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Development of a new model to predict the shear strength of unfilled natural joints

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Abstract

The strength of rock mass is significantly controlled by the presence of discontinuities and instabilities at shallow depth may occur by sliding along rock joints. This phenomenon is more important at shallow depths. The aim of this study is to investigate the shear behavior of natural, unfilled rock joints that were obtained from core drilling. For this purpose, direct shear tests were conducted on 37 natural rock joints in constant normal loading condition. All the specimens were obtained from core drilling of a geotechnical project. The morphological characteristics of joints were captured by photogrammetry before the test. Based on Barton and other classical theories, laboratory test data, and using the quantitative three-dimensional morphological parameters, a new peak shear strength criterion has been proposed for natural, unfilled rock joints. The predicted peak strengths using the new model match well with the observed values. In addition, a comparison of the new criterion with previous 3D models shows that compared to other criteria, the predicted value based on the new criterion is more consistent with the test data. It was observed that the estimation accuracy of the new criterion is appropriate for natural, unfilled joints.

I. INTRODUCTION

The rock mass is an assemblage of joints and rock blocks. The presence of joints controls strength and deformation properties of natural and engineering structures, such as rock slopes and underground excavations. Natural joints inherently possess asperities that radically alter the mechanical behavior of themselves. Mechanical properties of rock joints govern the strength and deformational behavior of a rock mass. The responses of a rock joint to shear and normal loadings highly depend on its surface properties, block size and matching state. In rock mass stability analysis, one of the most crucial factors to be considered is the joint shear strength. The existence of rock joints significantly decreases the strength and considerably influences the instability of rock masses along which sliding can easily occur. Understanding the shear behavior and predicting the shear resistance parameters of rock joints is a key step in the design of shallow depth geotechnical projects [1, 2]. Recently researchers began to study three-dimensional characterization of joint surfaces. They tried to quantify the joint surface and explore some correlation to the shear resistance. Several Peak Shear Strength (PSS) criteria based on advanced techniques (including a method based on the fractal, laser scanner, or photogrammetry, etc.) have been developed to evaluate the roughness, we can highlight those of [3-7].

Grasselli et al. (2002) undertook an extensive series of experiments and showed that the geometry of the joint surface influences the size and distribution of the contact area during the shearing process. They argued that only the asperities which are faced in the shearing direction and are steeper than a threshold inclination angle are involved in the shearing process. They digitized the surface of several fractures and divided the surface to finite triangles, as shown in Figure 5. They proposed the following empirical equation for estimation of the peak shear strength [8]:

$$\tau_\rho = \sigma_n \tan\left[\emptyset_b + \left(\frac{\theta^*_{max}}{c}\right)^{\frac{1}{18\cos\alpha}}\right]\cdot\left(1+e^{-\frac{\theta^*_{max}}{9A0C}\cdot\frac{\sigma n}{\sigma_t}}\right) \qquad (1)$$

where σ_t is the tensile strength of intact rock, A_0 is the maximum possible contact area, θ^*_{max} is the maximum apparent dip angle in the shearing direction and C is roughness parameter and α is the angle between the schistosity plane and the plane normal to the joint (if the rock exhibit no schistosity, α is equal to zero).

Most of the previous researches have been done on artificial, replica and produced mated joints with regular shapes (rectangular or square) in the laboratory. Since geotechnical engineering studies are performed by core drilling, and the direct shear tests should be performed on core sample joints, this issue has received less attention due to the lack of samples and difficulty to obtain data.

This study investigates the shear behavior of natural joints obtained from core drilling without filling. Based on the Barton and other classical theories, three-dimensional morphological characteristics, and test results a new predictive criterion was developed. The new proposed model was developed based on experimental data from direct shear tests on 37 sets of natural joints under Constant Normal Load (CNL) conditions. To validate this, the accuracy estimation of new model was compared with the existing classical criteria and it was observed that the new criterion has a good accuracy to predict the PSS of natural, unfilled rock joints.

II. LABORATORY TESTS PROCEDURE

A. *Rock joint specimens*

To investigate the shear behavior of natural, unfilled joints, we have performed direct shear tests on 37 natural sample joints under CNL conditions. The 37 joints were open, clean and non-weathered without cohesive infill, and no indication of prior shearing. The samples were in the form of cores and were obtained from different burial depths during geotechnical drilling. The test core samples were collected from the slope of an Iron open-pit mine in Southeast of Iran. The site is located at the south-eastern boundary of the Sanandaj-Sirjan zone basement with mostly medium to high grade metamorphic rocks of Neoproterozoic age (e.g., Amphibolite, Gneiss, Mica-Schist, and Marble) [9]. The rock mass was significantly fractured.

B. *Sample Preparation and determination of geo-mechanical properties*

To prepare the laboratory experiments, the specimens were set inside the molds and then were encapsulated in plaster to ensure a solid fit. To perform direct shear test, each half of the specimen was secured in the specimen holders. The dimensions of shear boxes were 140 × 140 × 10 mm^3. To perform the direct shear test on natural joints, we used the automatic direct shear test device of the Kashigar geo-mechanics research center (KGMC) (Fig. 1).

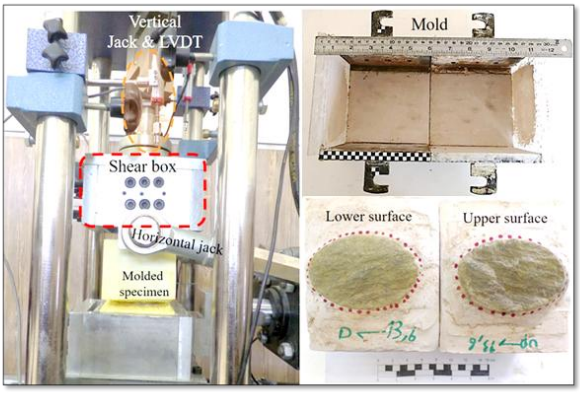

Fig. 1. View of the mold, encapsulated specimens and testing machine.

The normal and shear capacity of the vertical and horizontal hydraulic jacks were 10 and 15 tons, respectively and the shear box size is 140 by 140 mm. The shear and normal displacements were also measured by two LVDTs. The normal loads of 1.0 to 6.2 MPa were applied according to the burial depth samples. Under different normal stresses based on burial depth of samples, direct shear test results of 37 natural rock joints are shown in Fig. 2.

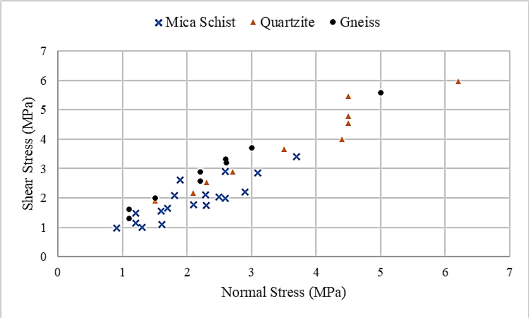

Fig. 2. Test results of the peak shear strength of natural rock joints for different lithology

It is found that the PSS of the samples are increasing with increasing normal stress. It is more obvious for Quartzite joints due to the high strength compared to other samples. To characterize the geo-mechanical properties of the three rock types (Mica-Schist, Gneiss, Quartzite), the uniaxial compressive strength (UCS) and Brazilian (indirect tensile) tests were used to determine the compressive and tensile strength of the specimens. The UCS and Brazilian tests were conducted on core samples with 63 mm diameter and L/D ratio of 2.2 to 2.5, and 0.5 respectively (Fig. 3). The saw cut samples were also used to measure the basic friction angle of planar joints. To minimize the deviation of the results, each test was repeated six times and the average values were considered.

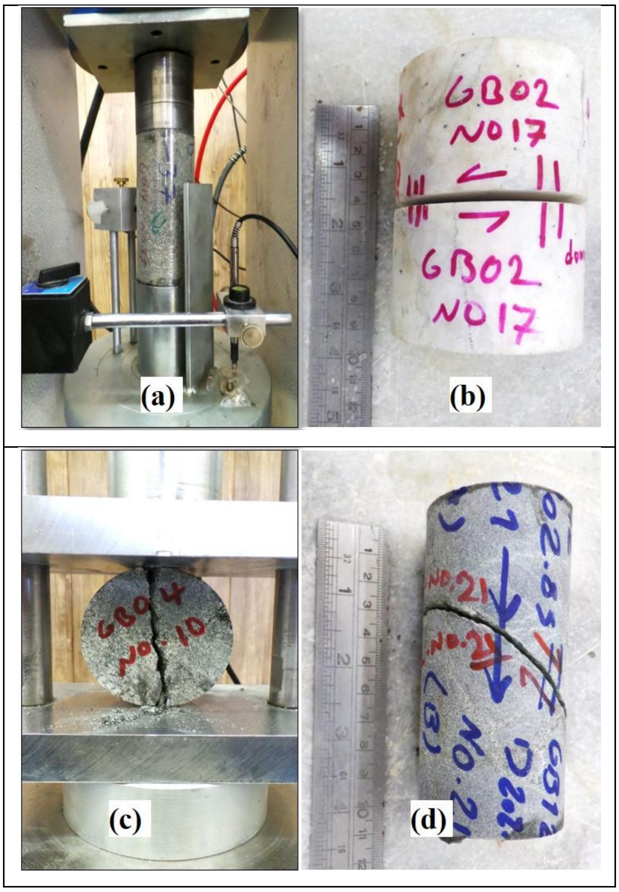

Fig. 3. a) UCS test machine and specimen, b) saw cut specimen, c) Brazilian test machine, and d) natural rock joint specimen.

C. *Surface Morphology Data*

The close-range photogrammetry (CRP) was used to survey the 3D surface morphology of joint samples of this study. Parameter settings and the photogrammetry were based on [10]. We used a single-lens reflex digital camera (Canon EOS 1300D), which has a high-resolution CCD sensor (5184×3456 = 18 megapixels), to capture images of the surface natural joints. The surface morphology of the rock joints was taken by CRP before the test and the surface morphological parameters such as θ*max and C were calculated. The surface of all samples was cleaned of dust or molding plaster before the photogrammetry and testing. The encapsulated specimens were placed into the referenced mold. Figure 4. shows the process of the photogrammetry of natural sample joints used in this study.

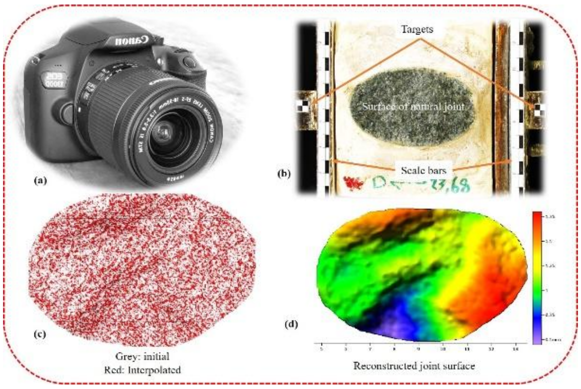

Fig. 4. The Procedure of the Photogrammetry: a) camera used to photogrammetry; b) natural joint surface; c) interpolation of cloud points; d) reconstructed joint surface

Figure 5. describes the methodology used for this study and shows the procedure of digitizing joint surfaces using photogrammetry and research process on natural joints.

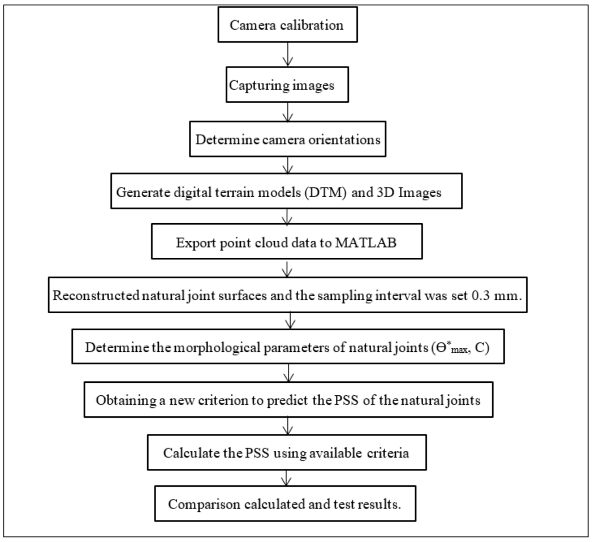

Fig. 5. The procedure of research on natural joints in this study.

According to previous recommendation by Grasselli and Egger [11], Tatone and Grasselli [4], Xia et al. [7], and Ríos-Bayona et al. , the surfaces of natural rock joints were reconstructed with a resolution of 0.3 by 0.3 mm.

III. A NEW PEAK SHEAR STRENGTH CRITERION FOR NATURAL ROCK JOINTS

Barton (1973) pointed out that friction angle (ϕ)j along joint surface is the sum of basic angle of friction (ϕb), peak dilation angle (dn), and the shear component (Sn) that represents failure of intact material (asperities) as given below:

$$\tau_{peak} = \sigma_n.\tan(\phi)_j = \sigma_n.\tan(\phi_b + d_n + S_n) = \sigma_n.\tan(\phi_b + \alpha_e) \qquad (2)$$

Where τ_{peak} is the peak shear strength of the joint, σ_n is the applied normal stress, and α_e is the joint effective asperity angle which is the contribution of asperities to the shear resistance.

Joint asperity angle depends on the applied normal load and surface morphology of the rock joint[12]. For a certain joint, the effective asperity angle varies with the applied normal load. Jing (1990) proposed an empirical relation between the normal stress and the effective asperity angle through a data fitting process as below:

$$\alpha_e = \alpha_e^0 (1 - {}^{\sigma_n}\!/\!_{\sigma_c})^b \tag{3}$$

Where α_e^0 is the initial effective asperity angle, and b is a material parameter.

As mentioned above, [5]Yang et. Al. (2016) proposed to define the variation of the effective asperity angle as a function of the described 3D morphology parameters:

$$\alpha_e = \frac{\theta^*_{\max}}{C^{0.45}} e^{-\frac{\sigma_n}{JCS} C^{0.75}} \tag{4}$$

Where α_e is the effective asperity angle, JCS is the Joint Compressive Strength and is the UCS of the intact rock obtained from a standard uniaxial test, $\theta^*_{\max}$ is the maximum apparent dip angle, and C is the roughness parameter. According to Eq. 3, the Yang's criterion cannot reflect the saw-tooth joints because of C is the denominator.

A. New peak shear strength criterion

As mentioned above, the effective asperity angle consists of two components, initial effective asperity angle and a reduction coefficient with different applied normal load. The strength of asperities plays a significant role in PSS. Some researchers [11, 13, 14] pointed out that during the shear test, the failure mode of the joint asperity was mainly tensile failure rather than compressive failure. Yang et. al. (2016) stated that compressive strength would be a key factor influencing the PSS. In this study, the effective asperity angle was obtained by back analysis from the laboratory of test results. It is found that the effective asperity angle shows good correlation with the 3D morphology parameters. Therefore, the following rational asperity angle function is proposed to predict the effective asperity angle based on three-dimensional topography parameters.

$$\alpha_e = \frac{\theta^*_{\max}}{\sqrt{1+C}} . (1 - {}^{\sigma_n}\!/\!_{\sigma_c})^b \tag{5}$$

The first part $\theta^*_{\max}/(1+C)^{0.5}$ is the initial effective asperity angle which is only related to the joint morphology. The second part $(1-\sigma_n/\sigma_c)^b$ is a reduction coefficient for the initial effective asperity angle with different normal loads. Based on the regression analysis, parameter b value is 1.8.

According to the mentioned above, based on the morphological data and the laboratory shear test results, also, considering the effective asperity angle changing trend, the following model for the PSS for natural, unfilled rock joints is proposed:

$$\tau_{p-nat} = \sigma_n . \tan(\varphi_b + \alpha_e) = \sigma_n . \tan\left(\varphi_b + \frac{\theta^*_{\max}}{\sqrt{1+C}} . \left(1 - \frac{\sigma_n}{\sigma_c}\right)^{1.8}\right) \tag{6}$$

where $\tau_{p\text{-nat}}$ is the PSS of the natural joint, α_e is the effective asperity angle, σ_n is the applied normal stress, ϕ_b is the basic friction angle, $\theta^*_{\max}$ is the maximum apparent dip angle with respect to the shear direction, C is the roughness parameter, σ_c is the uniaxial compressive strength. Compared the Yang's criterion, this model can be also used for saw-tooth joints.

IV. COMPARISON WITH EXISTING CRITERIA

To verify the global suitability of the new criterion, it is necessary to compare the accuracy of the new criterion to measure the PSS. For this purpose, the test data of Grasselli, Yang, and Tang are used to compare the prediction accuracy of Grasselli's criterion, Yang's criterion, Xia's criterion and the new criterion. The results estimated by the mentioned criteria and the PSS test are shown in Fig. 6. The measured PSS and curves calculated by the new model for three groups data of this study are also shown in Fig. 7.

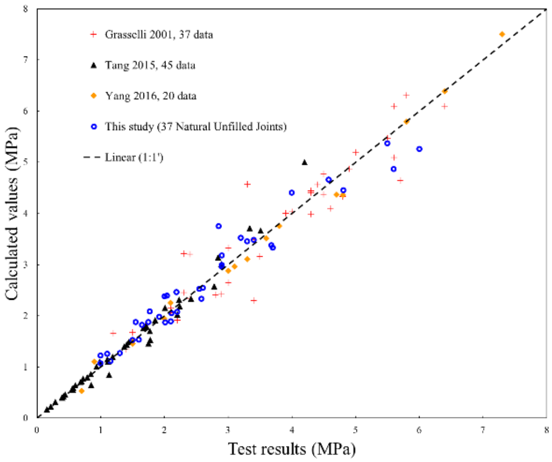

Fig. 6. Comparison between the tested and the estimated PSS in different models

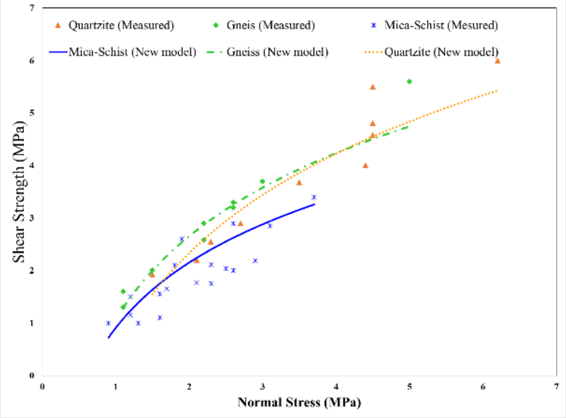

Fig. 7. The measured PSS and curves calculated by the new model for three Groups

It can be observed that the calculated values generally agree well with the measured data. For the Quartzite and Gneiss rock types, the calculated value is smaller than the measured value. The new criterion tends to underestimate the shear strength at high normal stress conditions, in contrast the Mica-Schists with lower strength that overestimate the PSS.

V. DISCUSSION

To examine the accuracy of some new and reliable classic models and the proposed model, the average relative error (δ_{avg}) is used to represent the average value of the error, the standard deviation (μ) of the relative error is used to represent the degree of error deviation. The fitting quality of the predicted value and the measured value is determined by the average value of the error and the degree of deviation of the error as following:

$$\delta_{avg} = \frac{1}{n}\sum_{i=1}^{n}\left|\frac{\tau_{mea}-\tau_{cal}}{\tau_{mea}}\right| \times 100\% \qquad (7)$$

$$\mu = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\delta_i - \delta_{avg}\right)^2} \qquad (8)$$

Where τ_{mea} is the measured value of the PSS, τ_{cal} is the predicted value of the PSS, n is the total number of tests, δ_i is the relative error of the i_{th} group. The average estimation error (δ_{avg}) and the standard deviation (μ) of the relative error of data from previous studies for mentioned criteria and the new model are presented in table 1.

TABLE I. COMPARATIVE ANALYSIS OF THE PREDICTED VALUE OF PSS FOR DATA PREVIOUS STUDIES

Criterion	Grasselli's, 37 data		Yang's, 20 data		Tang's, 45 data		all 102 sets of data	
	δ_{avg} (%)	μ (%)	δ_{avg} (%)	μ (%)	δ_{avg} (%)	μ (%)	δ_{avg}(%)	μ(%)
Grasselli et al, 2002	10.9	11.1	17.1	8.0	13.3	11.2	13.1	10.6
Xia et al, 2014	17.8	20.5	16.9	13.8	6.4	5.7	12.6	14.3
Yang et al, 2016	20.2	27.7	5.5	6.5	8.8	9.8	12.3	18.2
This study	19.8	20.0	8.9	8.0	9.1	11.0	12.9	14.5

For Grasselli's data, the δ_{avg} of the new model is less than Yang's model and higher than other criteria. For Yang's data, the δ_{avg} of the new model is higher than Yang's model and less than other criteria. For Tang's data, the δ_{avg} of the new model is higher than Xia's model, less than Grasselli's model, and slightly higher than Yang's model. For all 102 sets data, the δ_{avg} of the all criteria are very close to each other and the □□of the Grasselli's model is the smallest. In contrast, the predicted value of new criterion is more consistent with the test data than Yang's model.

The following is a comparative analysis of the estimated PSS of data of this study. The average estimation error (δ_{avg}) and the standard deviation (μ) of the relative error of data from this study for mentioned criteria and the new criterion are presented in table 2.

TABLE II. COMPARATIVE ANALYSIS OF THE PREDICTED VALUE OF PSS FOR DATA PREVIOUS AND THIS STUDY

Criterion	This study, 37 data		Previous and this study, 139 data	
	δ_{avg} (%)	μ (%)	δ_{avg}(%)	μ(%)
Grasselli et al, 2002	18.7	13.5	14.6	11.5
Xia et al, 2014	15.0	9.0	13.2	13.1
Yang et al, 2016	11.3	9.2	12.0	16.3
This study	9.0	6.9	11.9	12.9

For the data of this study (natural joints, 37 data), the δ_{avg} and the μ of the new model are the smallest. For all data sets of previous studies (102 data) and the current study (37 data) test, the δ_{avg} of the new criterion is the smallest and the Grasselli's criterion is the highest. In contrast, the predicted value of Grasselli's model, and new model is more consistent with the test data than other models. It means, the new criterion can predict the PSS of rock joint with acceptable accuracy, especially for natural rock joints.

A. Limitations and further work

The new model has some shortcomings that should also be discussed. First, this model contains the fitting parameter. The effective asperity angle of the new model is improved based on Grasselli's surface morphology parameters while obtaining these parameters is complex. The new proposed criterion is based on the test results of cored sample joints without filling and further research and improvement are needed to verify the criterion proposed in this study.

VI. CONCLUSIONS

The aim of this study was to provide a practical CNL model for estimating the shear strength of natural, unfilled rock joints using experimental data obtained from core drilling. For this purpose, laboratory analysis was conducted to study the shear behavior of natural unfilled joints with three different rock types in various burial depth. The new proposed model is a modified version of Barton's model (1973) that represents the basic friction angle and the effective asperity angle. The geo-mechanical properties of intact rock such as basic friction angle and compressive strength were obtained by direct shear test on saw-cut samples and uniaxial compressive strength test, respectively. The 3D morphology parameters (C, $\theta^*_{max)}$ were captured by photogrammetry, before test. By combining surface joint morphology and a coefficient with different normal loads $(1-\sigma_{n/}\sigma_c)^{\ b}$, a new practical predictive model was proposed. For what concerns the prediction accuracy, a comparison between the new model and the available classical models was conducted. For this purpose, the average relative error (δ_{avg}), and the standard deviation (μ) of the relative error were used. Based on the analysis, the new model has the smallest δ_{avg} and also the smallest μ for the data of this study. For all 102 sets data of previous studies, the δ_{avg} of the criteria is very close to each other. In contrast, the predicted value of Grasselli's model is more consistent with the test data than other models. For all 139 sets data (this study, 37 data and previous studies, 102 data), the new model has the smallest δ_{avg}. Experimental validation of the model showed an acceptable confidence level and thus it can be used in similar geotechnical projects where natural joints are obtained from core drilling with different morphological characteristics and irregular shapes under the conditions tested in this study.

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توسعه یک مدل جدید برای پیش‌بینی مقاومت برشی درزه‌های طبیعی بدون پرشوندگی

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چکیده

مقاومت توده سنگ به طور قابل توجهی توسط ناپیوستگی‌ها کنترل می‌شود و ناپایداری‌ها در عمق کم ممکن است با لغزش در امتداد درزه‌های سنگی رخ دهد. این پدیده در اعماق کم با اهمیت‌تر است. هدف از این مطالعه بررسی رفتار برشی درزه‌های سنگی طبیعی و بدون پر شوندگی است که از حفاری مغزه‌گیری به دست آمده‌اند. برای این منظور، آزمایش‌های برش مستقیم بر روی ۳۷ درزه سنگی طبیعی در شرایط بارگذاری نرمال ثابت انجام شد. تمامی نمونه‌ها از حفاری مغزه‌گیری در یک پروژه ژئوتکنیکی به دست آمدند. ویژگی‌های مورفولوژیکی درزه‌ها قبل از آزمایش با فتوگرامتری ثبت شدند. بر اساس نظریه‌های بارتن و سایر نظریه‌های کلاسیک، داده‌های آزمونهای آزمایشگاهی و با استفاده از پارامترهای مورفولوژیکی کمی سه‌بعدی، یک معیار مقاومت برشی اوج جدید برای درزه‌های سنگی طبیعی و بدون پرشدگی پیشنهاد شده است. مقاومت‌های اوج پیش‌بینی‌شده با استفاده از مدل جدید به خوبی با مقادیر مشاهده‌شده مطابقت دارند. علاوه بر این، مقایسه معیار جدید با مدل‌های سه‌بعدی قبلی نشان می‌دهد که در مقایسه با سایر معیارها، مقدار پیش‌بینی‌شده بر اساس معیار جدید با داده‌های آزمایشگاهی سازگارتر است. مشاهده شد که دقت تخمین معیار جدید برای درزه‌های طبیعی و بدون پر شدگی مناسب است.

واژگان کلیدی **درزه های طبیعی، مقاومت برشی، مورفولوژی، فتوگرامتری**