

Article

Shear Strength of Rock Discontinuities with Emphasis on the Basic Friction Angle Based on a Compiled Database

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Abstract

The shear strength of rock discontinuities is a critical parameter in rock engineering projects for assessing the safety conditions of rock slopes or concrete dam foundations. It is primarily controlled by the frictional contribution of rock texture (basic friction angle), the roughness of discontinuities, and the applied normal stress. While proper testing is essential for accurately quantifying shear strength, engineering geologists and engineers often rely on published historical databases during early design stages or when test results show significant variability. This paper serves two main objectives. First, it intends to provide a comprehensive overview of the basic friction angle concept from early years until its emergence in the Barton criterion, along with insights into distinctions and misunderstandings between basic and residual friction angles. The other, given the influence of the basic friction angle for the entire rock joint shear strength, the manuscript offers an extended database of basic friction angle values.

Keywords: shear strength of discontinuities; basic friction angle; residual friction angle; Barton criterion for rock joints



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1. Introduction

The basic friction angle ϕ_b is a parameter referred to in several rock joint shear strength criteria, being the most prominent the Barton criterion (e.g., [1,2]), though its appearance in geotechnics can be traced to earlier publications.

$$\tau = \sigma_n \tan(\phi_b + i) \quad (1)$$

Newland and Allely [3] proposed a criterion to estimate the shear strength of granular soils (sands, in general) at a given normal stress σ_n :

It includes two parameters: ϕ_b , referred to as the “angle of frictional sliding resistance between the particles”, and i , the average angle of deviation of particle displacement from the direction of the applied shear stress, which can be considered as the angle of dilation.

Based on laboratory tests of kaolinite-plaster specimens with regular, interlocking, triangular teeth, Patton extended this concept to rock joints under low normal stresses considering that ϕ_b is the shear strength of saw-cut or sand blasted rock surfaces and i the inclination of the triangular teeth [4]. For high normal stresses, the criterion assumes most

irregularities would be sheared off and the shear strength would follow a linear Coulomb relation:

$$\begin{cases} \tau = \sigma_n \tan(\phi_b + i) \\ \tau = c + \sigma_n \tan \phi \end{cases} \quad (2)$$

Barton pointed out that the shear strength of natural rock joints, which do not display regular, triangular profiles but a wide range of asperities with different inclinations, had to fall between these two somewhat extreme points depending on the applied normal stress [5]. He merged both equations considering that the shear strength is composed of the basic friction angle, the peak dilation angle d_n , and the strength of the sheared asperities S_n , as depicted in Figure 1.

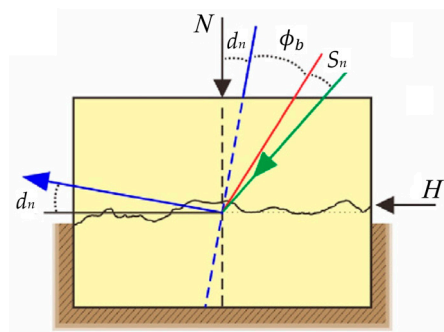


Figure 1. Contribution of basic friction angle and dilation induced asperities to the shear strength of rock joints [6].

Subsequently, the dilation component i gave way to a logarithmic function of the normalized strength and included a parameter related to the roughness of the joint surface [1].

$$\tan^{-1}\left(\frac{\tau}{\sigma_n}\right) = JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_b \quad (3)$$

Finally, in order to account for weathered joint surfaces, ϕ_b was replaced by the residual friction angle ϕ_r , resulting in the well-known Barton criterion:

$$\tau = \sigma_n \times \tan\left[\phi_r + JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right)\right], \quad (4)$$

where JRC is the Joint Roughness Coefficient and JCS is the Joint-wall compressive strength [2]. It should be noted that the value of ϕ_r in the previous equation is different from the shear strength of non-flat, dilatant rock joints measured at residual state-shear stress at a stage where the shear stress is stable as the shear displacement increases.

Barton & Choubey recommended the estimation of the residual friction angle as the ratio between the Schmidt hammer rebound on the natural, weathered, saturated joint surface r and the rebound on an un-weathered, dry, flat surface of the same rock R as follows [2]:

$$\phi_r = \phi_b - 20^\circ + 20 \frac{r}{R} \quad (5)$$

This equation implies that the residual friction angle is always smaller than the basic friction angle, or at most equal when the rock joint is fresh and totally un-weathered.

The Schmidt hammer rebounds R was primarily devised to provide estimates of the compressive strength of concrete. Its use was extended to the estimation of rock uniaxial compressive strength σ_c by Miller [7], and to JCS by Barton & Choubey [2] according to the following equation:

$$\log_{10}(\sigma_c \text{ or } JCS) = 0.88 \cdot 10^{-3} \gamma(R \text{ or } r) + 1.01, \quad (6)$$

with σ_c and JCS in MPa and the dry density of the rock γ in kN/m^3 .

Hoek and Bray [8] stated that the Barton & Bandis criterion produces more reasonable results for the normal stress in the range of $0.01 < \sigma_n/JCS < 0.3$. Figure 2 presents the contribution of joint surface roughness $JRC \log_{10}(JCS/\sigma_n)$ to the shear strength of rock joints based on Barton & Bandis criterion in this range for JRC values of 3, 5, 9, and 20. These JRC values were purposely chosen to cover as uniformly as possible the range of roughness contribution along all normal stresses. This figure also shows that this contribution is highest under lower normal stress and decreases significantly when normal stress increases.

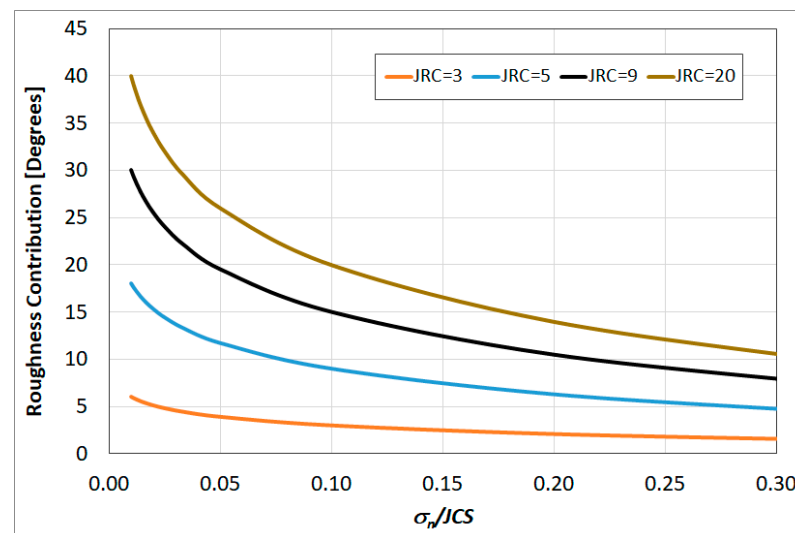


Figure 2. Contribution of roughness and shear strength of asperities on shear strength of rock joints.

The relative impacts of JRC , JCS , and φ_b in the shear strength of rock joints are illustrated by the graphs in Figure 3. The background figure is the original figure by Hobbs [9] with the relationship between shear and normal stresses at failure for broken specimens of Ormonde siltstone, a rock from a coal-bearing strata. First, Hobbs conducted triaxial tests that led to failure along inclined surfaces in the specimens; then, he applied different confining pressures and given the angles between the failure surface and the specimens axes calculated the shear and normal stresses required to produce movement. This figure was used by Barton [2] to show the fit of his criterion with JRC equal to 20, JCS equal to σ_c and φ_b (not yet φ_r) equal to 30° . In Figure 3, several graphs with different values of the Barton criterion are superimposed on the original figure allowing to observe the impact of the following:

- JRC , with values ranging between 20 and 5, and keeping JCS and φ_b constant;
- JCS , with values ranging between σ_c and $\sigma_c/4$, and keeping JRC and φ_b constant;
- φ_b , with values ranging between 26° and 34° , and keeping JRC and JCS constant.

The following conclusions can be deduced from this sensitivity breakdown. JRC has the lowest impact compared to JCS and basic friction angle. The contribution of JCS changes is much higher compared to JRC and basic friction angle, particularly when JCS drops to 25% of σ_c . However, it must be noted that for low JCS values, it is likely that JRC might drop to lower values too.

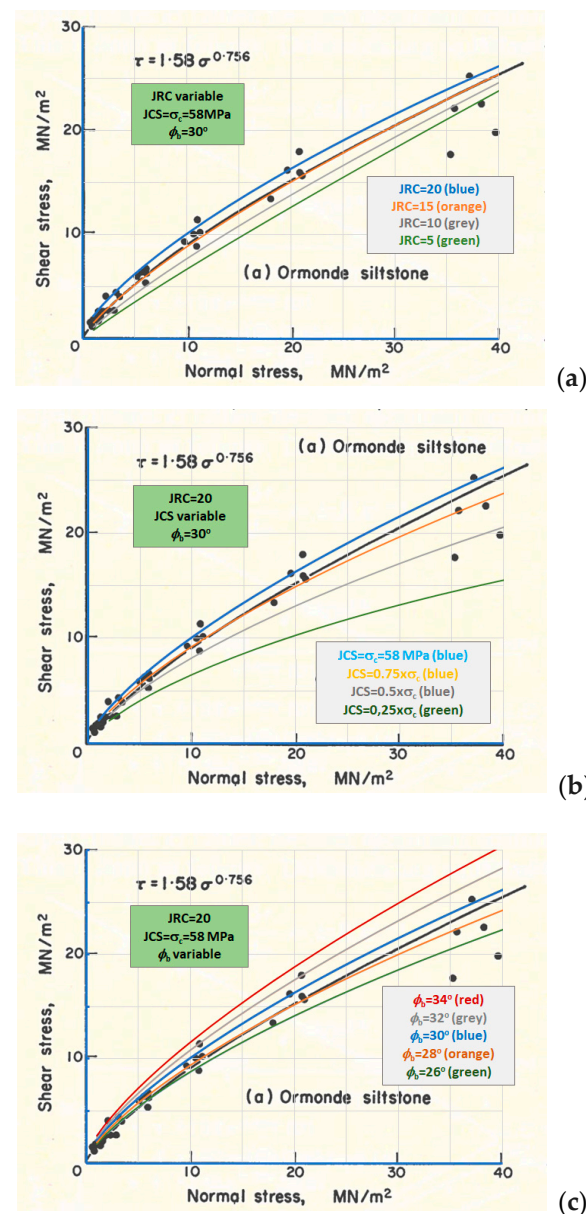


Figure 3. Contribution of JRC (a), JCS (b), and basic friction (c) to the shear strength of rock joints (adapted from [9]).

A slightly deeper observation of the graphs allows us to state that for normal stresses commonly encountered in rock engineering project—up to around 5 MPa—better fits are reached with a JRC value of 15 in Figure 3a), with a JCS value between 50% and 75% of the uniaxial compressive strength in Figure 3b), and with a value of 26° for the basic friction angle in Figure 3c).

Given the linear relation between the basic friction angle and $\tan^{-1}\left(\frac{\tau}{\sigma_n}\right)$, a reduction in the basic friction angle entails an equal reduction in the shear strength in degrees. However, as is the case of other geotechnical parameters, the shear strength of rock joints has to be assessed via the friction coefficient, and then the relationship is no longer linear and depends on the normal stress. Simple calculations show that for a basic friction angle of 30° a variation of $\pm 2^\circ$ will cause a change of around $\pm 7.5\%$ in the friction coefficient.

2. Basic Friction Angle and Tilt Tests

The basic friction angle should preferably be determined under laboratory-controlled conditions to ensure reproducibility and comparability. Tilt tests are among the most widely used methods to measure φ_b , given their simplicity and cost-effectiveness [10,11]. Figure 4 presents possible configurations for tilt tests [11].

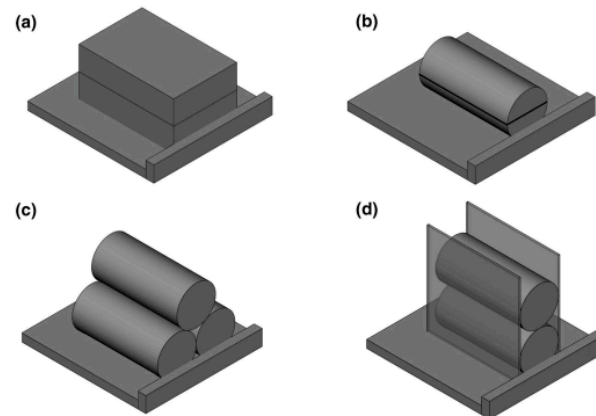


Figure 4. Different tilt-test arrangement depending on the type of contacts, (a) test performed on square based slabs; (b) test performed on cylindrical sample longitudinally cut; (c) Stimpson type tests; (d) two-core set-up [11].

Tilt test involves placing a planar rock surface, or other specified interface, on a tiltable plane and gradually increasing the tilt angle until the top block begins to slide due to gravity, while keeping the lower block fixed. The angle at which sliding occurs is recorded as the tilt angle α , which directly correlates to the basic friction angle for flat surfaces without roughness.

The tilt test procedure requires adequate sample preparation and equipment setup. The rock surfaces should be planar and smooth, typically achieved using a disk saw.

The most relevant factors influencing accuracy of the results are as follows:

- non-planarity of the surfaces, since small deviations from a planar surface can cause localized contacts and uneven stress distributions; proper machining and previous inspection are crucial.
- fast tilt rates may introduce dynamic effects leading to premature sliding; controlled, slow tilting is essential.

Besides these factors and inherent variability of the rock material, dispersion of tilt test results may arise from several other causes. Ambient temperature and humidity should be controlled or at least recorded, as they can influence results. Tests are often conducted under dry conditions, but wet conditions or other environmental factors may also be considered if they reflect field scenarios. Surface cleanliness, mainly related to rock dust resulting from surface wear, is also important to avoid contamination that could artificially alter friction properties.

Stimpson-type tilt tests (Figure 4c), where instead of a planar surface three rock cores are used and sliding occurs along cylinder generatrices, may yield incorrect results, since drilling may produce cores with smooth surfaces for soft, friable rocks, or polished core surfaces can result for hard, quartz-rich rocks. Further note that the formula in the original Stimpson paper is incorrect [10].

Tilt tests using only two rock cores are even more problematic (Figure 4b), as, on top of the previous drawbacks, equilibrium of the rock core along linear generatrices of the other core is problematic and requires the tilting equipment to be free of any dynamic effects related to non-constant tilting rates. Alejano et al. [10] stated that the tilt test results from

the contact arrangements of Figure 4a,b) with a contact area of at least 50 cm² and length to height ratio of at least 4 would provide more reliable results.

It is worth noting that for engineering purposes, for friction, whether on rock surfaces, soil, aggregates, or any other interfaces, the relevant entity is the friction coefficient. So, averaging should not be performed using the friction angle, even though sometimes errors could be minimal. This is another reason why using the median is preferable, as prescribed in the ISRM Suggested Method [11].

Furthermore, tilt test results are affected by several factors including geological origin, degree of weathering, specimen preparation, surface cutting and finishing, specimen shape and size, the tilting rate, equipment vibrations, the number of repetitions, and surface wear [12].

Concerning mineralogy and rock type, it is relevant to note that rocks with high quartz content, such as quartzites, dolerites, or granites, may display extremely low friction angles (i.e., in the range of 10° to 15°) due to the almost polished, glassy finishing of their surfaces after being cut or drilled. On the other hand, carbonate rocks with lower compressive strength may display high friction angles (i.e., in excess of 35°).

Another relevant factor refers to the preparation of the surface. In case it is not well performed, the disk saw creates grooves on the surface that may allow for indentation and that will result in higher dispersion and an increase in the friction angle.

3. New Database for Basic Friction Angles

3.1. Database

Over the years several researchers, such as Ripley & Lee, Barton, Coulson, Barton, Richards, Cruden & Hu, Waltham, and Geertsema [1,5,13–19], have published basic friction angle test results for various rock types. Likewise, Alejano et al. [10] reported a basic friction angle between 25 and 30 degrees for sedimentary rocks, and 30 and 35 degrees for metamorphic and igneous rocks based on data published by Barton [1,16]. These databases were usually limited to small number of tests on some selected rock types. This paper presents a statistical analysis of data gathered from additional sources in an effort to provide an extended and more reliable database for the basic friction angle of rock joints.

In addition to the above-mentioned literature examples, data from a wider range of other publications was gathered and analyzed in this paper, including Bruce et al., Hutchinson, Byerlee, Hoek & Bray, Ramana & Gogte, Goodman, NBG, Duzgun et al., Grasselli & Egger, Wines & Lilly, Lanaro & Fredriksson, Fuenkajorn, Kemthong, Alejano et al., Ruiz, Malkowski, HDR, Ulusay & Karakul, Jang et al., Anderson & Hagberg, Rahim, Zhang et al., Rahim, Lee et al., Behnia & Nateghpour, Bordehore et al., and Raj [7,20–44]. This extensive data collection is intended to contribute to the development of a new basic friction angle database containing 324 data points. It is important to refer that some of these points represent the average of multiple test results, as the related reference only provided statistical summaries. Though some references are not clear about the way the basic friction angle values were obtained, most of the collected data were from tilt tests (using various test configurations) and direct shear tests conducted on saw-cut samples. Data from tilt tests using three cores (Figure 4c) were deliberately excluded.

All data is in the tables compiled for sedimentary, igneous, and metamorphic rocks presented in the Annex at the end of this paper. To the best of the authors' knowledge, this database on the basic friction angle of rock joints is the most comprehensive ever compiled.

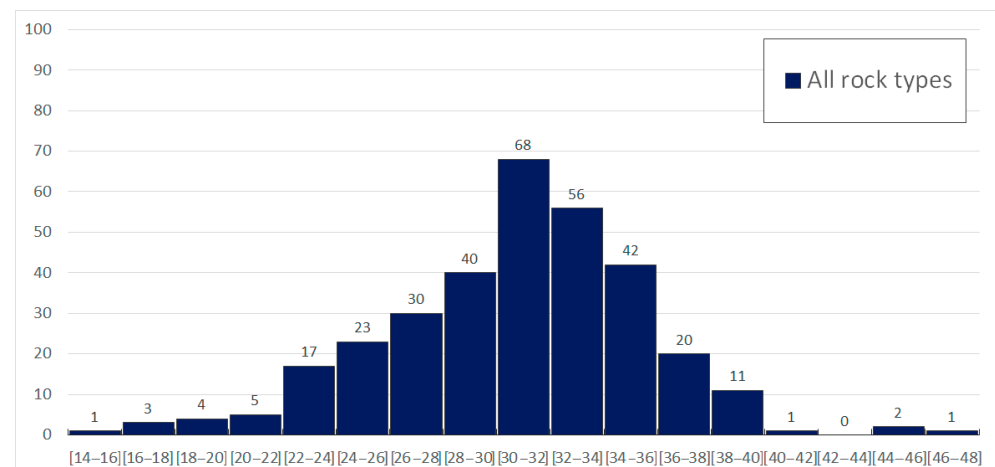
Table 1 contains some basic statistical descriptive parameters of the data, namely the minimum and maximum values, the median and the average values, the standard deviation, and the fifth percentile (95% probability of being exceeded). The values refer to all data points, but they are also split according to the three main rock types.

Table 1. Basic statistical parameters of the basic friction angle data.

	Total	Sedimentary	Igneous	Metamorphic
Nr. Data Points	324	143	113	68
Minimum [degrees]	14	14	17	20
Maximum [degrees]	48	45	45	48
Median [degrees]	31.4	32.0	31.0	30.0
Average [degrees]	31.2	31.6	30.9	30.8
Stand.Dev. [degrees]	6.56	6.42	6.63	6.76
5th percentile [degrees]	22.6	23.3	22.2	21.9

It should be stressed that, though all values in the table are in degrees, the averages, standard deviations and fifth percentiles were calculated using the corresponding friction coefficients and back-calculating to friction angles in degrees. Carrying out calculations with the friction angle is the appropriate way, since it is the parameter that is considered in safety analysis. However, since in this case the sets comprise a large number of data points, differences are not relevant as they do not amount to more than 0.5° , which only lead to deviations of around 2%.

Figure 5 illustrates the frequency analysis of the collected data across all rock types. It shows a centred distribution with the large majority of the values (94.5%) falling in the range of 22° to 40° and median, average, and mode falling in the range of 22° to 40° .

**Figure 5.** Absolute frequency histogram of the basic friction angle for all rock types.

Considering the basic friction coefficients instead of the basic friction angles, the histogram of the same values reveals a slightly right skewed distribution with more than 90% of the values above 0.45, which corresponds to a basic friction angle of 24.2° (Figure 6).

Separating the set of friction angles according to the three rock types (igneous, sedimentary, and metamorphic), the respective histograms are presented in Figure 7, which do not show relevant differences with the previous, except in the case of the metamorphic rocks, presumably due to its smaller amount of data.

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. However, it should be noted that for some rocks variability may be significantly high. For instance, within the datapoints pertaining to sedimentary rocks, the lowest recorded basic friction angle of 14° refers to a shale suggesting the possibility that this rock might have been relatively low friction angles. However, basic friction angle values for shale higher than 36° are denoted in the annex.

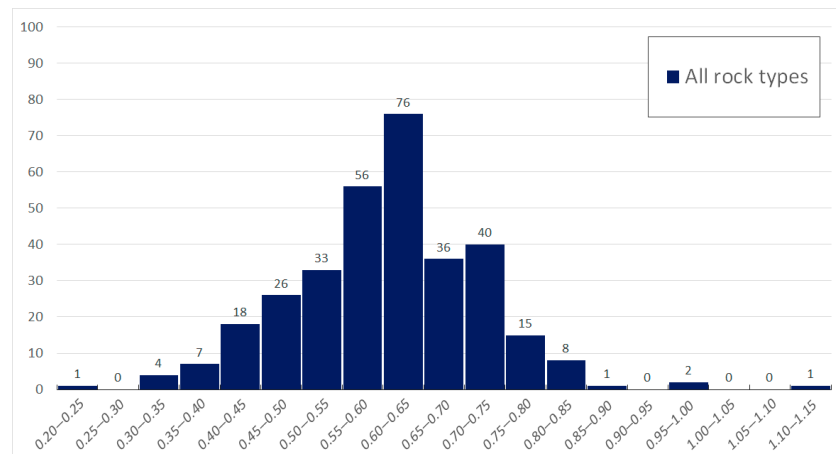


Figure 6. Absolute frequency histogram of the basic friction coefficient for all rock types.

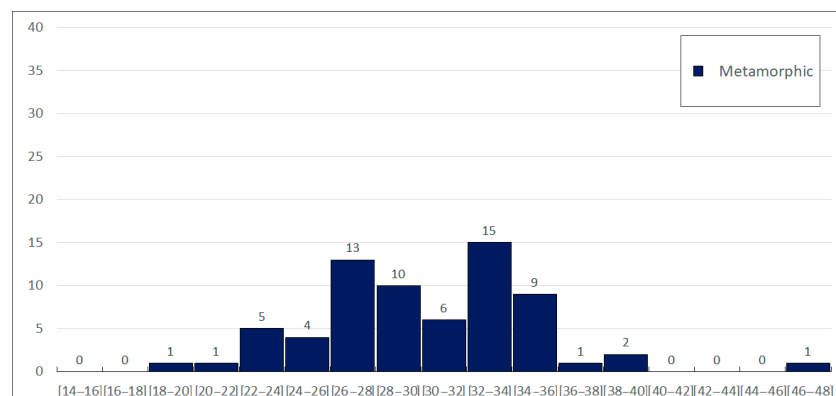
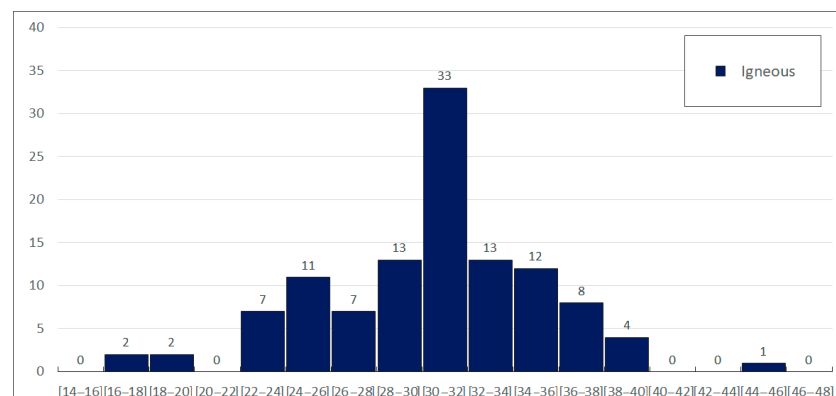
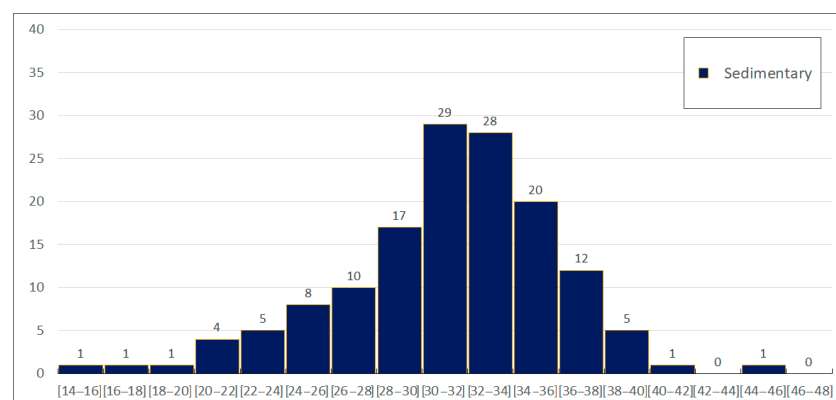


Figure 7. Basic friction angle histograms for the different rock lithologies: sedimentary, igneous, and metamorphic.

Low friction values (below 20°) are usually associated with hard rocks, e.g., quartzite and marble, that often yield polished and even vitreous cut surface. On the opposite end, since they are hard to cut, these same rock lithologies may also produce surfaces with grooves that may wrongly increase the basic friction angle. It is worth stressing that to deal with these issues, Barton specifies that the rock surface should be sand-blasted prior to performing the tilt tests.

These conclusions align with the findings of Byerlee [22], 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.

3.2. Impact of Moisture and Weathering

As previously mentioned, Barton and Choubey [2] introduced Equation (5) to estimate the magnitude of the residual friction angle required for the Barton criterion for rock joints in their natural conditions where they might have experienced some level of weathering and water saturation. They observed that saturation has a small reduction impact of 1° to 3° on the basic friction angle of rock joints if the joint surface does not have water sensitive materials.

The residual friction angle of rock joints can be significantly impacted by the weathering of the surficial (few millimetres) of the joint walls. The impact of weathering can be indirectly inferred from the Schmidt hammer rebound. Tugrul & Zarif [45] measured the Schmidt hammer rebound for a selected sandstone unit from Istanbul, Turkey with various levels of weathering (Table 2). This case study showed that the Schmidt hammer rebound reduces more than 65% with increasing weathering level from fresh rock to completely (extremely) weathered rock. Based on Equation (5), the residual friction angle will be more than 13° less than the basic friction angle of fresh and dry rock.

Table 2. Impact of weathering on Schmidt hammer rebound for a sandstone unit. Istanbul, Turkey [45].

Weathering Level	Schmidt Hammer Rebound	Uniaxial Compressive Strength [MPa]
Fresh	34–40	42–63
Slightly Weathered	22–36	26–45
Moderately Weathered	14–24	11–28
Highly Weathered	12–15	3.5–12
Completely Weathered	<12	Not determined

It should also be noted that the presence of clayish infill materials can significantly reduce the basic friction angles. In worst case where the infills are relatively thick not allowing contact between rock walls of the joint, the friction angle will be equal to the drained friction angle of the infills.

In addition to its direct effect on the residual friction angle of rock joints, weathering also influences the Joint Compressive Strength (JCS) of the joint surface. The JCS represents the uniaxial strength of the joint surface, and weathering can alter its magnitude. The Norwegian Rock Mechanics Group (NMG) [25] introduced a rating factor f_w to quantify the effect of weathering on the uniaxial compressive strength of rocks, based on the weathering class, as follows:

$$UCS = \frac{UCS_{fresh}}{f_w} \quad (7)$$

The rating factor f_w is determined using the guide provided in Table 3. Given that the JCS of rock joints depends on UCS of the host rock and the weathering condition of the joint surface, Equation (7) can be used to evaluate the effect of weathering on JCS According to

Equation (7) and Table 2, the JCS of rock joints in the highly weathered class can decrease to 10% of the JCS of joints in fresh rock.

Table 3. Weathering rating factor as defined by NBG [25].

Class	Term	Description	Rating (f_w)
I	Unweathered	No visible signs of rock material weathering; perhaps slight discolouration on major discontinuity surface.	1
II	Slightly weathered	Discolouration indicates weathering of rock material and discontinuity surface. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition.	1.75
III	Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present, either as a discontinuous framework or as corestones.	2.5
IV	Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present, either as a discontinuous framework or as corestones.	10
V	Completely weathered	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.	
VI	Residual soil	All rock material is converted to a soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.	

Note: The suggested ratings of the factor f_w are based on few data and should be therefore be considered very approximate, especially for high grades of weathering.

3.3. Residual Friction Angle from Direct Shear Tests

As previously referred, Barton & Choubey [2] replaced ϕ_b with ϕ_r —the friction angle of a flat, non-dilatant rock joint—to account for the effects of weathering and moisture. It allowed to extend the initial peak shear criterion for rough, fresh rock joints to natural, weathered rock joints.

Figure 8 presents the shear stress vs. shear displacement graph and the corresponding normal displacement vs. shear displacement graph depicting the typical behaviour of rock joints under direct shear. It is well known that the shear behaviour of rock joints observed

in direct shear tests is characterized by an initial increase in the shear stress (τ_{peak}) for relatively small shear displacements until a peak, or maximum, shear strength is reached, followed by a threshold where the shear displacement increases with almost no change in the shear stress. In this latter stage, the shear strength should preferably be referred to as ultimate (τ_{ult}), though it is also designated as residual more often than it should. Hence, despite all notices, it is common to come across in rock engineering design the use of the friction angle obtained for large shear displacements from in situ or laboratory rock joint direct shear tests.

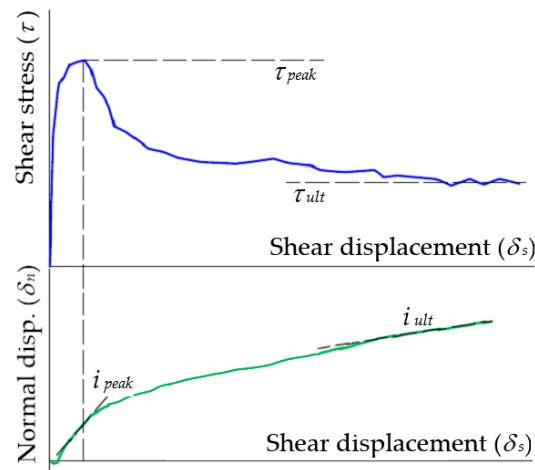


Figure 8. Typical behaviour of rock joints under shear [46].

This figure shows that for large shear displacements rock joints usually still display some dilation (referred by i_{ult} in the figure), since it is likely that some asperities, but not all, have been sheared at peak displacement. This implies that using the ultimate shear strength will overestimate the value of ϕ_r in Barton's criterion most of the times.

4. Conclusions

This paper begins with a brief review of the shear strength of rock discontinuities, with a focus on the Barton criterion, along with key insights into its main parameters. These insights provide readers with a fundamental understanding of rock joint shear strength. The focus then shifts to the basic friction angle, a crucial parameter for assessing the stability of rock slopes, underground structures, and rock foundations such as bridges and dams.

Over the years, various researchers have published databases on the basic friction angle, though these were often limited to specific rock types. Given the significance of this parameter, a comprehensive database was compiled by collecting data from accessible published literature spanning from 1960 to recent years. Basic statistical analysis of this dataset indicates that a basic friction angle of 31° serves as a reasonable average for unweathered and clean rock joints across most rock types, except for weak clay-rich rocks. Additionally, the results from this statistical analysis can enhance the reliability of estimated averages and standard deviations of new test results through a Bayesian approach.

Furthermore, this paper elucidates the distinctions between the basic friction angle and the residual friction angle, the latter being a key input for the Barton criterion. It also examines the effects of weathering and infill materials on the residual friction angle and the Joint Compressive Strength of rock joints. These discussions provide valuable guidance for more accurate quantification of shear strength parameters in rock discontinuities.

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draft preparation, M.Z.; writing—review and editing, M.Z. and J.M.; visualization, M.Z. and J.M. All authors have read and agreed to the published version of the manuscript.

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Data Availability Statement: All data is included in Appendix A.

Conflicts of Interest: Author Mahdi Zoorabadi was employed by the company Mahdi Zoorabadi. The remaining authors declare that the research was conducted in the absence of any commercial or financial relationships that could be construed as a potential conflict of interest.

Appendix A

Sedimentary		
Rock Type	Reference	Basic Friction Angle
Ripley & Lee [13]	Mudstone	31
	Sandstone	29
Patton [4]	Sandstone	30
	Sandstone	29
Hutchinson [21]	Chalk	30
Coulson [14]	Limestone	34
	Limestone	31
	Mudstone	32
	Mudstone	29
	Sandstone	33
	Sandstone	32
Richards [15]	Sandstone	33
	Shale	27
	Siltstone	32
	Siltstone	29
Barton & Choubey [2]	Sandstone	30
Hoek & Bray [8]	Dolomite	29
	Shale	27
Cruden & Hu [17]	Carbonate-Dolomite	35.4
	Carbonate-Dolomite	33.5
	Carbonate-Dolomite	24.5
	Carbonate-Dolomite	26.8
	Carbonate-Dolomite	38
	Carbonate-Dolomite	22.8
	Carbonate-Dolomite	38.1
	Carbonate-Dolomite	32.4
	Carbonate-Dolomite	41.3
	Carbonate-Dolomite	30.6
	Carbonate-Dolomite	28.6
	Carbonate-Dolomite	23.9
	Carbonate-Dolomite	21.5
	Carbonate-Dolomite	33.4
	Carbonate-Dolomite	35.7

Sedimentary		
Rock Type	Reference	Basic Friction Angle
	Carbonate-Dolomite	39.4
	Carbonate-Dolomite	34.7
	Carbonate-Dolomite	30.8
	Carbonate-Dolomite	30.3
	Carbonate-Dolomite	34.8
	Carbonate-Dolomite	39
	Carbonate-Dolomite	34.3
Ramana & Gogte [23]	Limestone	34
	Limestone	32
	Sandstone	31
	Sandstone	36
	Sandstone	23
Goodman [24]	Dolomite	35
	Shale	14
	Shale	22
	Siltstone	32
Waltham [18]	Claystone	20
	Chalk	25
	Greywake	45
	Gypsum	30
	Limestone	35
	Mudstone	30
	Shale	25
Grasselli & Egger [27]	Sandstone	37
Duzgun et al. [26]	Limestone	34
	Mudstone	25
	Sandstone	32
	Siltstone	32
	Tuff	21
Grasselli & Egger [27]	Limestone	36
Geertsema [19]	Mudstone	33
	Sandstone	28
	Sandstone	35
	Sandstone	36
	Shale	32
	Shale	32
	Siltstone	38
Fuenkajorn [30]	Sandstone	33.7
	Sandstone	31.7
	Sandstone	30.7
	Sandstone	31.7
Kemthong [31]	Sandstone	32
	Sandstone	35
	Sandstone	34
	Sandstone	29
	Sandstone	33

Sedimentary		
Rock Type	Reference	Basic Friction Angle
	Sandstone	33
	Sandstone	29
	Sandstone	34
	Sandstone	32
	Sandstone	27
	Sandstone	32
	Sandstone	33
Ruiz & Li [33]	Limestone	26.4
	Sandstone	34.1
	Sandstone	38.1
	Sandstone	36.8
Malkowski [34]	Sandstone-medium	26
	Sandstone-fine	25
Ulusay & Karakul [36]	Limestone	32.4
	Limestone	25.9
	Limestone	37.6
	Limestone	30.5
	Limestone-Fossiliferous	35.4
	Limestone-Oolitic	36.9
	Travertine	38.3
	Travertine	28.2
	Travertine	31
Jang et al. [37]	Sandstone	29.9
	Sandstone	29
	Sandstone	29.5
	Sandstone	25.2
	Sandstone	27.8
	Sandstone	26.5
Zhang et al. [39]	Shale	36.8
	Shale	33.5
	Shale	35
	Sandstone	36.4
	Sandstone	33.3
	Sandstone	31.4
	Sandy Shale	34.5
	Sandy Shale	32.4
	Sandy Shale	31.3
Rahim [40]	Sandstone-fine	24
Lee et al. [41]	Sandstone	36.4
	Sandstone	33.3
	Sandstone	31.4
	Sandstone	32.5
	Sandstone	33.7
	Sandstone	30.1
	Sandy Shale	34.5
	Sandy Shale	32.4

Sedimentary		
Rock Type	Reference	Basic Friction Angle
	Sandy Shale	31.3
	Sandy Shale	34.1
	Sandy Shale	32.7
	Sandy Shale	33.5
	Shale	36.8
	Shale	33.5
	Shale	31.5
	Shale	33.1
	Shale	35.1
	Shale	33.6
Raj [44]	Conglomerate	21
	Sandstone	18
Behnia & Nateghpour [42]	Limestone	35
	Limestone	27
	Limestone	36
	Limestone	27.5
Bordehore et al. [43]	Limestone	24
	Sandstone	37
Igneous		
Rock Type	Reference	Basic Friction Angle
Barton [5]	Porphyry	31
	Porphyry	31
Coulson [14]	Basalt	36.5
	Basalt	33
	Granite-course	33
	Granite-course	32
	Granite-fine	33
	Granite-fine	30
Richards [15]	Dolerite	36
	Dolerite	32
Hoek & Bray [8]	Basalt	34
Richards [15]	Granite	39
Goodman [24]	Basalt	31
Ramana & Gogte [23]	Granite	28
	Granite	25
	Basalt	28
	Basalt	25
	Dolerite	23
Waltham [18]	Granite	45
Duzgun et al. [26]	Aplite	33
	Granite	33

Igneous		
Rock Type	Reference	Basic Friction Angle
Geertsema [19]	Basalt	35
	Dolerite	36
	Granite	31
	Mudstone	31
	Dolerite	33
	Dolerite	31
	Riolite	35
Grasselli & Egger [27]	Granite	34
Wines & Lilly [28]	Basalt	33
	Basalt	35
	Basalt	36
	Basalt	37
	Basalt	37
	Basalt	38
	Basalt	38
	Basalt	39
	Basalt	40
	Basalt	40
	Dolerite	32
	Dolerite	34
	Dolerite	35
	Dolerite	35
	Dolerite	36
Lanaro & Fredriksson [29]	Granite	28.9
	Granite	31.2
	Granite	32
	Granite	31.4
	Granite	30.8
	Granite	29
	Granite	31.9
	Granite	30.1
	Granite	31.5
	Granite	30.7
	Granite	30.7
	Granite	30.9
Fuenkajorn [30]	Basalt	35.3
	Syenite	18.3
	Plagiogranite	24.7
	Monzonite	25.7
Kemthong [31]	Basalt	36
	Basalt	37
	Basalt	33
	Granite	17
	Granite	20
	Granite	18
	Granite	24
	Granite	25
	Granite	25

Igneous		
Rock Type	Reference	Basic Friction Angle
	Granite	26
	Granite	26
	Granite	25
Alejano et al. [10]	Granite	29
	Granite	26
	Granite	29
	Granite	30
Ruiz & Li [33]	Granite	30.8
HDR [35]	Basalt	22.2
	Basalt	31.1
	Basalt	22.9
	Basalt	33.7
	Basalt	36.1
	Basalt	25.5
	Basalt	28.1
	Basalt	31.9
	Basalt	31.6
Jang et al. [37]	Granite	26.3
	Granite	22.9
	Granite	22.4
	Granite	26.9
	Granite	30.1
	Granite	30
Ulusay & Karakul [36]	Andesite	27.9
	Andesite	28.4
	Andesite	32.02
	Andesite	30.9
	Ignimbrite	31.4
	Ignimbrite	36.7
	Ignimbrite	34.9
	Ignimbrite	30.8
	Basalt	30.9
	Granite	22.5
	Andesite	31.2
Zhang et al. [39]	Granite	28
	Granite	30.2
	Granite	30.8
Lee et al. [41]	Granite	28
	Granite	30.2
	Granite	30.8
	Granite	28.7
	Granite	29.7
	Granite	29.7
Bordehore et al. [43]	Rhyodacite	29

Metamorphic		
Rock Type	Reference	Basic Friction Angle
Ripley & Lee [13]	Slate	27
Barton [5]	Schist	27.5
Coulson [14]	Gneiss	27.5
	Gneiss	24.5
Richards (1975) [15]	Schist	30
	Schist	21
Goodman [24]	Quartzite	48
Ramana & Gogte [23]	Gneissic Granite	24
	Charnockite	24
	Quartzite	28
	Phyllite	28
	Amphibolite	28
Waltham [18]	Hornfels	40
	Schist	25
	Slate	25
	Gneiss	30
Duzgun et al. [26]	Hornfels	33
	Micaschist	26
	Quartzite	30
Duzgun et al. [26]	Slate	27.5
	Gneiss	28
	Soapstone	20
Geertsema [19]	Quartzite	30
	Quartzite	28
	Tillite	33
	Quartzite	29
Grasselli & Egger [27]	Serpentine	39
	Gneiss	36
Kemthong [31]	Marble	34
	Marble	34
	Marble	35
	Marble	34
	Marble	37
	Marble	36
Fuenkajorn [30]	Marble	34.3
	Marble	35.7
Alejano et al. [10]	Slate	27
	Magnesite	27
Ruiz & Li [33]	Greenstone	35.7
	Gneiss	32
	Gneiss	30.1
	Marble	32.9
	Gneiss	32.2
	Gneiss	31.5

Metamorphic		
Rock Type	Reference	Basic Friction Angle
	Gneiss	30
	Gneiss	27.3
	Micaschist	32.8
Ulusay & Karakul [36]	Serpentine-Carbonated	32.3
Zhang et al. [39]	Gneiss	29.4
	Gneiss	23.4
	Gneiss	31
	Marble	32.6
	Marble	33.8
	Marble	35.4
Lee et al. [41]	Marble	32.6
	Marble	33.8
	Marble	35.4
	Gneiss	23.4
	Gneiss	31
	Gneiss	29.4
	Gneiss	29.9
	Gneiss	27.9
	Gneiss	28.4
	Marble	33
	Marble	33.9
	Marble	34.7
Bordehore et al. [43]	Schist	23

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