International Journal of Rock Mechanics & Mining Sciences 70 (2014) 252–263

Contents lists available at ScienceDirect

International Journal of
Rock Mechanics & Mining Sciences

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

Model for the shear behavior of rock joints under CNL and CNS conditions

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ARTICLE INFO

Article history:
Received 2 January 2013
Received in revised form
28 April 2014
Accepted 7 May 2014
Available online 5 June 2014

Keywords:
Rock joint
Surface resistance index
Normal displacement behavior
Empirical shear behavior model
Constant normal load
Constant normal stiffness

ABSTRACT

Empirical models for the shear behavior of rock joints are suggested from the results of direct shear tests using a servo-controlled shear testing machine. Cement mortar replicas of rock joints are tested while varying the normal stiffness, initial normal stress, joint roughness coefficient and joint wall compressive strength. The test results are analyzed to investigate the effects of loading conditions and material properties on the surface resistance index and normal displacement behavior of the joints. In this study, the ratio of the shear stress to normal stress was defined as the surface resistance index because it shows friction-related characteristics between joint surfaces. Empirical models of the surface resistance index and normal displacement behavior are suggested. In the empirical models, dimensionless terms are adopted to avoid the scale effect and thus enhance the applicability of the suggested models. The suggested models can be applied to predict the shear behavior of rock joints, including pre-peak and post-peak shear stress levels, regardless of the loading condition. To verify the suggested models, additional shear tests of the rock joints and cement mortar replica specimens are carried out, and the performance of the models is compared with those of other models such as Barton's empirical model. The suggested models are applied to the test results of previous studies. The predictions of the suggested models corresponded well with the experimental results for the overall stress levels.

1. Introduction

The shear behavior of rock joints is an important issue in the design of rock structures in jointed rock masses. To investigate the overall shear behavior of rock blocks around a rock structure, rock joints should be tested under constant normal load (CNL) or constant normal stiffness (CNS) conditions. The CNL condition is applicable to rock slopes, whereas the CNS condition is based on underground rock masses where the shear behavior of rock joints depends on the stiffness of the rock mass. In terms of field application, the CNS condition is more general because the CNL condition can only be applied to limited situations, as it does not consider alteration to the normal stress by dilation. Therefore, these conditions must be selectively applied by considering the field conditions [1,2].

Direct shear tests under the CNL condition have been more frequently adopted in studies on the shear behavior of rock joints primarily because of the simplicity and ease of experimentation. Since Patton [3] suggested a bilinear model for idealized saw-tooth joints, many studies have been performed under the CNL condition [4–7]. On the other hand, there have been few studies conducted under the CNS condition because of the experimental difficulties. In early CNS tests, springs were inserted between a rock specimen and normal loading plates to keep the normal stiffness constant. Even with some successful applications of the springs to CNS tests, the drawbacks of this early device are obvious: changing springs because of changes in rock mass stiffness is a cumbersome and inconvenient job, and joint surfaces are easily damaged under high normal stiffness conditions [8,9].

Since then, shear tests under the CNS condition have increased with the development of servo-controlled direct shear testing machines. Rim [10] investigated the correlation between the inclination angle, initial normal stress, normal stiffness and shear parameters under the CNS condition, and Son [11,12] employed rock joints and their replicas made of cement mortar in CNS tests to investigate the effect of boundary conditions and the roughness of joints on shear behavior.

Moreover, many researchers have performed studies related to models of shear behavior. Barton et al. [13] suggested the Barton–Bandis model based on various shear test results, and this model is

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http://dx.doi.org/10.1016/j.ijrmms.2014.05.005
1365-1609/

widely used even today. Saeb and Amadei [14] presented an explanatory diagram of shear behavior of rock joints under the CNL condition, and Haque and Ranjith [15] compared the results of shear tests with numerical analysis using a continuous yielding model in Universal Distinct Element Code (UDEC). Seidel et al. [16,17] suggested a theoretical model for shear behavior under the CNS condition, but the decision processes of the suggested parameters are very complex. These studies suggested shear behavior models for joints mainly based on theoretical analysis; only a few studies use an empirical approach to shear behavior under the CNS condition. It cannot be too emphasized to verify the theoretical models by using physical measurements considering the applicability of the models. The empirical verification, however, often suffers from insufficient data or difficulty in obtaining the parameters of the suggested models. The empirical approach, on the other hand, usually has a weak theoretical basis and can lead to more conservative design solutions [18]. Barton–Bandis model is suggested by empirical approach using many test results, but its applicability is limited in specific situations such as soft rocks or joints with low roughness [19,20]. Moreover, according to Son [11,12], the model tends to overestimate the shear stress under low normal stiffness conditions. Although Son suggested a modified Barton–Bandis model for low normal stiffness conditions, it has insufficient applicability because of its indefinite range of application, such as for normal stiffness. Recently, Park et al. [21] suggested a new roughness coefficient and an empirical shear behavior model based on this coefficient that considers the contact area in joint planes. Analysis method used in Park et al.'s study is distinguished from method used in this study, even though specimens with same property are used. This study focused on easy application by adopting generally used roughness concept. On the other hand, Park et al. mainly studied about developing new roughness coefficient and thus the constitutive model is completely based on the new coefficient, which needs a highly complicated and new approach to estimate the shear behavior of rock joint. Also, the constitutive model was constructed by using regression analysis and not compared with any other test results in other study. It was only compared with test results performed in the study.

In this study, an empirical model for the shear behavior was suggested based on direct shear tests using a servo-controlled shear testing machine. Building the model requires multiple tests under various conditions with specimens having the same roughness. It is, however, impossible to obtain such rock specimens from nature. That is why cement mortar has been adopted to make replicas for the model building. The cement mortar replicas of rock joints were tested while varying the normal stiffness (K_n), initial normal stress (σ_0), joint roughness coefficient (JRC), and joint wall compressive strength (JCS). Test results were analyzed to investigate the effects of loading conditions and rock properties on the surface resistance index and normal displacement of the joints. Empirical models of the surface resistance index and normal displacement were suggested based on the results. To verify the applicability of the suggested model, which is based on the test results of replica specimens, to real rock specimens, the joint shear test results of Hwangdeung granite and Onyang gneiss were compared with the test results predicted by the suggested model. The suggested model was also compared with the existing results of other researchers for verification.

2. Experiment

2.1. Test equipment

In this study, direct shear tests were performed using a direct shear testing machine where the displacement and stress were servo-controlled in both the normal and shear directions. The displacement and stress were measured with linear variable differential transformers (LVDTs) and load cells. The normal and shear load cells were of a strain gauge type where the load is measured by detecting changes in the resistance of the strain gauge built in the load cells. These load cells had a safe loading range of 30 ton, a load capacity of 60 ton, and less than 0.05% nonlinearity and elastic hysteresis.

A three-dimensional laser profiling machine was used to profile the joint surface with a spacing of 0.5 mm. This machine consists of a laser displacement meter, positioning system, and control computer. The laser displacement meter measured the displacement of each point using the angle of the reflected laser, which had a wavelength of 600 nm and maximum capacity of 1.9 mW. This machine had a minimum measurement interval of 0.5 μm on the X and Y axes, and a measurement range of ± 9 mm on the Z axis. The performance of both machines was verified in previous studies [10,22,23].

2.2. Specimens

In this study, specimens were named after their rock type and joint roughness; e.g., GR-14.04 indicates a Hwangdeung granite specimen with a *JRC* of 14.04. GN represents Onyang gneiss, and R represents a replica; e.g., R-12.72 indicates that the specimen is a replica with a JRC of 12.72. The JRC of specimens was calculated using the regression equation between the JRC and Z_2, as suggested by Tsu and Cruden [24]

$$\mathrm{JRC} = 32.2 + 32.47 \log Z_2 \tag{1}$$

where

$$Z_2 = \left[\frac{1}{L}\int_{x=0}^{L}\left(\frac{dy}{dx}\right)^2 dx\right]^{1/2} = \left[\frac{1}{L}\sum_{i=1}^{N-1}\frac{(y_{i+1}-y_i)^2}{x_{i+1}-x_i}\right]^{1/2} \tag{2}$$

This process was performed at measurement intervals of 0.5 mm. Table 2 lists all of the sample types used in this study and their roughness.

Hwangdeung granite and Onyang gneiss blocks were split to produce tensile fracture at the center, and their replicas for the direct shear tests were made by using non-contraction and high-strength grout consisting of soil, cement, anhydrite gypsum, and additives mixed at a weight ratio of 50:32:15:3. Replicas were cured for 4 days at room temperature and prepared with seven types of roughness: JRC values of 4.63, 11.30, 12.33, 12.72, 12.84, 13.71, and 13.86. Table 1 summarizes the properties of the specimens used in this study. The JCS was set to be the same as the uniaxial compressive strength (UCS) because all tensile fractures were fresh

Table 1
Properties of the specimens used in this study.

Properties	Hwangdeung granite	Onyang gneiss	Mortar (14%)	Mortar (15%)	Mortar (17%)	Mortar (19%)
Uniaxial compressive strength (MPa)	184.08	170.11	83.91	74.03	65.33	54.19
Young's modulus (Gpa)	55.43	54.37	–	30.31	–	–
Poisson's ratio	0.28	0.28	–	0.27	–	–
Brazilian tensile strength (Mpa)	8.38	12.63	5.41	5.04	3.92	3.39
Basic friction angle (°)	34.29	34.22	31.03	34.57	32.21	31.45