International Journal of Rock Mechanics & Mining Sciences 90 (2016) 62–73

Contents lists available at ScienceDirect

International Journal of Rock Mechanics Mining Sciences

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

Anisotropic shear behavior of rock joint replicas

Rakesh Kumar, Abhiram Kumar Verma*

Department of Mining Engineering, Indian Institute of Technology Kharagpur, Kharagpur 721302, West Bengal, India

ARTICLE INFO

Keywords:
Anisotropic shear strength
Direct shear test
Joint roughness coefficient
Constant normal load condition

ABSTRACT

In this study, an anisotropic shear behavior of joint replicas is investigated under constant normal load condition (*CNL*). A new anisotropic peak shear strength model is proposed incorporating actual peak dilation angle and shear component of asperities. In this model, peak dilation angle is modified with mean plane to find out actual dilation angle and shear component is back calculated from experimental results. The results show that as the ratio of normal stress to compressive strength increases, peak dilation angle decreases exponentially whereas shear component increases in power function form. Further, an empirical model for estimation of peak shear displacement is proposed based on maximum asperity angle and friction angle. Afterwords, shear stiffness is calculated as a ratio of predicted shear strength and peak shear displacement. To conduct this study, three different natural joint roughness is transferred to silicon rubber molds and these molds are used to make joint replicas of mated joint of 90 mm diameter and 50 mm height by mixture of cement, sand, and water in the ratio of 1:1.5:0.45 by weight. The surface cloud of joint surface are generated using 3D non-contact type profiler. In this study, total 144 direct shear tests are conducted on prepared joint replicas using four normal stresses (0.25, 0.5, 1, and 1.5 MPa).

1. Introduction

Joint is a line of break of geological origin along which there is no visible displacement. It is a three dimensional discontinuity composed of two matched/mismatched surfaces. The presence of joints in rock mass plays an important role to define its overall shear strength, deformability behavior, in-situ stresses, and hydro-geological properties. The shear strength of rock joints is important in the design of near surface/deep geotechnical works (mining excavation, dam foundation, power plants, underground caverns, and slopes).

Patton[1] developed first bilinear model for saw toothed joints and suggested that at low normal stress, shear strength of joints is only due to sliding over asperity angle (i) as shown in Eq. (1) while at high normal stress, tips of asperities are sheared off and shear strength of rock is expressed with cohesion (c_j) and residual friction angle (ϕ_r) as given in Eq. (2).

$$\tau = \sigma_n \tan(\phi_b + i) \tag{1}$$

$$\tau = c_j + \sigma_n \tan\phi_r \tag{2}$$

Ladanyi and Archambault [2] proposed nonlinear shear strength model by identifying the areas on the joint surface where sliding and shearing of asperities are most likely to occur. Later on, Saeb[3] used stress-dilatancy theory of sand and simplified the Ladanyi and Archambault's shear strength model. Barton[4] carried out experiments on natural rough joints and explained that friction angle (ϕ_j) along joint surface is sum of basic friction angle (ϕ_b), roughness component or peak dilation angle (d_n), and shear component (S_n) as given in Eq. (3).

$$\phi_j = \phi_b + d_n + S_n \tag{3}$$

Here, basic friction angle represents the minimum resistance between two flat, unpolished rock surfaces whereas peak dilation angle and shear component are related to inclination of asperities (roughness) and failure of asperities, respectively. Based on experimental results, Barton proposed an empirical model to predict shear strength of rock joints as shown in Eq. (4).

$$\tau = \sigma_n \tan\left[\phi_b + JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right)\right] \tag{4}$$

Where, *JRC* is the joint roughness coefficient and *JCS* is the joint wall strength which is equal to compressive strength of rock. *JRC* can be estimated either by back calculation of direct shear tests results or by visual comparison with ten standard profiles given by Barton and Choubey.[5]

In literature, many constitutive models [6–24] are developed to predict shear strength of rock joints under *CNL* condition. The movement along rock joints in foundations, dams, tunnels, slopes, and in underground excavation can occur in any direction, depending on

* Corresponding author.
E-mail addresses: rkindliya@gmail.com (R. Kumar), akverma@mining.iitkgp.ernet.in, abhiram.verma@gmail.com (A.K. Verma).

http://dx.doi.org/10.1016/j.ijrmms.2016.10.005
Received 4 March 2016; Received in revised form 9 June 2016; Accepted 31 October 2016
1365-1609/

kinematic constraints and the external force (such as induced forces, water pressures, earthquake forces etc.). Therefore, it is imperative to know the variation of shear strength in various directions. In other words, shear behavior of rock joint is anisotropic due to roughness variation in shearing direction.

To deal with anisotropic shear strength, many constitutive models developed in literature. Firstly, Huang and Doong [25] developed an empirical peak shear strength model in which *JRC* is determined by an equation proposed by Tse and Cruden.[26] Jing et al.[7] proposed a shear strength model which shows magnitude of asperity angle (i) is directional dependent and follows elliptical distribution. Kulatilake et al.[12] divided roughness of joint surface into non-stationary and stationary components. For any shearing direction, they suggested that average inclination angle and fractal dimension (D) are sufficient to capture non-stationary roughness and stationary roughness, respectively. Grasselli and Eager[19] reconstructed joint surface from three dimensional point clouds with triangulation algorithm method and found an empirical parameter θ_{max}/C as measure of roughness of joint surface in each direction. Where, θ_{max} and C is maximum asperity angle and roughness parameter characterizing the distribution of apparent dip angles over joint surface, respectively. Park et al.[27] proposed a constitutive model based on active roughness coefficient (C_r) which is derived from probability distribution of active micro-slope angle in shearing direction. Xia et al.[24] modified the Grasselli model stating that this model is not suitable for smooth rock joints. Till date, among all constitutive models, Barton model is widely used in practice due to its simplicity despite *JRC* is subjective in this model and unable to differentiate shear strength in forward and backward direction of shearing.

To quantify *JRC*, several methods like statistical [26,28–30], fractal [31–41], and tilt tests [42] are used in literatures. The visual comparison method found to be subjective and it is difficult to compare a 3D joint surface with ten standard profiles.[43] The statistical and fractal methods have many limitations such as they are based on analysis of single profile in shearing direction and do not account for three dimensional geometry of joint wall. Moreover, these methods are sensitive to sampling interval and do not capture anisotropy of roughness in backward and forward direction of shearing. Back calculated *JRC* is not useful because it requires prior knowledge of shear strength of joints.[19]

Further, due to roughness variation in shear direction, peak shear displacement (d_p) and shear stiffness (K_{ss}) of rock joints are also anisotropic. Barton and Choubey [5] suggested that peak shear displacement exists at 1% of joint length. Moreover, Barton and Bandis [44] and Asadollahi et al.,[22] proposed empirical models to predict peak shear displacement based on *JRC* of joint surface.

Existing constitutive models of shear strength extend our understanding towards the complex shear behavior of rock joints. From literature, it can be seen that most of the constitutive models are formulated using basic friction angle (ϕ_b), peak dilation angle (d_n), whereas contribution of shear component (S_n), is exceptional. Therefore, in this paper, a new shear strength criterion is proposed in which ϕ_b, d_n, and S_n are discretely estimated in shearing direction. Moreover, to predict peak shear displacement in shearing direction, existing models requires prior knowledge of *JRC*. So in this paper, an empirical model for peak shear displacement is also proposed which is independent of *JRC*.

2. Sample preparation

On natural joints, parametric studies are not possible due to damage of asperities after each direct shear test. To overcome this problem, joint replicas having similar roughness are prepared with model material. In this study, natural rock surfaces are collected and taken to the laboratory for investigation. On these surfaces, three different profiles of 90 mm diameter are visually selected (Fig. 1). In laboratory, mixture of room temperature vulcanizing (RTV) silicon rubber and catalyst in a ratio of 10:0.5 is poured on a 90 mm diameter circular zone of the natural joint surface. From this process, a silicon rubber mold with the natural joint roughness similar to upper half of rock is obtained. Subsequently, mold release agent is sprinkled on first silicon rubber mold and again mixture of RTV silicon rubber and catalyst are poured on it. From this step, second silicon rubber mold, again with natural joint roughness similar to lower half of rock results. Mold release agent is used to avoid sticking of the silicon rubber molds. By this process, it is possible to obtain multiple silicon rubber molds with same geometric features of natural joint surface.

Further, prepared silicon molds are placed in wooden molds of 90 mm diameter and 50 mm height. The model material (mixture of cement, sand, and water) in the ratio of 1:1.5:0.45 by weight is poured into the wooden molds and vibrated for one minute on a vibrating table. After 12 h, the casted joint replicas are taken out from wooden molds and cured in water for 28 days. Finally, joint replicas (JR1, JR2, and JR3) with three different roughness profiles are obtained (Fig. 2).

The uniaxial compressive strength (σ_c), Brazilian tensile strength (σ_t), Poisson's ratio (v), density (ρ), and Young modulus (E) of model material are determined as per ISRM standards. For uniaxial compressive strength and Brazilian tensile strength the L/D ratio is maintained as 2.5 and 0.5, respectively. Basic friction angle (ϕ_b) of joint replicas are determined by performing four direct shear tests on saw cut joint replicas under four normal stresses (0.25, 0.5, 1.0, and 1.5 MPa). The average values of these mechanical properties are reported in Table 1.

3. 3D surface measurement of joint replicas

A 3D non-contact type rock surface profiler is developed in the department of Mining Engineering, IIT Kharagpur. It is suitable for rock joint samples of size not exceeding 150 cm with least count of 0.5 mm in X,Y directions and 0.1 mm in Z direction. It consists of a laser distance sensor and two stepper motors. One steeper motor is used for movement of sample in "X" and "Y" direction while another stepper motor is used for movement of laser distance sensor (Fig. 3).

In this study, a joint replica is scanned at 30° interval in only six directions (0°, 30°, 60°, 90°, 120°, and 150°) because in backward and forward directions, surface measurement will be same. In each shearing direction, total 180 parallel lines were obtained with X, Y, and Z data. In each shearing direction, these data are used to recognize active and inactive asperities. The active asperities are those facing the shear direction and recognized by their positive slope ($dy/dx \geq 0$) in the shearing direction whereas inactive asperities are opposite to shearing direction and recognized by negative slope ($dy/dx \leq 0$).[27] Considering the concept of active and inactive asperities, in each shearing direction, maximum asperity angle (θ_{max}) and average asperity angle (θ_A) are calculated by processing of X, Y, and Z data in Matlab software. Further, these values are modified with mean plane of joint surface (π) and reported in Table 4. The description of mean plane is given in Section 6.2.

4. Direct shear test

In this study, total 144 direct shear tests are performed using 4 normal stresses (0.25, 0.5, 1.0 and 1.5 MPa) under CNL condition. The induced normal stress, σ_n/σ_c varies between 0.006 and 0.036, which are reasonable to simulate the shear behavior of rock joints at relatively shallow depth of about 75 m. For each normal stress, twelve direct shear tests are performed at 30° apart in anticlockwise direction from the assumed strike direction (0°) of the natural joint (Fig. 4a) and this 0° remains constant throughout the tests for particular profile.

The electro-mechanical direct shear apparatus (Fig. 4b) make HEICO, India is used for conducting all experiments. This apparatus consists of shear box of dimension($100\text{mm} \times 100\text{mm} \times 100\text{mm}$), loading