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A joint shear model incorporating small-scale and large-scale irregularities

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ABSTRACT

The strength and dilation of rock joints in the field cannot be evaluated solely on the basis of parameters scaled from laboratory data, but also requires assessment of large-scale irregularities not present in the laboratory sample. A constitutive model for rock joints has been developed that considers the dilation and strength along both small-scale joint roughness scaled from laboratory data, and large-scale waviness determined from geologic observations. The model's performance is illustrated by providing its correlation with experimental results taken from literature. The degradation in dilation and post-peak strength along small-scale irregularities is modeled using the plastic work done in shear, and the degradation along large-scale irregularities is modeled using a sinusoidal function. A dimensionless product of plastic work, rock strength, and wavelength of irregularities has been developed which fits the direct shear test results. An approach to scaling shear strength and shear displacement from laboratory to field-scale is also suggested.

1. Introduction

The behavior of rock joints dominates the behavior of rock masses by providing planes of weakness along which shear and dilation can occur. A number of experimental studies have been conducted to understand the behavior of rock joints and many joint constitutive models have been proposed to predict their mechanical behavior. Patton's model [1], one of the earliest and most fundamental models for peak shear strength, was developed from the basic mechanics of sliding up the asperity with inclination angle or shearing through the asperity depending on the normal stress level. Later, Ladanyi and Archambault [2] proposed a semi empirical model which featured the curved failure envelopes. Barton [3] developed a useful empirical model by introducing a morphological parameter known as the joint roughness coefficient (JRC) and using the concept of roughness mobilization. A significant advanced theoretical model was developed by Plesha [4], in which asperity degradation is a function of the plastic work during shear. More recently, approaches such as fractal [5] and geostatistical analysis [6,7] have been proposed to evaluate the mechanical behavior of rock joints under shear.

A large body of literature (e.g., [1,8–17]) indicates that strength and shear behavior of rock joints vary both qualitatively and quantitatively as a result of a change in sample- or in situ block-size. Ignoring scale effect may lead to overestimation or under-estimation of field shear strength of joints if the peak strength obtained from laboratory joint shear test is used. Shear behavior of rock joints in the field should be evaluated by considering the dilation and strength along both small-scale joint roughness scaled from laboratory data, and large-scale waviness determined from geologic observations. However, most rock joint constitutive models proposed in literature have been developed on the basis of data obtained from laboratory tests on natural or model rock joints. Thus, they do not fully represent the behavior of rock joints in the field. This paper describes a rock joint constitutive model which can generate shear stress–displacement–dilation curves for both small-scale and large-scale joints. The model is incorporated in 3DEC [18] using the built-in programming language, FISH and is correlated with experimental results of direct shear tests taken from literature.

2. Description of a constitutive model for small-scale joints

2.1. Mobilized shear strength

The shear stress–displacement–dilation curves generated by the proposed joint model can be characteristically divided into five stages: (1) elastic region, (2) pre-peak softening, (3) mobilized

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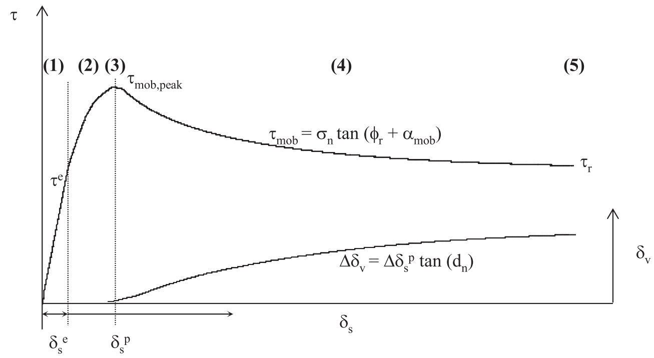

Fig. 1. Schematic shear stress–displacement–dilation curves from the joint model. Curves of simulation results using the same material properties as used in [19] in the direct shear tests as illustrated in Fig. 6. Superscript e and p indicate elastic and plastic, respectively.

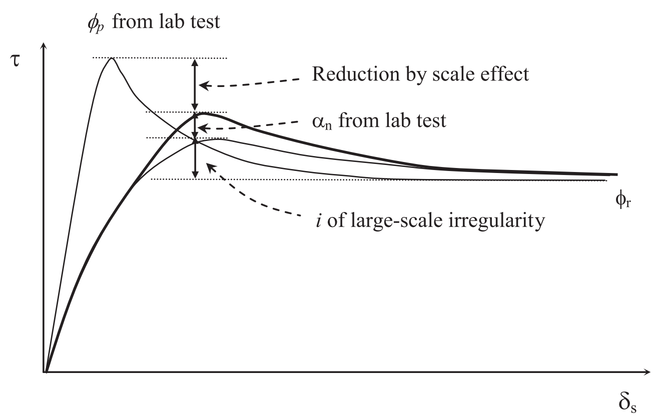

Fig. 2. Schematic shear stress–displacement curves. Bold one represents shear stress–displacement curve for joint in the field.

peak strength, (4) post-peak softening, and (5) residual strength. Schematic curves in Fig. 1 shows there are five stages in the shear stress–displacement–dilation curves. It is frequently observed from experimental results of direct shear tests in the literature that the shear stress–displacement curves show almost linear elastic behavior to a stress approximately equivalent to the residual strength of the joint. The proposed joint model accounts for these observations. Therefore, during elastic region, the shear stress is mobilized as a function of joint shear stiffness (K_s) and elastic shear displacement (δ_s^e). The shear stress increment ($\Delta\tau^e$) is calculated as

$$\Delta\tau^e = K_s \Delta\delta_s^e \quad (1)$$

After the elastic region, the joint starts to slide and dilation takes place, which means the plastic shear displacement occurs from this point on. The degradation in asperity and also dilation is modeled as a function of the plastic work done in shear. This is discussed in detail in the following sections.

Most rock joints show that peak shear strength is mobilized at very small deformation. A peak shear displacement is sometimes considered to be a material constant, not affected significantly by changes of normal stresses, which is supported by experimental results such as Leichnitz [20] and Herdocia [21]. On the other hand, as shown in some direct shear test results, such as Jaeger [22], Schneider [23], and Flamand et al. [19], this parameter is not always constant for a given joint. On the basis of results from shear tests on model tension fractures, Barton and Choubey [8] suggested that a peak shear displacement is dependent on sample scale and occurs after a shear displacement equal to 1% of the joint sample length up to some limiting size. Later, Barton [3] assumed that elastic shear displacement (δ_s^e) is 0.3 times peak shear displacement (δ_{peak}). By performing direct shear tests on different sized replicas casts from various natural joint surfaces, Bandis et al. [10] concluded that for practical purposes a peak shear displacement (δ_{peak}) can be taken as approximately equal to 1% of the joint length for a large range of block sizes and types of roughness. Since no general relationship for peak shear displacement (δ_{peak}) is yet available, as the preceding discussion indicates, a peak shear displacement (δ_{peak}) is treated as an input parameter in the proposed joint model, as 'n' times of elastic shear displacement (δ_s^e), where δ_s^e is determined by the combination of stress level and joint shear stiffness (K_s). For the initial analysis or with limited test data available, the value of 'n' can be selected to be 3, based on direct shear test results frequently observed in the literature [3,24].

After peak shear strength, a mobilized shear stress is gradually decreased until it reaches a residual value. The mobilized shear stress during plastic region is calculated as

$$\tau_{mob} = \sigma_n \tan(\phi_r + \alpha_{mob}) \quad (2)$$

where σ_n is a normal stress, τ_{mob} is a mobilized shear stress, ϕ_r is a residual friction angle, and α_{mob} is a mobilized asperity angle that degrades as plastic work increases.

2.2. Asperity degradation

The model proposed here simulates the progressive degradation of a joint asperity under shear. It is modeled by assuming that degradation is a function of the plastic work, W_p and its relationship is given by Eq. (3), which was suggested by Plesha [4].

$$\alpha_{mob} = \alpha_0 \exp(-cW_p) \quad (3)$$

where α_0 is an initial asperity angle, c is a asperity degradation constant, and W_p is expressed as $W_p = \sum \Delta\delta_s^p \tau$.

Although Eq. (3) possessed good qualitative and quantitative agreements with experimental observations, it is difficult to relate the asperity degradation constant, c to other properties of rock joints [4]. From the cyclic shear test results on some thirty real rock granite and limestone joints, Hutson and Dowding [25] proposed an advanced relationship for the asperity degradation constant, c given by

$$c = -0.141\ \alpha_0 N / \sigma_c [\mathrm{cm}^2/J] \quad (4)$$

where α_0 is an initial asperity angle, N is a normal stress, and σ_c is an unconfined compressive strength of rock. This relation indicates that asperity degradation is a function of material strength as well as stress level, which has been mentioned by many researchers [2,8,13].

While the asperity degradation constant, c, in Eq. (4) describes general behavior of a rock joint, it still has some questions to be considered. First, it is not a dimensionless relation. The constant c has a unit of [cm^2/J] with a dimensional constant of 0.141. Thus Eq. (4) is valid in only one consistent system of units. Second, Eq. (4) is valid for limited range of scale of irregularities. The Hutson–Dowding model was determined from tests on rock joints having wavelengths of 3.2 and 5.0 cm, and is not expected to apply to the typical laboratory samples. Hutson mentioned in his thesis [26] that large laboratory models, such as the largest used by Bandis [9], should produce similar results. Finally, the effect of the applied normal stress in Eq. (4) is duplicated since this constant is multiplied by the plastic work, W_p as shown in Eq. (3).

In order for the asperity degradation constant to apply to any scaled laboratory samples, Eq. (4) appears to be modified. A geometric parameter, that is, a wavelength is introduced and thus the wavelength, λ with the initial asperity angle, α_0 describes the joint shape of typical laboratory samples. By removing the normal