International Journal of Mining Science and Technology 26 (2016) 577–580

Contents lists available at ScienceDirect

International Journal of Mining Science and Technology

journal homepage: www.elsevier.com/locate/ijmst

A modified model of a single rock joint's shear behavior in limestone specimens

Dindarloo Saeid R. [a,*], Siami-Irdemoosa Elnaz [b]

[a] Department of Mining and Nuclear Engineering, Missouri University of Science and Technology, Rolla 65409, USA
[b] Department of Geosciences and Geological and Petroleum Engineering, Missouri University of Science and Technology, Rolla 65409, USA

ARTICLE INFO

Article history:
Received 6 December 2015
Received in revised form 13 January 2016
Accepted 10 March 2016
Available online 24 May 2016

Keywords:
Limestone rock joints
Constant normal load
Direct shear test
Shear behavior

ABSTRACT

The shear behavior of a single rock joint in limestone specimens, under a constant normal load (CNL), was analyzed in this study. Test specimens with different asperity roughness were prepared and tested. Goodman's model of a rock joint's shear behavior, under CNL, was modified to render a better representation of the data obtained. The model's applicability was validated. The proposed model showed better correlation with experimental data. It also, requires fewer variables. The steps to calculate all the necessary variables for the model are discussed.

1. Introduction

Rock joints are mechanical discontinuities that have geological origins. In general, the strength and deformability properties of these discontinuities are quite different from those of intact rock. In many cases, the discontinuities completely dominate both the shear and the deformation behavior of the in situ rock mass in given stress conditions [1,2]. Engineers in the mining, civil, and petroleum industries often face problems that are associated with jointed rock masses. Rock joint's shear behavior must be examined comprehensively to understand the jointed rock mass mechanical behavior. Many applications could benefit from the study of joints at a smaller scale, such as petroleum and energy recovery applications [3]. A number of researchers have tried to model the shear behavior of a single rock joint under laboratory conditions-most use the direct shear test. The test is conducted under two major boundary conditions. A direct shear test under constant normal load (CNL) and a direct shear test under constant normal stiffness (CNS). A CNL is used when the rock can dilate freely i.e. with constant normal load under shear displacement. This situation is typically encountered in surface rock structures such as rock slopes. In case, the joint is constrained with surroundings materials and cannot dilate freely upon shearing, the normal load will increase. This load's curve is controlled by the stiffness of surrounding rocks. The CNS condition is typically encountered in deep underground cavitations. The shear behavior of rock joints is not simply controlled by boundary conditions (i.e., either CNL or CNS). It is also controlled by a number of other important factors, including the intact rock properties, joint roughness, shear rate, and filling materials [4,5].

A comprehensive mathematical model that considers all of these effective variables has not been developed. The application of experimental methods and models is necessary to addressing the difficulties of modeling this complex behavior analytically [6]. Experimental results are useful both in modeling and calibrating several of the model's parameters. They are also useful in validating the results. Direct shear tests under the CNL condition were conducted on natural rock joints in this study. The results were used to render an experimental equation for the shear behavior. Tests specifications, specimens, and materials are introduced in Section 2. The Goodman's model under the CNL condition and the proposed model are discussed in Sections 3 and 4. Finally, Section 5 concludes the paper.

2. Specimens and tests specifications

Fourteen limestone specimens were collected and prepared for the purpose of understanding the shear behavior of joints in limestone rocks. These specimens were collected from a dam site located inside a limestone zone. The direct shear test procedure conducted by Bandis et al. [7] was used in this study. The material's basic properties were examined through a series of direct shear tests on solid blocks. The basic sliding resistance tests were

* Corresponding author. Tel.: +1 573 2010737.
E-mail address: srd5zb@mst.edu (S.R. Dindarloo).

http://dx.doi.org/10.1016/j.ijmst.2016.05.007
2095-2686/

Nomenclature

τ	shear stress	s_r	shear strength of asperities
τ_p	peak shear strength	k_1, k_2	empirical constants in Ladanyi archambault
τ_r	residual shear strength	C_1, C	first and second constants of the experiment in the proposed model
u	shear displacement	$\dot{\nu}$	secant rate of dilatancy at peak shear strength
u_p	shear displacement at peak shear strength	i	$\arctan(\dot{\nu})$
u_r	residual displacement	i_m	triangular asperity angle
σ_n	normal stress	Z	shear strength factor in the proposed model
σ_T	transitional stress in Ladanyi_Archambault		
φ	internal friction angle		
a_s	proportion of total joint area sheared through asperities		

performed on planar solid surface under various normal stresses. The shear displacement rate was 0.5 mm/min. Triangular asperities with different angles (from 4° to 20°) were presented in the samples. These asperity angles have considerable effect on the shear's behavior [8].

3. Model description

The three-line model proposed by Goodman [9] served as the mathematical model's starting point. Eqs. (1)–(3) define the shear stress versus shear displacement for the region before peak shear strength, between peak and residual shear strengths, and after residual, respectively.

$$\tau = \frac{\tau_p}{u_p}u, \quad u < u_p \tag{1}$$

$$\tau = \left(\frac{\tau_p - \tau_r}{u_p - u_r}\right)u + \left(\frac{\tau_r u_p - \tau_p u_r}{u_p - u_r}\right), \quad u_p \leqslant u \leqslant u_r \tag{2}$$

$$\tau = \tau_r, \quad u > u_r \tag{3}$$

These equations are plotted in Fig. 1.

In this study, a two-domain model is proposed. The first part is the same as Goodman's, as it predicts the joint behavior almost exactly in the same way as the experimental results. However, the second and third parts of Goodman's are simplified mathematical representation of the actual behavior. These deviated from the actual test results in this case. In this paper, the after region peak is modeled as a non-linear functional relationship with a shear displacement in form of $\tau \propto \frac{1}{u}$ as calculated in Eq. (5). The curve in Fig. 2 has a better correlation than Fig. 1 with experimental results. The proposed model is defined as below:

$$\tau = \frac{\tau_p}{u_p}u, \quad 0 \leqslant u \leqslant u_p \tag{4}$$

$$\tau = \tau_r + (\tau_p - \tau_r)\frac{u_p}{u}, \quad u > u_p \tag{5}$$

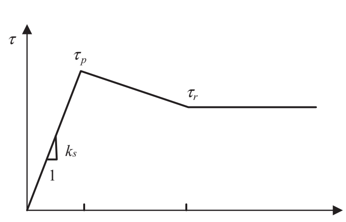

Fig. 1. Goodman's model for the shear behavior of rock joints.

Only three parameters are required for the complete mathematical modeling of a single joint's shear behavior under the CNL condition, as verified in (4) and (5). These parameters include the peak shear strength, the residual shear strength, and shear displacement at the peak shear strength. Hence, these three parameters should be defined from the intact rock properties, the joint geometry, the mechanical properties, loading type, and the loading rate. Saeb and Amadei [10] applied (6) to estimate the peak shear strength.

$$\tau_p = \sigma_n \tan(\varphi + i)(1 - a_s) + a_s s_r \tag{6}$$

where a_s and S_r are the proportion of the total joint area sheared through asperities and the shear strength of asperities, respectively. An accurate measurement of these parameters is not practical, particularly in in-situ tests. Ladanyi and Archambault [11] proposed that the following formula be used to calculate a_s and i. This requires determination of further unknowns [10] as shown in (7)–(9).

$$a = 1 - \left(1 - \frac{\sigma_n}{\sigma_T}\right)^{k_1} \tag{7}$$

$$i = \arctan(\dot{\nu}) \tag{8}$$

$$\dot{\nu} = \left(1 - \frac{\sigma_n}{\sigma_T}\right)^{k_2} \cdot \tan(i_0) \tag{9}$$

where k_1 and k_2 are empirical constants and σ_T is a transitional stress. The uniaxial compressive strength of the intact rock can be used as an estimate of σ_T [11].

More unknowns need to be defined before τ_p in (6) can be calculated. Several of these parameters can be measured accurately; several are only estimations. Thus, the application of (6) does not guarantee exact results that are comparable to actual ones obtained either by laboratory or in-situ tests. Eq. (10) was used in this study to obtain a sufficiently accurate estimate of the peak shear strength while using minimum number of variables. The values obtained from (10) were compared with the actual values obtained and plotted in Fig. 3. This shows a very good agreement between the two data sets.

$$\tau_p = \sigma_n \tan(\varphi + i_m) \tag{10}$$

Goodman proposed the following model for τ_r at different normal stresses [9].

$$\tau_r = \tau_p\left(B_0 + \frac{1 - B_0}{\sigma_T}\sigma_n\right), \quad \text{for } \sigma_n < \sigma_T \tag{11}$$

where B_0 is the ratio of the residual strength to the peak shear strength at a zero normal stress and Eq. (12) holds for the residual strength.