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What are the representative values for the mechanical properties of a rock joint?

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Abstract

The European design standard (code) for geotechnical structures – Eurocode 7 – has been updated in a second generation. The new code has a larger scope than the current code: the new code will also concern the safety of dams and levees, as well as rock engineering structures above ground. For many European countries, this will likely be a considerable change, as neither rock slopes nor dams until now have been designed using a European design code. In this context, there is a need to study how spatial variability in rock mass properties best is managed in the verification of limit states, i.e. determination of their representative values (formerly characteristic values). In this paper, we discuss specifically how representative values can be determined using statistical methods for relevant rock properties, like the friction coefficient of rock joints, for verification of the structural safety regarding sliding stability of dams founded on rock. As a basis for the investigations in the paper, we use data from laboratory direct shear tests on the peak shear strength of rock joints. The result illustrates the importance of having a sufficient number of samples and the need for a thorough understanding of the effect of the sampling location. We hope that this contribution will stimulate a broader discussion in the scientific rock engineering community on limit state verification of rock engineering structures, as there are currently very few studies on this issue.

Keywords

Eurocode 7, Representative value, Dams, Rock joint, Friction angle.

1 Introduction

The second generation of the Eurocodes are being published. For rock engineers the most relevant parts are the general Eurocode EN 1990 (CEN, 2023) and the three parts of EN 1997 (CEN, 2024a; 2024b; 2025), known as Eurocode 7. The second generation aims, among other things, to provide

better applicability to above-ground rock engineering structures, such as rock slopes and rock foundations (Lamas et al, 2023). For the rock engineering community, this means a safety philosophical shift from applying deterministic safety factors to applying limit state design, which is based on a probabilistic view on structural safety (Hansen, 1956; Cornell, 1969). For practising engineers, limit state design implies safety verification using partial factors, which is a simplified, semi-probabilistic verification method.

While the applicability of limit state design to design of rock engineering structures has been questioned on theoretical grounds in some papers (see e.g. Harrison, 2019; Cao and Harrison, 2021), the current literature is largely in the state of inquiry. There are also issues that need to be studied further regarding the practical application of limit state design to rock engineering structures. For example, Spross et al. (2025) noted that there is little to no guidance on how to determine representative values (either as statistically determined *characteristic values* or their subjective counterpart *nominal values*) for common rock engineering properties. There is also no guidance on how to determine and address the uncertainty of geometrical properties such as the orientation of discontinuities.

For verification of rock slope stability, as well as sliding stability of concrete dams, the friction angle of the rock joint is a key parameter. In practice, the determination of the friction angle is often characterised by challenging sampling and consequently small datasets. In this context, there is a need to study how spatial variability in rock mass properties like the friction angle best is managed in the verification of limit states, i.e. the determination of their representative values. In this paper, we discuss specifically how representative values can be determined for friction angles of rock joints, for verification of the structural safety regarding sliding stability of concrete dams founded on rock.

2 Limit state design principles for sliding stability

According to limit state design concepts, structural failure is defined by the ultimate limit state, which can be described by a limit state function $G(\mathbf{X})$, where $\mathbf{X}$ contains uncertain properties which are described as random variables. When $G(\mathbf{X}) < 0$, failure occurs. Thus, the goal of the safety verification is to ensure that the probability $P(G(\mathbf{X}) < 0)$ is less than the acceptable failure probability p_{target}. For sliding stability of a concrete dam founded on rock, the limit state function can be described by $G = N' \tan\varphi - E$, where N' is the effective normal force to the sliding plane, $\tan\varphi$ is the friction coefficient of the rock joint, and E is the driving force (action) along the sliding plane. Cohesion is ignored for simplicity.

Applying partial factors in the safety verification EN 1990 (Eq. 8.1 in CEN, 2023), the design action E_d shall be smaller than the design resistance, R_d. Applying these concepts, we achieve

$$E_\text{d} \leq R_\text{d} \text{ for } E_\text{d} = \gamma_\text{F} F_\text{c} \text{ and } R_\text{d} = \frac{\tau_{dis}}{\gamma_\text{M}} = \frac{N' \tan(\varphi)_\text{rep}}{\gamma_\text{M}} \quad (1)$$

where γ_F and γ_M are partial factors, F_c is the characteristic value for the actions, τ_dis accounts for shear stress of rock-to-rock joint contact at failure, from an appropriate shear strength envelope, and $\tan(\varphi)_\text{rep}$ is the representative friction coefficient of the rock-to-rock joint. The representative value $\tan(\varphi)_\text{rep}$ can be determined either as a statistically evaluated characteristic value, or as a subjectively estimated nominal value. Both alternatives shall in principle yield the same representative value. In this paper, we focus on the determination of the characteristic value, for which Appendix A of EN 1997–1 provides some guidance. Acknowledging that the friction coefficient for the rock joint along which sliding takes place is both spatially varying and associated with epistemic uncertainty, the friction coefficient for the joint can be described as a random variable. Depending on whether the limit state is considered sensitive to spatial variability of the friction coefficient or not, the characteristic value is defined either by the 5$^{\text{th}}$ percentile of the probability distribution or by the average value of the random variable determined with 95% confidence. This distinction is discussed further in section 4.

Ideally, the characteristic value is determined from a large enough sample of laboratory tests of the friction angle from the relevant rock joint, allowing both the average value and the coefficient of variation to be statistically estimated from the data. The average value is estimated by its sample statistics $\overline{\tan\varphi} = \sum[\tan(\varphi)_i]/n$, where n is the number of independent samples. Note that the average of the angles $\bar{\varphi} \neq \text{atan}(\overline{\tan\varphi})$. The coefficient of variation is estimated from a probabilistic

characterisation of the uncertainty associated with the parameter. The total uncertainty expressed as a coefficient of variation, $V_{\tan\varphi}$, has four error components: natural variability, statistical error, measurement error, and transformation model error. The combined effect of these components can be assessed, approximately, as (JRC, 2024)

$$V_{\tan\varphi} \approx \sqrt{\Gamma^2 V_{\text{nat}}^2 + V_{\text{stat}}^2 + V_{\text{meas}}^2 + V_{\text{tr}}^2} \tag{2}$$

where the four squared V denote the respective coefficients of variations of the four error components, as indicated by the subscripts, and Γ is the variance reduction function, which takes value 1 when the limit state is sensitive to spatial variation and 0 when it is not. For the friction coefficient of a rock joint determined from laboratory shear tests, the measurement error and transformation model error are generally negligible; thus, only the natural variability and statistical error need to be accounted for. Depending on how well known the natural variability is, one can assume that $\tan(\varphi)$ is either normally distributed (for an assumed known variability) or Student's-t distributed (for unknown variability in which case the true variability σ^2 is approximated by the sample variance s^2). This gives two alternative equations (Equations 3a and 3b) for the characteristic value:

$$\tan(\varphi)_{c,N} = \overline{\tan(\varphi)}\left(1 - N_{95}\sqrt{\Gamma^2 V_{nat}^2 + V_{stat}^2}\right) = \overline{\tan(\varphi)} - N_{95}\sigma_{\tan(\varphi)}\sqrt{\Gamma^2 + \frac{1}{n}} \tag{3a}$$

$$\tan(\varphi)_{\text{c,Stud t}} = \overline{\tan(\varphi)}\left(1 - t_{95,n-1}\sqrt{\Gamma^2 V_{\text{nat}}^2 + V_{\text{stat}}^2}\right) = \overline{\tan(\varphi)} - t_{95,n-1}s_{\tan(\varphi)}\sqrt{\Gamma^2 + 1/n} \tag{3b}$$

where N_{95}denotes the one-sided 95% quantile of the standard normal distribution (i.e. $\Phi^{-1}(0.95) = 1.645$), and $t_{95,n-1}$denotes the one-sided 95% quantile of the Student's t distribution with$n - 1$ degrees of freedom. Equation 3a is simpler to evaluate than 3b but may underestimate the uncertainty especially when there are few samples. Depending on the testing program set-up, $s_{\tan(\varphi)}$, may also be challenging to determine accurately. If samples are taken from more than one discontinuity, $s_{\tan(\varphi)}$ will represent not only the within-joint variability of an identified governing sliding plane, but also between-joint variability of the averages across comparable joints in the geological domain. This variability is likely to be larger than the within-joint variability.

3 Illustrative example

To illustrate the practical challenges in determining the characteristic value of a friction coefficient, let us assume that the peak friction angle has been determined for 8 rock joint samples under a concrete dam (Figure 1).For this purpose, 8 laboratory shear tests on rock-to-rock contacts of naturally occurring discontinuities in grey coarse-grained granite are used (Table 1), as reported by Ríos-Bayona et al. (2021). The corresponding friction coefficients are presented for the peak friction angle, φ_p (°). Furthermore, we consider that the shear strength of the rock joint in the scale of the governing sliding plane can be represented as the corresponding mean value of a number of smaller laboratory direct shear tests, as shown by Rios Bayona et al. (2022).

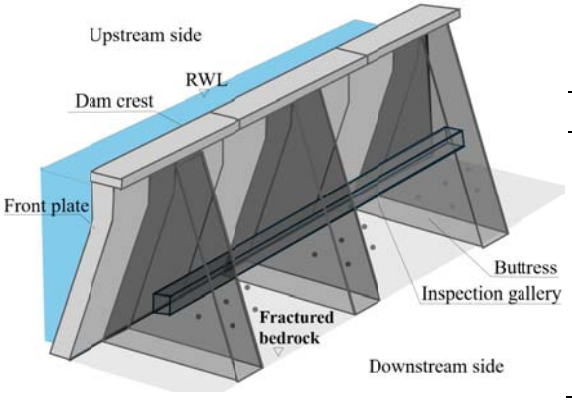


Fig. 1 Buttress dam founded on fractured bedrock.

Table 1. Friction coefficient values from 8 direct shear tests (retrieved from Ríos-Bayona et al. (2021)).

Test	Friction coefficient, $\tan(\varphi_p)$
1	tan(62.1) = 1.889
2	tan(50.5) = 1.213
3	tan(71.8) = 3.042
4	tan(63.1) = 1.971
5	tan(59.9) = 1.725
6	tan(53.5) = 1.351
7	tan(55.3) = 1.444
8	tan(58.1) = 1.607

The 8 determined friction coefficients, tan φ_p, displayed a mean value $\overline{\tan\varphi}$= 1.78, a sample standard deviations $_{\tan(\varphi)}$= 0.57, a maximum value of 3.04, and a minimum value of 1.21. (Subscript p for peak henceforth dropped for convenience). The difficulty in selecting a characteristic value does not lie solely in the statistical treatment of the data, but primarily in the geological representativeness of the samples. In rock engineering, uncertainty is often dominated by whether the tested discontinuities correspond to the actual governing sliding plane. To illustrate this distinction, three conceptual cases are considered, reflecting different assumptions in the probabilistic modelling.

Case 1: All 8 samples are taken from the relevant rock joint (A) where sliding is expected to occur. The $s_{\tan(\varphi)}$ captures the within-joint variability well, which however is of lesser importance as the mean value of the samples governs the limit state; thus Γ = 0.

Case 2: The 8 samples are obtained from 8 different comparable rock joints within the general area surrounding the dam. The specific rock joint forming the critical sliding plane is assumed unknown, which means that this is a prediction problem of sliding along a random joint (Γ = 1). In this case, $s_{\tan(\varphi)}$captures between-joint variability well but says nothing about the within-joint variability. Individual rock joints may exhibit significantly different frictional properties.

Case 3: Only 4 samples are obtained from the governing sliding plane (A), and the other 4 samples from comparable discontinuities in the same geological domain. While this likely is the most realistic sampling scenario, the evaluation of $s_{\tan(\varphi)}$ is less straightforward, as the samples represent a mix of both within-joint and between-joint variabilities. In fact, if plane A is known to be the governing sliding plane (**Case 3a**), it could be argued that the 4 samples from other joints have no effect: the problem reduces to Case 1 but with only n = 4 samples, and since Case 1 is governed by the mean value (Γ = 0), the between-joint variability is not of interest. In that case, the information from the other joints is not at all considered. With only 4 samples, the statistical uncertainty increases.

However, if the governing sliding plane is unknown (**Case 3b**), we again have a prediction problem, but now we get information about both between-joint variability and within-joint variability (albeit only from one joint). In principle, it can be argued that this can be approximated by Case 2, if we assume that $s_{\tan(\varphi)}$ evaluated from all joints now can represent both within-joint and between-joint variabilities. A more accurate assessment of Case 3b could be to set up a more advanced hierarchical model where both within-joint and between-joint variabilities are explicitly estimated, but that would require considerably more samples from all joints to be meaningful. Case 3b is therefore not further addressed in this paper.

Error! Reference source not found. and Figure 2 present representative values for the friction coefficient, determined as statistically evaluated characteristic values from the data in Table 1 for Case 1, 2 and 3a. Case 3a is represented by two results to illustrate the potential effect of random sampling, where (I) shows the result of "sampling" the four highest values of tan(φ) and (II) of "sampling" the four lowest values of tan(φ).

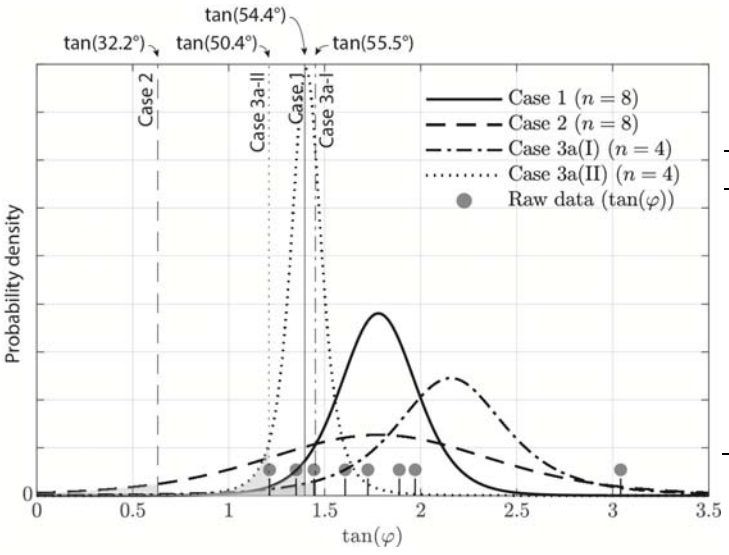


Table 2. Friction coefficients corresponding to the three cases, assuming either a normal or Student's-*t* distribution.

Case	Distribution	$\tan(\varphi)_{rep}$
1	Normal	1.45 = tan(55.4°)
2	Normal	0.782 = tan(38.0°)
3a (I)	Normal	1.66 = tan(59.0°)
3a (II)	Normal	1.27 = tan (51.7°)
1	Student's *t*	1.34 = tan(54.4°)
2	Student's *t*	0.631 = tan(32.2°)
3a (I)	Student's *t*	1.45 = tan(55.5°)
3a (II)	Student's *t*	1.21 = tan(50.4°)

Fig. 2 Estimated Student's-*t* probability distributions and evaluated characteristic values for the three cases.

4 Discussion

It is clear from the results in Figure 2 that the location of the samples and the understanding of the sliding plane governing the failure mode have a considerable effect on the calculated characteristic values. Taking samples from a rock joint that is known to be the governing sliding plane provides a considerably larger characteristic friction coefficient, than if the sliding plane is unknown. Thus, it may be more cost-effective to try to determine the governing sliding plane and take a few samples from that joint, than to try to estimate the between-joint variability by taking samples from many different joints in the area. This effect also seems to be larger than the difference between using a normal distribution and Student's-*t* distribution, at least for this type and amount of data. In this context, there is a note in EN 1997-1, paragraph A.8, saying that it is often "preferable" to use equation 3a with a conservative upper estimate of $\sigma_{\tan(\varphi)}$, than using equation 3b. The code clearly gives room for engineering judgement for handling the small sample problem. As we see in comparing Cases 3a (I) and (II), randomness in sampling can affect $s_{\tan\varphi}$ considerably in small samples. Particularly interesting is the result of Case 3a (II): Even though this case applied the four lowest values of tan(φ), the evaluated representative value is considerably larger than that of Case 2, which considered 8 samples. The reason lies in the relatively small $s_{\tan\varphi}$obtained when the four lowest samples are in a rather narrow range. This illustrates the importance of applying these statistical concepts correctly to the situation at hand, as well as the potential effect of randomness in small samples.

Worth noting is also that Eurocode 7 provides indicative values—table values—of $V_{\tan\varphi}$. For "all soils and rock", $V_{\tan\varphi}$ is in the range of 5–15% (EN1997-1, Table A.1). This indicative value directly gives the standard deviation in equation 3a via$\sigma_{\tan(\varphi)} = V_{\tan\varphi}\ \overline{\tan\varphi}$,as an alternative to determining $s_{\tan(\varphi)}$ from the test results. Using the indicative value might be particularly tempting when there is a lack of data; however, our evaluation of the 8 direct shear tests from Ríos-Bayona et al. (2021) indicates that the variability can be considerably higher than that (we had$V_{\tan\varphi}$= 32.1%), at least for this type of rock joints.

Another important issue is that we assumed that the peak shear strength of a rock joint can be represented by the mean value of a series of tests taken from a single fracture, following the approach of Rios Bayona et al. (2022). They showed that the large-scale peak shear strength is equal to the mean value obtained from small-scale (borehole) samples. However, whether this constitutes an appropriate design strategy remains a matter of debate. Several researchers have reported a negative scale effect, where the shear strength decreases with increasing scale (e.g., Pratt et al. 1974, Barton and Choubey 1977, and Bandis 1980). If such a negative scale effect exists, it means that our assumption may be unconservative. On the other hand, a comprehensive testing program recently carried out by Larsson et al. (2025) found no indication of a scale effect on the peak shear strength of rock joints. Nevertheless, further research is required before the assumption adopted in this paper—that the large-scale peak shear strength is equal to the mean value obtained from small-scale (borehole) samples—can be confidently applied in safety verification of dams.

Another comment is that the representative value of tan(φ) in this study was derived from results obtained from direct shear tests. A common alternative approach to estimate the peak shear strength is to take samples from a discontinuity, measure their general characteristics, and apply that information to a failure criterion. If this approach is adopted, a transformation model error V_{tr} must be considered according to Eq. (2), which will further increase the variability.

5 Conclusions

In this paper, an illustrative example was used to discuss how representative values according to Eurocode can be determined for the friction coefficient of rock joints, when verifying the structural safety of concrete dams against sliding in the rock foundation. The example, which uses the peak friction angle obtained from 8 laboratory direct shear tests as input, clearly illustrates not only the importance of having a sufficient number of samples, but also the need for a thorough understanding of whether the samples are taken from the governing sliding plane or are randomly sampled within the foundation. If this is not known, the design engineer may be forced to adopt a conservative approach, which significantly influence the representative value of the friction angle used in the design. This

highlights the importance of a sound understanding of the site geology, adequate site investigations, and the development of a robust ground model.

The discussed Case 3b provides the most complex collection of samples to evaluate statistically. This illustrates the fine balance between statistical rigour and practical feasibility—we see many potential pitfalls! Providing a reasonable sampling strategy for that situation and recommendation for the evaluation of the test results is an important topic for future studies.

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