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Shear strength criteria for rock, rock joints, rockfill and rock masses: Problems and some solutions

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ABSTRACT

Although many intact rock types can be very strong, a critical confining pressure can eventually be reached in triaxial testing, such that the Mohr shear strength envelope becomes horizontal. This critical state has recently been better defined, and correct curvature or correct deviation from linear Mohr–Coulomb (M-C) has finally been found. Standard shear testing procedures for rock joints, using multiple testing of the same sample, in case of insufficient samples, can be shown to exaggerate apparent cohesion. Even rough joints do not have any cohesion, but instead have very high friction angles at low stress, due to strong dilation. Rock masses, implying problems of large-scale interaction with engineering structures, may have both cohesive and frictional strength components. However, it is not correct to add these, following linear M-C or nonlinear Hoek–Brown (H-B) standard routines. Cohesion is broken at small strain, while friction is mobilized at larger strain and remains to the end of the shear deformation. The criterion '*c then* σ_n *tan* φ' should replace '*c plus* σ_n*tan* φ' for improved fit to reality. Transformation of principal stresses to a shear plane seems to ignore mobilized dilation, and caused great experimental difficulties until understood. There seems to be plenty of room for continued research, so that errors of judgement of the last 50 years can be corrected.

1. Introduction

Non-linear shear strength envelopes for intact rock and for (non-planar) rock joints are the reality, but traditional shear test interpretation and numerical modelling in rock mechanics has ignored this for a long time. The non-linear Hoek–Brown (H-B) criterion for intact rock was eventually adopted, and many have also used the non-linear shear strength criterion for rock joints, using the Barton and Choubey (1977) wall-roughness and wall-strength parameters JRC (joint roughness coefficient) and JCS (joint compressive strength).

Non-linearity is also the rule for the peak shear strength of rockfill. It is therefore somewhat remarkable why so many are still wedded to the '$c+\sigma_n\tan\varphi$' linear strength envelope format. Simplicity is hardly a substitute for reality. Fig. 1 illustrates a series of simple strength criteria that predate H-B, and that are distinctly different from Mohr–Coulomb (M-C), due to their non-linearity.

The actual shear strength of rock masses, meaning the prior failure of the intact bridges and then shear on the fractures and joints at larger strains, is shown in Fig. 1 (units of σ_1 and σ_2 are in MPa).

2. Intact rock

The three-component based empirical equations (using roughness, wall strength and friction) shown in Fig. 1 were mostly derived in Barton (1976). The similarity of shear strength for rock joints and rockfill was demonstrated later in Barton and Kjærnsli (1981).

At the time of this mid-seventies research by the writer, it was recognized that the shear strength envelopes for intact rock, when tested over a wide range of confining stress, would have marked curvature, and eventually reach a horizontal stage with no further increase in strength. This was termed the 'critical state', and the simple relation $\sigma_1 = 3\sigma_3$ suggested itself, as illustrated in Fig. 2.

An extensive recent study by Singh et al. (2011) in Roorkee University involving re-analysis of thousands of reported triaxial tests, including their own testing contributions, has revealed the

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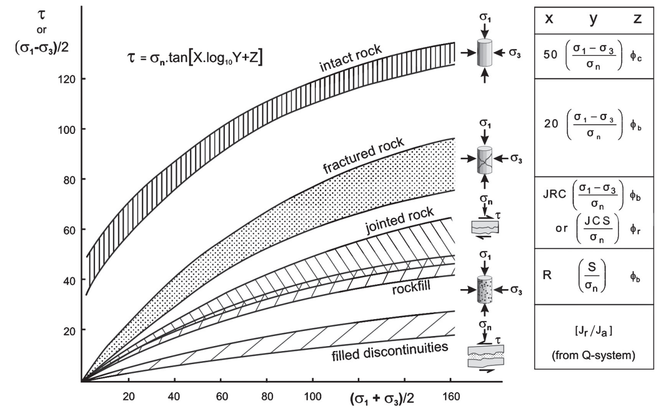

Fig. 1. Simple empiricism, sometimes based on hundreds of test samples, suggested these simple ways to express peak shear strength (τ). Note the general lack of cohesion (Barton, 1976).

astonishing simplicity of the following equality: $\sigma_c \approx \sigma_{3(\text{critical})}$ for the majority of rock types: in other words, the two Mohr circles referred to in Fig. 2 are touching at their circumference. This is at once an 'obvious' result *and* an elegantly simple result, and heralds a new era of triaxial testing.

The curvature of peak shear strength envelopes is now more correctly described, so that few triaxial tests are required and need only be performed at low confining stress, in order to delineate the whole strength envelope. This simplicity does not of course apply to M-C, nor does it apply to non-linear criteria including H-B, where triaxial tests are required over a wide range of confining stress, in order to correct the envelope, usually to adjust to greater local curvature.

Singh et al. (2011) basically modified the M-C criterion by absorbing the critical state defined in Barton (1976), and then quantifying the necessary deviation from the linear form, using a large body of experimental test data.

Singh and Singh (2012) have developed a similar criterion for the shear strength of rock masses, with σ_c for the rock mass potentially based on the simple formula $5\gamma Q_c{}^{1/3}$ (where $Q_c = Q\sigma_c/100$ (MPa)). The rock density is γ, and Q is the rock mass quality (Barton et al., 1974), based on six parameters involving relative block size, inter-block friction coefficient and active stress.

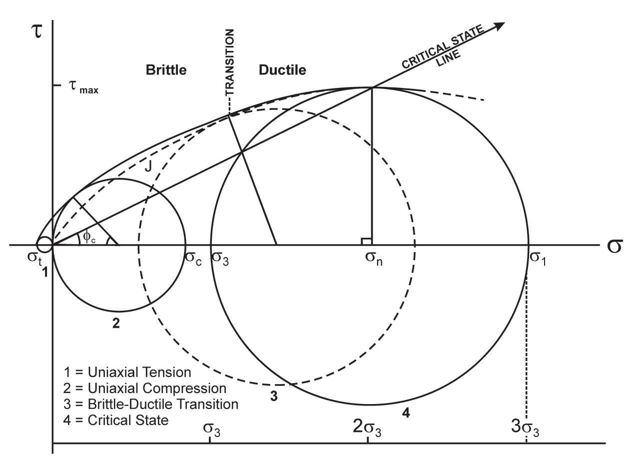

Fig. 2. Critical state line defined by $\sigma_1 = 3\sigma_3$ was suggested by numerous high-pressure triaxial strength tests. Note the chance closeness of the unconfined strength (σ_c) circle to the confining pressure $\sigma_{3(\text{critical})}$ (Barton, 1976). Note that 'J' represents jointed rock. The magnitude of φ_c is 26.6° when $\sigma_1 = 3\sigma_3$.

3. Shear strength of rock joints

Recent drafts of the ISRM suggested methods for testing rock joints, and widely circulated errors on the Internet and in commercial numerical modelling software, caused the writer to spend some time on the topic of shear strength of rock joints, in his 6th Müller Lecture (Barton, 2011). Problems identified included exaggeration of 'cohesion intercept' in multi-stage testing, and continued use of φ_b in place of φ_r, thirty-five years after φ_r was introduced in a standard equation for shear strength.

Unfortunately, Hoek's downloadable rock mechanics texts and related RockScience software represent the limit of a lot of consulting offices contact with rock mechanics, so they have little knowledge of advances in the field that are not picked up by those who for some reason feel it their duty to feed the internet with 'free' rock mechanics. This is a dangerous and unnecessary state of affairs.

Following the tests on 130 fresh and slightly weathered rock joints (ten of which are shown in Fig. 3), the basic friction φ_b was replaced by φ_r, which may be several degrees lower. This occurred in 1977, and was unfortunately overlooked/not read by the chief supplier of the Internet with his version of rock mechanics.

Due to the dominance of this 'downloadable rock mechanics', there have been a significant number of incorrectly analyzed rock slopes, and incorrectly back-calculated JRC values in refereed Ph.D. studies, not to mention a number of refereed publications with incorrect formula, due to failure to read outside the downloaded materials.

The reconstructed shearing events shown in Fig. 4 were derived from specific tension fractures with the (two-dimensional, 2D) surface roughness as shown, and displaced and dilated as measured in the specific direct shear tests. These tests on tension fractures were performed in 1968, and represented the forerunner of the non-linear criterion shown in Fig. 4 (#3).

In 1971 (Ph.D. studies of the writer), the 'future' 'JRC' had the value 20, due to the roughness of tension fractures, and the 'future' 'JCS' was merely the uniaxial strength of the (unweathered) model material. For the same reason of lack of weathering, the 'future' φ_r at this time was simply φ_b.

Fig. 5 illustrates the form of the third strength criterion shown in Fig. 4(top). It will be noted that no cohesion intercept is intended. It will also be noted that subscripts have been added to indicate scale-effect (reduced) values of joint roughness JRC_n and joint wall strength JCS_n. This form is known as the Barton–Bandis criterion. Its effect on strength–displacement modelling is shown later.

The scale-effect correction by Barton and Bandis (1982) is illustrated by three peak shear strength envelopes in Fig. 5. It will be noted that the peak dilation angles vary significantly. This is important when transforming principal stresses to normal and shear stresses that act on a plane. This topic will be discussed later.

Recent drafts and earlier versions of the ISRM suggested methods for shear testing rock joints have suggested multi-stage testing of the same sample, to increase the numbers of test results when there are insufficient samples. Naturally, the first test is recommended performed at low stress to minimize damage. Successive tests are performed at higher normal stress, using the same sample, reset in the 'zero-displacement' position. Since there will be a gradual accumulation of damage, there is already a 'built-in' tendency to reduce friction (and dilation) at higher stress, and therefore to increase the apparent cohesion intercept (if using M-C interpretation). These problems are accentuated if JRC is high, and JCS low and normal stress high in relation to JCS, therefore causing more damage during each test.

A further tendency to rotate the 'peak' strength envelope clockwise (and exaggerate an actually non-existent M-C cohesion) is