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Revisiting the shear strength criteria for rock discontinuities and the contact between rock and concrete for foundation assessment of concrete dams

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Abstract

The shear strength of rock discontinuities (joints, weak foliation, cleavage, and bedding surfaces) and concrete-rock contacts are the fundamental parameter to determine the stability of concrete dams. Despite the important role of in-situ and laboratory shear strength tests, these types of tests are not often conducted as part of dam projects, in most cases due to high costs, particularly in the case of in-situ tests. Therefore, dam engineers usually resort to published data from available literature and historical projects. This is often the case in early design stages of new dams or for the initial assessment of the safety review of existing dams. However, the most of the commonly used published literature such as Barton (1971), Coulson (1972) and Richards (1975) for the rock discontinuities and Zeigler (1972), Benson (1986) and EPRI (1992) for the concrete-rock contacts have only limited number of test data. In addition to this, EPRI (1992) as the most used historical database for the concrete dam projects only includes the results of two in-situ direct shear tests and the remaining data points had been obtained from laboratory tests. Given the constraints outlined regarding the current databases, this paper presents an extensive database for both rock discontinuities and the concrete-rock contacts. For the rock discontinuities, this database includes 309 datapoints for the basic friction angle of sedimentary, igneous, and metamorphic rocks. For the concrete-rock contacts, it focuses on both peak and residual shear strength of the contact, including 381 datapoints for the peak shear strength and 441 datapoints for residual shear strength. To the authors' knowledge, this database of shear strength of concrete-rock contacts is the most complete ever assembled. In this paper, a statistical analysis of the compiled databases will be presented to provide the industry with a reliable basis for assessing the shear strength parameters required for the dam foundations.

1. INTRODUCTION

Sliding along unfavorable oriented defects in the dam foundation and along the concrete-rock contact are two main potential failure modes for concrete dams. Therefore, determination of the shear strength of rock joints and the concrete-rock interface is fundamental in estimating the stability of the dam. For the rock joints, basic friction angle presents the frictional resistance of rock joint at its practically possible flatness, where the impact of visible asperities and undulation is negligible. This parameter, along with the roughness of joint surface, applied normal stress and the strength of joint surface (including the roughness), are the crucial inputs to estimate the shear strength of rough rock joints. For the concrete-rock contacts, applied effective normal stress, roughness of the contact, and strength of both concrete and rock foundation are the main parameters controlling the shear strength.

Despite the important role of in-situ and laboratory shear strength tests, these types of tests are not often conducted as part of dam projects, in most cases due to high costs, particularly in the case of in-situ tests. Therefore, dam engineers usually resort to published data from available literature and historical projects. This is often the case in early design stages of new dams or for the initial assessment of the safety review of existing dams. Considering the importance of developing more comprehensive and reliable database, an extensive database for both basic friction angle of rock joints and the peak and residual shear strength of concrete-rock contacts was gathered. This paper present the statistical analysis of these large databases. To the authors' knowledge, both databases are the most complete ever assembled.

2. BASIC FRICTION ANGLE OF ROCK JOINTS

Based on Patton (1966) criterion, the peak shear strength of rock joints (τ_p) is a function of basic friction angle (φ_B), asperity angle of joint surface roughness (i) and applied normal stress (σ_n), as follows:

$$\tau_p = \sigma_n \tan{(\varphi_b + i)} \tag{1}$$

Barton-Bandis failure criterion's initial format has the following form where the basic friction angle had been directly used in the formulation:

$$\tau_p = \sigma_n \tan{[\varphi_b + JRC Log_{10}\left(\frac{JCS}{\sigma_n}\right)]} \tag{2}$$

where JRC is Joint Roughness Coefficient, a coefficient representing the roughness of joint surface, and JCS is Joint wall Compressive Strength, which can be equal with UCS (Uniaxial Compressive Strength) of intact rock when the joint surface is fresh with no weathering. To take the impact of weathering and moisture into the consideration, φ_b has been replaced by φ_r(representing the friction angle of a flat and non-dilatant rock joint) in Equation (2), which is defined as follows (Barton & Bakhtar, 1977):

$$\varphi_r = (\varphi_b - 20) + 20(^r/_R) \tag{3}$$

where $^r/_R$ represents the ratio between the Schmidt hammer rebound number of wet and weathered joint surface over the rebound obtained from a dry and unweathered rock surface. It should be noted that value of φ_r in Equation (3) is different than the residual shear strength measured in laboratory or in-situ direct shear tests where the rock joint is not flat and presents dilation event at residual state.

The basic friction angle is mainly determined in laboratory by using tilt test. However, direct shear tests on the saw-cut samples have also been used to determine the basic friction angle on rock joints. There are various arrangements for tilt test based on the type of contact geometry, as discussed in ISRM suggested method (Alejano et al., 2018).

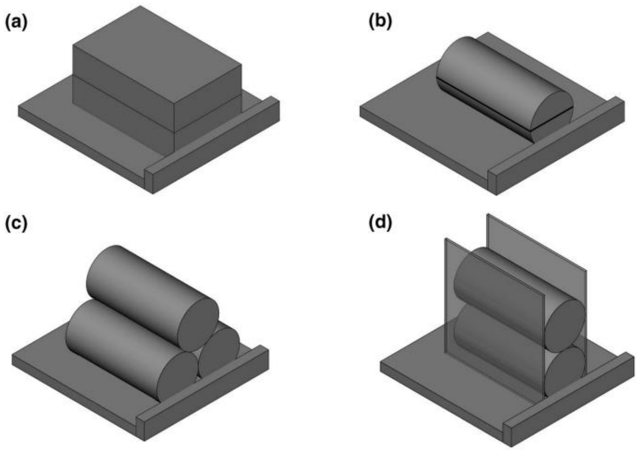

Figure 1. Tilt test with various contact geometry arrangement to measure the basic friction angle (Alejano et al., 2018).

Over the years several researchers such as Rupley & Lee (1962), Barton (1971), Coulson (1972), Barton (1973), Richards (1975), Barton (1976), Cruden & Hu (1988), Waltham (1994), Geertsema (2003) has published basic friction angle test results for various rock types. Alejano et al., (2012) reported a basic friction angle between 25 to 30 degrees for sedimentary rocks, and 30 to 35 degrees for metamorphic and igneous rocks based on data published by Barton (1973, 1976). These databases were usually limited to a small number of tests on some selected rock types.

To develop a more comprehensive database for the basic friction angle of rock joints, a total of 309 datapoints have been gathered from published literature on this topic. It should be noted that some of the presented datapoints the correspond to the average of several test results (the relevant literature only presented the statistical results), which means that the real number of test datapoints are more than 309.

Figure 2 presents the frequency analysis of the gathered data for all rock types. Based on this figure, approximately 42% of all gathered data have a basic friction angle in the range of 29° to 34°. This corresponds to an average basic friction angle of 31°, with a standard deviation of 4.87°.

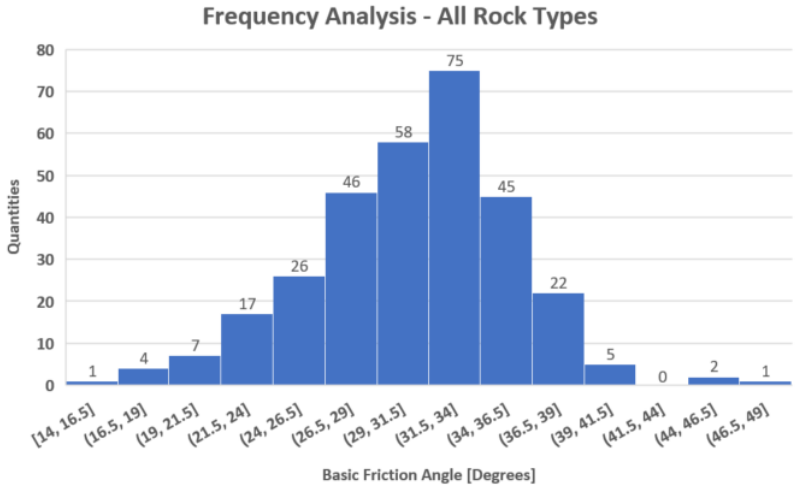

Figure 2. Frequency analysis the basic angle of friction of gathered data for all rock types.

The data collected were categorised into three groups based on sedimentary, metamorphic, and igneous rock types. The statistical analysis pertaining to these three groups of rock types can be found in Table 1 and Figures 3 to 5. The maximum recorded basic friction of 45° for sedimentary and igneous rocks in the gathered belongs to the graywacke sandstone and granite samples, respectively. For metamorphic rocks datapoints, a quartzite sample presented the maximum basic friction angle of 48°.

Table 1. Statistical characteristics of basic friction angle for various rock types

Rock Type	*Number of Datapoints*	*Minimum angle of friction (°)*	*Maximum angle of friction (°)*	*Average (°)*	*Standard Deviation (°)*
Sedimentary	133	14	45	31.3	4.8
Metamorphic	63	20	48	30.6	4.8
Igneous	113	17	45	30.8	4.9

Despite a relatively wide range of minimum and maximum values reported, the average and standard deviation of the measured basic friction angles exhibit a remarkable degree of similarity across all rock types. For example, within the datapoints pertaining to sedimentary rocks, the lowest recorded basic friction angle of 14° is associated with shale, suggesting the possibility that this shale might have been relatively soft. However, it is noteworthy that there is also a data point where shale exhibits a maximum and average basic friction angle of 36.8° and 30.6°, respectively. This discovery aligns with the findings of Byerlee (1978), who conducted an analysis on the peak and residual shear strength of rock joints using compiled data. Byerlee's work revealed that the average friction angle of rock joints appears to be only minimally influenced by the specific rock type.

3. SHEAR STRENGTH OF CONCRETE-ROCK CONTACT

As mentioned before, dam engineers usually resort to published data from available literature and historical projects. Dawson et al. (1996) and Ruggeri (2004) present useful summaries of shear strength values of concrete-rock contacts published in previous historical databases such as Rocha (1964), Link (1969), Lo et al., (1991), EPRI (1992), and ISMES (1999), Zoorabadi and Carter (2022).

Even though EPRI (1992) is the most commonly used historical database for concrete dam projects, surprisingly it only includes the results of two in-situ direct shear tests. The remaining data points had been obtained from laboratory tests (35 bonded samples, 1 unbonded sample, 11 sawcut samples, and 11 rock cores with concrete cast on them). The database published by Lo et al. (1991) consists of laboratory direct shear tests conducted on bonded and unbonded samples obtained from drilling through dams with 15 to 80 years of operation. Similarly, ISMES (1999) databases include only laboratory direct shear test data. Vizini and Futai (2019) also compiled a database mainly consisting of relatively new published laboratory tests on concrete-rock contacts (mainly on granite samples).

Considering the mentioned limitations on existing information, the authors of this paper decided to develop a new comprehensive database including both in-situ and laboratory tests. This broad data gathering campaign helped develop a new concrete-rock database with 381 datapoints for peak shear strength and 441 datapoints for residual shear strength. To the authors' knowledge, this database of shear strength of concrete-rock contacts is the most complete ever assembled.

3.1. Scale Impact

In the newly developed database in the current study, between 49% and 51% of the datapoints (188 out of 381 peak shear strength, and 225 out of 441 residual shear strength) were from the in-situ direct shear tests with the block size larger than 0.5 m. Figures 3 and 4 plot the peak and residual shear strength versus the applied normal stress, respectively, for both in-situ (large scale) and laboratory tests (small scale).

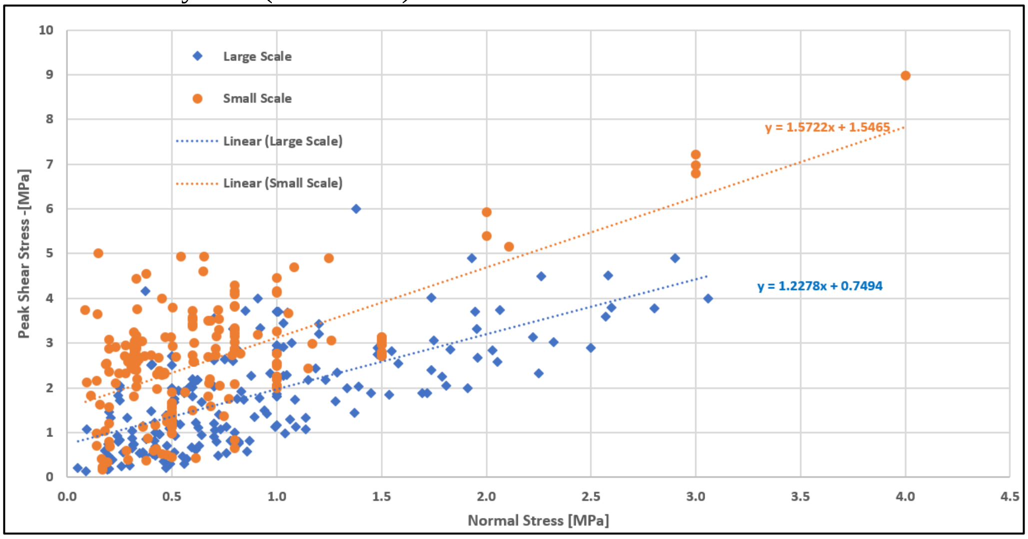

Figure 3. Peak shear strength versus normal stress for both laboratory and in-situ tests.

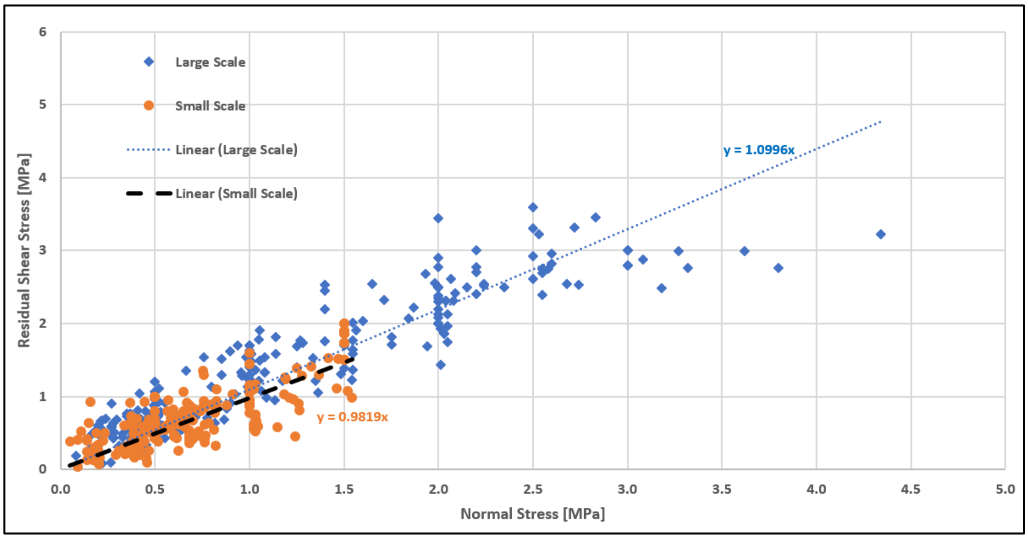

Figure 4. Residual shear strength versus normal stress for both laboratory and in-situ tests.

Despite the scatter in the peak shear strength data observed in Figure 3, particularly for normal stress below 1.0 MPa, the figure clearly shows that the peak shear strength of concrete-rock contacts is scale-dependent and decreases with increasing the size of the sample. The Mohr-Coulomb linear criterion fitted to small scale and large scale datapoint in Figure 3 shows that both cohesion and friction components of the peak shear strength are lower for the large-scale tests. The peak cohesion drops from approximately 1.55 MPa to 0.75 MPa (≈ 52% drop) and the peak friction angle drops from 58° to 51° (≈ 12% drop).

Figure 5 shows that the residual shear strength versus applied normal stress trend for the results obtained from both large and small scale tests are relatively close to each other for normal stresses below 1.5 MPa. It should be noted that for very low normal stresses (less than 0.3 MPa), the residual shear strength obtained from small scale tests are relatively higher. However, with increasing normal stress the residual shear strength datapoints from large scale tests show slightly more scatter (potentially due to the dilation over the remaining large-scale asperities).

The estimated residual friction angle for normal stress between zero and 3.0 MPa are 48° and 44°, for large scale and small scale test data respectively. When the normal stress increases above 3.0 MPa, the residual shear strength data plateaus at around 3.0 MPa shear strength, showing some level of apparent cohesion component and a lower residual friction angle, most probably induced by interlocking of the remaining asperities in the contact.

3.2. Inferred Peak and Residual Shear Strength Correlations

As previously discussed, the peak shear strength of concrete-rock contacts is significantly dependent on the scale of the test samples, with small scale test in general overestimating the shear strength of the interfaces. Therefore, in his paper only large scale datapoints have been used to develop representative correlations for peak shear strength. Figure 5 presents the estimated upper bound, best fit and lower bound Mohr-Coulomb (linear regression) correlations for the in-situ peak shear strength datapoints. The upper and lower bound correlations are likely to be representative of the peak shear strength of concrete to strong rock types (such as Granite, Gneiss, crystalline Limestones) and concrete to weak rock types (Shale and clay rich rocks), respectively. However, the reader should be cautious when adopting a strong rock fit as that might also be influenced by high strength concrete of the dam. As a comparison, EPRI (1992) presented a peak cohesion between 1.3 to 1.9 MPa and a peak friction angle of 54° to 68° for most rocks, which ise relatively higher than the peak shear strength parameters presented in Figure 5.

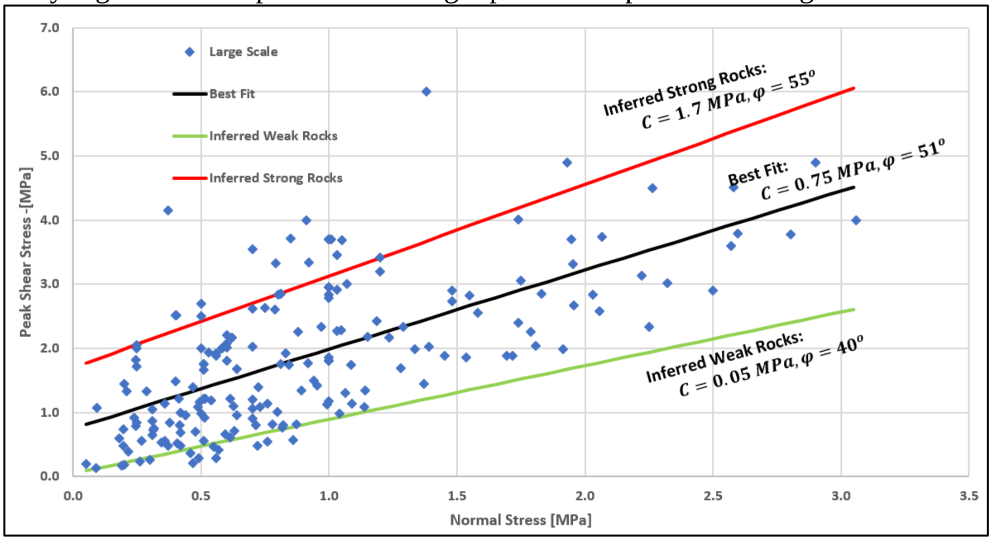

Figure 5. Linear Mohr-Coulomb correlations for the peak shear strength of concrete-rock contacts.

Both large and small scale tests datapoints for residual strength have been analysed to develop the linear Mohr-Coulomb correlations presented in Figure 6. The residual shear strength correlations were limited to the normal stress up to 3.0 MPa. As discussed previously, the residual shear strength of concrete-rock contacts shows an apparent cohesion component induced by interlocking of the remaining asperities at the larger scale tests.

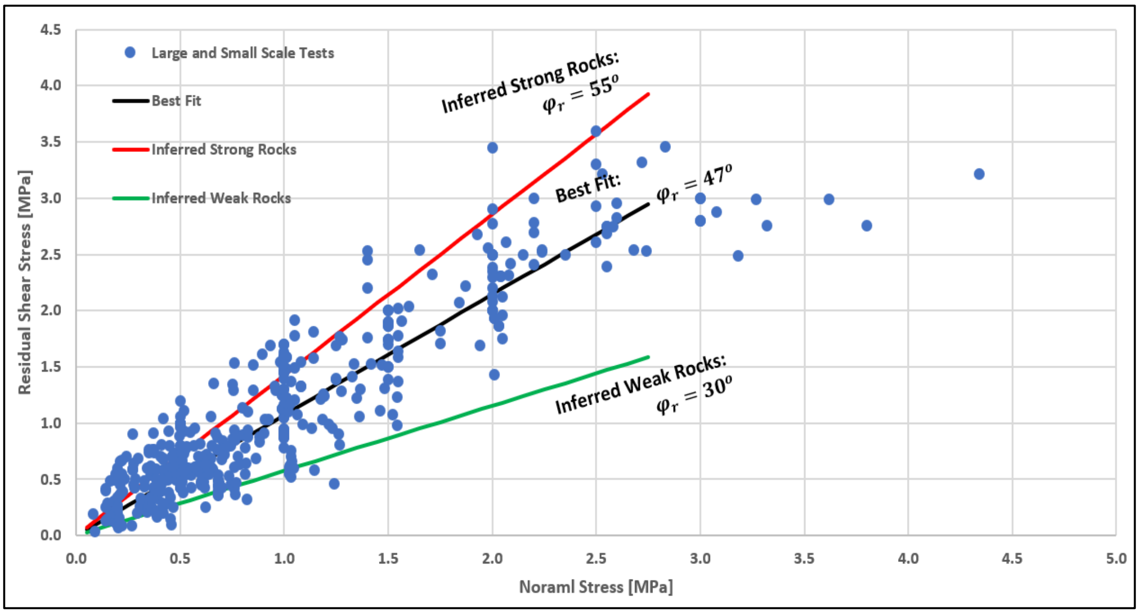

Figure 6. Linear Mohr-Coulomb correlations for the residual shear strength of concrete-rock contacts.

4. CONCLUSION

As a part of an in-house research and development program, the authors developed one of most comprehensive databases on basic friction angle of rock joint and shear strength of concrete-rock contacts by gathering test data from published literature and reports. The compiled database includes 309 datapoints for the basic friction angle of unweathered and clean rock joints and 773 datapoints (355 peak shear strength and 418 residual shear strength) for shear strength of concrete-rock contacts.

The statistical analysis of the gathered data showed that a basic friction angle of 31° represents a reasonable average for unweathered and clean rock joints for most rock types, except for weak clayish rocks. The statistical results of the basic friction angle data presented in Table 1 can be used in combination with Equation (3) to calculate the residual friction angle for application of Barton & Bandis failure criterion. It should be noted that the calculated residual friction angle must be equal or less than the basic friction angle of associated rock joint.

The statistical assessment of the gathered database for shear strength of the concrete-rock contacts showed that the peak shear strength obtained from the small-scale tests considerably overestimate higher the shear strength of the contact, particularly for low applied normal stresses. Based on that, the authors proposed the adjustment of laboratory results using the linear and nonlinear correlations presented in Table 1.

Table 2. Scale effect factors for peak shear strength obtained from new database.

Applied Normal Stress Range [MPa]	Strength ratio between (τ_p/σ_n) for large scale to small scale tests
0 - 0.1	17%
0.1 - 0.3	43%
> 0.3	57%

Unlike peak shear strength, the statistical analyses showed that the impact of test scale on the residual shear strength is relatively insignificant. In the absence of site-specific tests, the correlations presented in Figures 5 and 6 can be used as a practical estimation of the shear strength for preliminary phases of design of new dams, or at early stages of the safety reviews of existing concrete dams. Readers must use caution in applying the proposed correlations when the foundation contains clay-rich rocks with high sensitivity to water content, such as soft shale or claystone.

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