                | 1.0          | 0.52  | 5.34             | 71.72  | 0.84           | 1.28    | 0.49  |
| S8              | 126         | 178                  | 1.0          | 0.67  | 4.59             | 53.67  | 0.84           | 0.50    | 0.37  |
| L1              | 125         | 125                  | 0.85         | 0.29  | 7.82             | 45.74  | 0.83           | 0.91    | 0.66  |
| L2              | 240         | 240                  | 0.90         | 0.65  | 7.34             | 60.01  | 0.81           | 1.00    | 0.52  |

*Measured in the shear direction

### Table 2. Values of \( c_i, \sigma_{ci}, \phi_i \) and \( \phi_b \) estimated for the rock joint samples from Flivik [22], Storfinnforsen [17] and Långbjörn [23] (after Andersson [24]).

|                | Flivik     | Storfinnforsen | Långbjörn |
|----------------|------------|----------------|-----------|
| \( c_i \) (MPa) | 32.8       | 18.3           | 23.3      |
| \( \sigma_{ci} \) (MPa) | 197 | 110 | 140 |
| \( \phi_i \) (°) | 53.2 | 53.2 | 53.2 |
| \( \phi_b \) (°) | 31 | 31 | 31 |

### 4. Results
A comparison between calculated \( \phi_p \) with the criteria by Casagrande et al. [15] and Rios-Bayona et al. [17] and measured \( \phi_p \) in the laboratory for the analysed rock joint samples from Flivik [22], Storfinnforsen [17] and Långbjörn [23] is presented in figure 1a and b, respectively.

The values of \( \phi_p \) of all analysed samples calculated with the criterion by Casagrande et al. [15] in figure 1a were determined using equations (4) and (5) by Andersson [24] as part of his master thesis work. The values of calculated \( \phi_p \) for the tensile-induced samples from Flivik in figure 1b were determined by Johansson [22] using the criterion by Johansson and Stille [11] in equations (6) to (8) and assuming perfect match between the joint surfaces. The values of calculated \( \phi_p \) for the samples of Storfinnforsen and Långbjörn in figure 1b were determined by Rios-Bayona et al. [17] using equations (6) to (9), and accounting for their surface matedness based on their measured \( \alpha \).
Figure 1. Comparison between calculated and measured $\phi_p$ and obtained linear fit through regression analysis for the tensile-induced rock joint samples from Flivik [22] and the natural, unfilled rock joints from Storfinnforsen [17] and Långbjörn [23]: a calculated values of $\phi_p$ with the criterion by Casagrande et al. [15] (after Andersson [24]); b calculated values of $\phi_p$ with the criterion by Johansson and Stille [11] and the revised criterion by Ríos-Bayona et al. [17].

The results of $\phi_p$ calculated with the criterion by Casagrande et al. [15] for the tensile-induced samples from Flivik presented in figure 1a were in good agreement with the results from the direct shear tests in the laboratory. The calculated $\phi_p$ varied between 63.9° and 66.5°. Furthermore, the difference between calculated and measured $\phi_p$, expressed in absolute values, varied between 0.6° and 5.3°. The calculated values of $\phi_p$ for the natural, unfilled rock joint samples from Storfinnforsen with the criterion by Casagrande et al. [15] were in less good agreement with the results from the laboratory investigations. The values of calculated $\phi_p$ varied between 58.7° and 65.4°. As expressed in absolute values, the difference between calculated and measured $\phi_p$ varied between 0.2° and 13°. The calculated values of $\phi_p$ for the natural, unfilled rock joint samples from Långbjörn with Casagrande et al. [15] deviated from their measured shear behaviour in the laboratory. Samples L1 and L2 had a calculated $\phi_p$ of 55.9° and 58.2°, respectively. These values of calculated $\phi_p$ were 11.3° and 15.8° higher than the measured $\phi_p$ in the laboratory, respectively. Furthermore, the slope of the obtained linear fit through regression analysis, accounting for all samples, was 0.3.

The results of calculated values of $\phi_p$ presented in figure 1b for the tensile-induced samples from Flivik determined by Johansson [22] were in good agreement with the results from the laboratory. The calculated values of $\phi_p$ with this criterion varied between 64.4° and 67.8°. The difference between calculated and measured $\phi_p$, expressed in absolute values, varied between 0.2° and 2.7°. The calculated values of $\phi_p$ for the natural, unfilled rock joint samples from Storfinnforsen with the criterion by Ríos-Bayona et al. [17] were also in good agreement with the results from the laboratory. The calculated $\phi_p$ in these samples varied between 52.4° and 64.7°. The difference between measured and calculated $\phi_p$, as expressed in absolute values, varied between 0.4° and 9.3°. The calculated values of $\phi_p$ for samples L1 and L2 were 41.1° and 47.6°, respectively. These calculated values were in good agreement and deviated 3.5° and 5.2° from the measured $\phi_p$, respectively. The slope of the obtained linear fit through regression analysis, accounting for all samples, was 0.8.
5. Concluding remarks

This paper investigates and compares the capabilities of the criteria developed by Casagrande et al. [15] and Ríos-Bayona et al. [17] to predict the peak shear strength of rock joints with different joint surface matedness. Based on the comparison made between calculated and measured shear strength in figure 1a and b, it can be concluded that both criteria are able to predict well the peak shear strength of the tensile-induced perfectly mated rock joints from Flivik under the conditions tested in this study. However, the predicted values of peak shear strength of the natural, unfilled rock joints from Storfinnforsen and Långbjörn with Casagrande et al. [15] are in less good agreement and generally overestimate their measured shear behaviour in the laboratory. This is not surprising, since the criterion by Casagrande et al. [15] assumes a perfect match between the joint surfaces, which is not the case of samples S1 to S8, L1 and L2. On the other hand, the criterion by Ríos-Bayona et al. [17] is capable of accounting for the interaction between surface roughness and matedness of samples S1 to S8, L1 and L2 and predict a peak shear strength in good agreement with the one measured in the laboratory.

The criterion by Ríos-Bayona et al. [17] uses objective measurements of the average aperture between the joint surfaces of natural, unfilled rock joints derived from high-resolution optical scanning to estimate their matedness, as a step in the calculation of their peak shear strength. Aperture measurements can be obtained directly from visible rock traces or from core-drilling, which may make it possible to account for the matedness of natural, unfilled rock joints under conditions of difficult access (e.g., the rock foundation under an existing concrete dam). However, a major limitation of the criterion by Ríos-Bayona et al. [17] is that it assumes sliding along the active asperities as the predominant failure mode, which is not always the case.

The criterion by Casagrande et al. [15] is not yet developed to account for the matedness of natural, unfilled rock joints when predicting their peak shear strength. However, they present a promising approach that can predict both peak and residual shear strength of rock joints. The criterion by Casagrande et al. [15] uses also field information, such as visible rock traces in 2D, to synthetically generate a large number of joint surfaces in 3D supported by a random field model. As a result, the criterion calculates the peak and residual shear strength of all the joint surfaces synthetically generated. Furthermore, both sliding and shear failure modes are considered when computing the shear resistance of the active asperities.

Therefore, a possible way forward to reduce the limitations of the criteria by Casagrande et al. [15] and Ríos-Bayona et al. [17] could be to combine their strengths.

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