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SHEAR FAILURE IN ROCK USING DIFFERENT CONSTANT NORMAL LOAD

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Abstract

SEILDEL – HABERFIELD's [1] work was based on the analysis of the regular triangular asperities and assumed that the asperities were rigid. They used normal stiffness shear stress (CNS) to measure the joint dilatation rate and extended the LADANYI – ARCHAMBAULT's [2] approach. The aim of this research is to investigate the dependence of the constant normal load (CNL) on the rate of the dilatation. Three points were chosen on the bilinear failure envelope: the first in the first linear part, the second in transition stress, and the third in the second part of the bilinear failure envelope. 12 regular triangular cement mortar specimens were used to carry out this research. These cement mortar specimens were investigated by TISA – KOVARI [3] before so all the material and shearing constants were well-known. The other purpose of this research was to observe the influence of the normal stress on the dilatation–displacement curves.

Keywords: direct shear test, constant normal load, shear–dilatation and shear–displacement, rock joints failure envelope.

1. Introduction

Generally rock is not homogeneous and isotropic due to the presence of discontinuities. Therefore for practical applications a distinction between the terms 'rock material' and 'rock mass' has to be made. Rock material refers to the continuous body found between joints, faults, etc. and consists of one or more lithologic units which form a structure framework and may or may not contain joints, faults, etc. The term 'discontinuity' refers to a separation surface, i.e. a break in the spatial continuity of a material. In the case of rock mass discontinuities can be present in the form of joints, faults, cavities, stratification surfaces and shistosity surfaces. The prediction of the shear strength developed in rock joint undergoing shear displacements is a subject which has brought increased attention in recent years.

Stability analysis in jointed rock is often governed by the stability of a critical joint. In such cases, it is assumed that no interaction exists between the rock and the joint; the joint can therefore be considered isolated. The mechanical characteristics of the joint plane are necessary for assessing instability. The direct shear test is a suitable experiment to determine these parameters. Apart from a few exceptions, the normal force has been generally held constant in shear tests reported in the literature.

This experimental boundary condition corresponds *in situ* to a rock block slide at the surface (*Fig. 1*). The block is able to slide freely and the normal force acting on the joint remains constant during the slide.

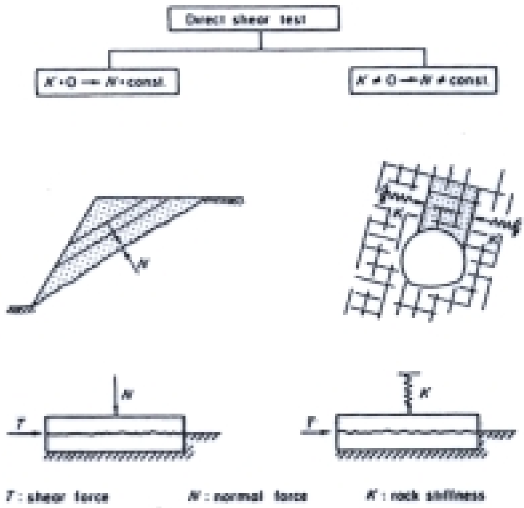

Fig. 1. Comparing the rock joint strength envelope

When non-planar surfaces are in sliding contact, they tend to dilate if the boundary conditions permit such deformation. The amount of dilatation depends on the geometry of the contacting surfaces, normal stiffness of the joint, deformability of the surrounding media. Dilatation causes enhanced shear strength due to the increased normal stress. Proper modelling of the shear behaviour of rock discontinuities must take into account the conditions imposed by the rock mass rigidity. In this situation, the normal stress path should be properly understood for the accurate prediction of shear strength. Such behaviour is more realistically represented in the controlled normal stiffness shear tests (CNS) rather than in the conventional constant normal load (CNL) tests.

2. Theoretical Background

PATTON [4] performed a series of constant load stress direct shear tests with regular teeth inclination, i, at varying normal stresses. From these tests he established a bilinear failure envelope – failure from an asperity sliding and asperity shearing mode. The equations for the two portions of the failure envelope are given below:

$$\tau = \sigma_n \tan\left(\phi_\mu + i\right), \tag{1}$$

$$\tau = c_j + \sigma_n \tan\phi_\mu, \tag{2}$$

where

c_j = cohesion;
ϕ_μ = basic friction angle;
i = regular teeth inclination.

LADANYI – ARCHAMBAULT [2] provided an extension to Paton's model to account for the sliding and shearing mechanisms found in natural rock joints. They considered the shear resistance of joints comprising regular triangular asperities with asperity angles of i. They suggested that the joint strength of natural joints could be determined from the applied normal stress, the basic angle of rock and the joint dilatation rate. They used the energy principles and rewrote the Patton's equation in the asperity sliding mode as

$$\tau = \sigma_n \tan\left(\phi_\mu + \dot{\nu}\right), \tag{3}$$

where

ϕ_μ = basic friction angle;
$\dot{\nu}$ = the rate of dilatation at failure.

SEIDEL – HABERFIELD [1] analyzed using extensions of LADANYI – ARCHAMBAULT's [2] approach. They suggested that the shear resistance of multi-asperity profiles in degrading rocks with varying asperity angles should be computed by combining the individual shear resistance of each individual asperity face as follows:

$$\tau = \sigma_n \tan\left[\frac{\tan i + \tan\phi_\mu}{1 - \tan\phi_\mu}\right], \tag{4}$$

where

ϕ_μ = basic friction angle;
$\dot{\nu}$ = the rate of dilatation at failure,
i = regular teeth inclination.

According to (4) the effective friction angle is given:

$$\phi_{\text{deg rade}} = \frac{\tan i + \tan\phi_\mu}{1 - \tan\phi_\mu}. \tag{5}$$

Even though the bilinear model is easier to use, the rock joint strength is better characterized by a curve than by a bilinear envelope. JAEGER'S [5] equation has been proposed for this conceptually simple case, and the contributions to joint strength from different factors can be separated easily. Jaeger's basic equation is shown with Patton's equation in *Fig. 2* and is given by:

$$\tau = c_j\left[1 - e^{b\sigma_n/q_u}\right] + \sigma_n \tan\phi_\mu, \tag{6}$$

where

c_j = cohesion;
b = empirical constant;
σ_n = normal stress;
q_u = uniaxial compressive strength;
ϕ_μ = basic friction angle.

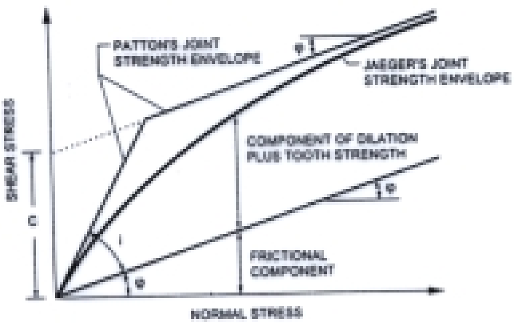

Fig. 2. Schematical view of the shear apparatus used in combination with a loading machine

3. Test Equipment

The CNL (constant normal load) equipment was designed and built in the Rock Mechanics Laboratory of the Swiss Federal Institute of Technology, Zurich [3]. To transmit force to the specimen it was cast in epoxy resin in the usual manner. The shear box fits exactly between two massive L-shaped steel pieces (*Fig. 3*). The normal force (N) and shear force (S) were applied through one of these L-shaped steel pieces. The other L-shaped steel piece provided the reactions, which were the lower loading plates of the servo-controlled press. This was achieved by resting the L-shaped steel piece against the front side of the frame as well as on a cylindrical bearing box with rests on the lower loading plate.

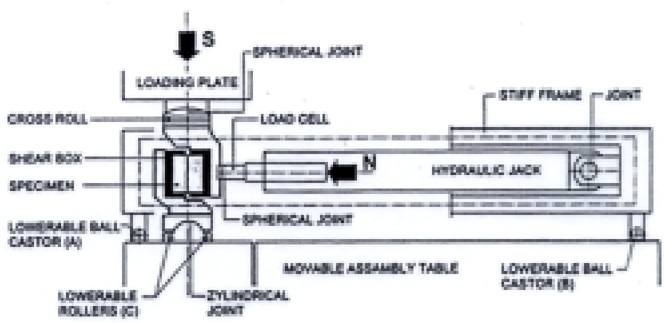

Fig. 3. A typical stress–strain and dilatation–displacement curve

The shear apparatus is designed in such a way that both parts of the test specimen can be displaced and rotated with respect to each other without constraints. This is made possible by load transfer through the cross roll and the spherical joint arrangement. The center of the sphere is aligned with the center of the joint surface at the start of the test.

A fairly long hydraulic jack was chosen in order to minimize the component of the jack force due to its rotation under the action of the shear deformations. The hinged load cell placed between the sack and the specimen was selected on account of its high accuracy, simple calibration and its direct electrical signal. The measuring system consists of load cells (S and N) and 6 displacement transducers. The measurement of the relative displacement of the joint surfaces in the directions of N and S is carried out directly at the joint itself, on both sides of test specimen. The maximum normal stress and shear stress which can be applied to a joint surface of the size of 24×12 cm are 20 N/mm^2 and 100 N/mm^2, respectively.

4. Material Description

The interpretation of the results of shear tests on real rocks is usually complicated by sample variability. To overcome this difficulty the laboratory specimens were made from an artificial material so that the shape, size and internal strength of the irregularities or 'teeth' on the surface of the test specimen could be evaluated separately.

Cement mortar was selected as test material as it has rock-like properties. *Table 1* presents the physical properties of the cement mortar. Basic shearing constants were determined in the previous researches of TISA – KOVARI [3].

Table 1. Physico-mechanical properties of the cement mortar

Physico-mechanical properties	Index	Unit	Value
Modulus of elasticity	E	GPa	17.5
Poisson's ratio	ν	–	0.1
Maximum stress	σ	MPa	60
Tensile stress	σ_t	MPa	6.5
Basic friction angle	ϕ_μ	degree	33.64
Asperity sliding angle	α	degree	59
Cohesion	c_j	N/mm^2	2.13
Transition stress	σ_T	N/mm^2	2.1
Density	γ	g/cm^3	2.2

All specimens were 150 mm wide and 140 mm long with four regular teeth where the distance between the teeth was kept constant at 20 mm. The teeth were 5 mm high, 15 mm thick at the bottom and the inclination angle of the teeth was 26.57°.

5. Results and Discussion

Using cement mortar joints, 12 tests were carried out with 3 different normal stress values: 1.05; 2.10 and 3.15 N/mm^2. These values were chosen because the aim of this research was to investigate the influence of the normal load in three points: in the first linear part, in the transition stress and in the second part of the bilinear failure envelope. The measured maximum and residual stresses were in the known lines. The measured results with the variances are summarized in *Table 2.*

Table 2. Measured results with variances in parentheses

Normal force σ_n N/mm^2	Peak shear stress τ_p N/mm^2	Res. shear stress τ_r N/mm^2	$\tan^{-1}\left(\tau_p/\sigma_n\right)$	$\tan^{-1}(\tau_r/\sigma_n)$
1.05	1.71 (0.19)	0.72 (0.10)	58.45 (2.88)	34.43 (3.14)
2.1	2.99 (0.28)	1.29 (0.07)	54.91 (2.55)	31.71 (1.46)
3.15	3.19 (0.27)	2.03 (0.09)	45.36 (2.31)	32.74 (1.19)

Table 3 compares the effective friction angles calculated by different methods according to *Eq.* (4) [1], *Eq.* (3) [2] (the variance in the bracket). The effective friction angle was 60.21° according to Patton's *Eq.* (1).

Table 3. The measured and calculated results with the different theories

Normal force σ_n N/mm^2	Measured dilatation angle $\dot{\nu}$	*Eq.* (5) $\phi_{degrade}$	*Eq.* (3) $\phi_\mu + \dot{\nu}$
1.05	24.76 (1.19)	59.07 (0.82)	58.40 (1.19)
2.10	22.42 (1.18)	58.11 (0.57)	56.06 (1.18)
3.15	12.48 (2.44)	53.83 (0.95)	46.12 (2.44)

Comparing the results of the CNS tests, in the case of *Eq.* (3) all three points are closer to the original than the others. This *Eq.* (3) is correct where the other equations fail. This means that this is a more general equation for the measured dilatation–displacement curve and it should be valid until the 'teeth' or irregularities are shorn off or till the deformation of the teeth can be measured. This is possibly the point where the Jaeger's joint failure envelope curve (*Eq.* (5)) 'touches' the Patton's linear curve.

HABERFIELD – JOHNSTONE [6] idealized the shear behaviour of concrete–rock joints under CNS conditions. The other aim was to study the character of the dilatation–displacement curve under CNL conditions. The rate of dilatation is not the maximum at peak shear response (because of the sliding) and continues to increase at a decreased rate until the minimum shear strength is reached. *Fig. 4* shows a typical shear stress vs. displacement and dilatation vs. displacement relation of a typical specimen. All the dilatation–displacement curves can be divided into three parts: in the first part (I) during the increasing strain-stress the angle of the teeth is constant. This angle is not changing in the second part (II) where the stress is decreasing. In the third part (III) during the decreasing shear-stress the changing of the angle is successive and it becomes linear in the residual part of the shear-stress. The maximum dilatation is, of course, decreasing with the increased normal load. In this case the average maximum dilatation was 1.10 mm at the 1.05 N/mm^2 normal load, 0.67 mm at 2.1 N/mm^2 and 0.37 mm at the 3.15 N/mm^2 applied normal load.

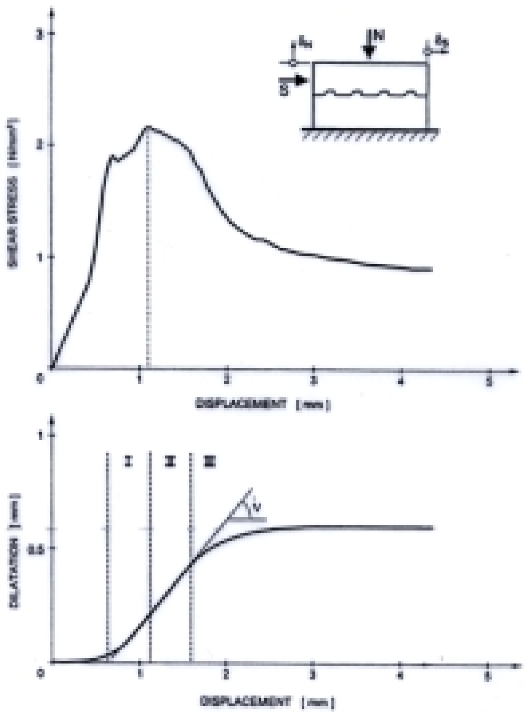

Fig. 4.

6. Conclusion

The results show that the measured dilatation angle ($\dot{\nu}$) decreases with the increased normal force and it is always present. However, *Eq.* (3) is also correct for the cases when the Patton’s and the Seidel and Haberfield equations fail. This means that this is a more general equation and it should be valid until the 'teeth' (or irregularities) are not shorn off. This point is not at the transition stress, but in the meeting point of the Jaeger's curve and the bilinear curve. The measured dilatation–displacement curves show that after the peak stress the rate of dilatation does not change for a long time.

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References

[1] SEIDEL, J. P. – HABERFIELD, C. M. (1995): The Application of Energy Principles to the Determination of the Sliding Resistance of Rock Joints. *Rock Mech. Rock Engng.,* Vol. 28/4, pp. 211–226.

[2] LADANYI, B. – ARCHAMBAULT, G. (1970): Simulation of Shear Behavior of Jointed Rock Mass. *Proc. 11th Symp. on Rock Mechanics: Theory and Practice, AIME,* New York, pp. 105–125.

[3] TISA, A. – KOVARI, K. (1984): Continuous Failure State Direct Shear Tests. *Rock Mech. Rock Engng.,* Vol. 17/2, pp. 83–95.

[4] PATTON, F. D. (1966): Multiple Modes of Shear Failure in Rock. *1st Congress of Int. Soc. Rock Mech.,* Lisbon, Vol. 1, pp. 509–513.

[5] JAEGER, J. C. (1971): Friction of Rocks and Stability of Rock Slopes. *Geotechnique,* Vol. 21/2, pp. 97–134.

[6] HABERFIELD, C. M. – JOHNSTONE, I. W. (1994): A Mechanistically-based Model for Rough Rock Joints. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* Vol. 31/4, pp. 279–292.