Методика. Использован аналитический метод исследований, основанный на фундаментальных положениях механики сплошных сред.

Результаты. Рассмотрены механизмы процессов разрушения породы в зоне мелкодисперсного разрушения. Выполнена оценка зависимостей от времени увеличения радиуса скважины, скорости смещения ее стенок и изменения давления продуктов детонации за первые 300 мкс. Показано, что в зоне мелкодисперсного разрушения давление убывает по экспоненте и обратно пропорционально корню квадратному из расстояния до оси скважины.

Научная новизна. Установлено, что основным механизмом в зоне мелкодисперсного разрушения является мгновенное разрушение породы от сдвиговых напряжений. Размер частиц, на которые разрушается порода, прямо пропорционален ширине зоны химических реакций во взрывчатом веществе. Используя уравнение адиабаты для продуктов детонации с постоянным показателем, при учете возбуждения в породе сильной волны сжатия, оценены зависимости изменения от времени радиуса полости взрыва, скорости разрушения породы на контакте с продуктами детонации и давление продуктов детонации. Показано, что в зоне мелкодисперсного разрушения давление убывает по экспоненциальной зависимости от расстояния до оси скважины.

Практическая значимость. Результаты работы позволяют разработать взрывчатые вещества с малой зоной мелкодисперсного разрушения, обосновать параметры зарядов с инертными и водными промежутками, что дает возможность снизить величину зоны мелкодисперсного разрушения.

Ключевые слова: *взрыв, зона мелкодисперсного разрушения, волны разрежения и сжатия, ударные волны*

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O. M. Shashenko, Dr. Sc. (Tech.), Prof.,
O. S. Kovrov, Cand. Sc. (Tech.), Assoc. Prof.

State Higher Educational Institution "National Mining University", Dnipro, Ukraine, e-mail: shashenkoa@ nmu.org.ua; kovrova@nmu.org.ua

COMPARATIVE ANALYSIS OF TWO FAILURE CRITERIA FOR ROCKS AND MASSIFS

О. М. Шашенко, д-р техн. наук, проф.,
О. С. Ковров, канд. техн. наук, доц.

Державний вищий навчальний заклад „Національний гірничий університет", м. Дніпро, Україна, e-mail: shashenkoa@ nmu.org.ua; kovrova@nmu.org.ua

ПОРІВНЯЛЬНИЙ АНАЛІЗ ДВОХ КРИТЕРІЇВ РУЙНУВАННЯ ГІРСЬКИХ ПОРІД І МАСИВІВ

Purpose. The analysis of two failure criteria for rocks being in the stress-strain state.

Methodology. The study is based on an integrated approach with the use of analysis and synthesis of the literature sources on the topic related to failure of the rocks with heterogeneous structure, and use of analytical and empirical failure criteria to assess the strength of rocks.

Findings. The analysis of the two failure criteria for compliance with the results of laboratory testing of rocks in the volumetric stressed state is carried out. It is established that the expressions of both analytical criteria reflect the process of rock failure by introducing factors that take into account the mining and geological conditions and mining technology: in the Hoek-Brown criterion – m_b, s, a, D, GSI; in the O. M. Shashenko criterion – ψ, η_0, l_t. Both criteria meet the results of laboratory tests provided m_i coefficient from the Hoek-Brown analytical expression, which takes into consideration rock structure and genesis, should not exceed 4 (mi ≤ 4).

Originality. Analytical comparison of two criteria has shown that, taking into consideration scattering experimental points obtained as a result of laboratory testing of rocks in the volumetric stressed state when $0 \leq m_i \leq 4.0$, they reflect the fact of the destruction of structurally inhomogeneous rock quite well. However, the Hoek-Brown criterion does not fully take into account the components of the spherical stress tensor ($I = \sigma_1 + \sigma_3$) and if the $m_i > 4$ its application requires additional study.

Practical value. Comparison of the analytical criterion with the results of laboratory testing of structurally heterogeneous materials in the volumetric stressed state allows predicting the rock failure in the massif with the precision of 94 %.

Keywords: *rock failure criterion, Hoek-Brown criterion, A. N. Shashenko criterion, strength in uniaxial compression, geological strength index, coefficient of structural attenuation, coefficient of brittleness*

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Introduction. Choosing an adequate failure criterion for evaluation of rock destruction is one of the key points in geomechanical analysis. Priority is given to those criteria which sufficiently describe the behavior of both homogeneous and inhomogeneous rock mass being in the volumetric stressed state.

Analysis of recent research and objectives of the article. Despite some discrepancies between failure criteria and the data obtained from testing rocks in 3D compression state, lots of scientists have proposed analytical equations to fit the available laboratory data. Thus, P. R. Sheorey et al. presented the manner in which five parameters obtained from the triaxial compression test, in particular the compressive strength, tensile strength and shear strength, friction coefficient and cohesion can be interconnected by three equations (Sheorey, Biswas & Choubey). Compliance of the failure criterion has been studied in a rock mass of coal seams, mine workings and pillars.

A. Jaiswal and B. K. Shrivastva proposed a new generalized criterion for solving the problems of rock failure in the volume that takes into consideration the expansion effect and allows describing three-dimensional state of the rock massif [1]. Comparative analysis with the Hoek-Brown criterion has shown that σ_c and m_i are the most essential parameters for assessment of intact rock failure.

The experimental data revealed that strength of geo-materials, such as soil and rocks, largely depends on the intrinsic anisotropy, and other factors, such as layering and the effect of the intermediate principal stress, which is not always sufficiently described by isotropic failure criterion. Z. Gao et al. presented a generalized failure criterion for geo-materials with transverse anisotropy for predicting the strength of clay, sand and some rocks with complex strength characteristics caused by the massif anisotropy [2].

Despite the destruction of a large number of criteria used for geomechanical analysis of rocks, the issue of the most optimal and adequate criteria remains open. Thus, according to the forecast of the rock failure while drilling with the use of thirteen criteria, R. Rahimi et al. established that the most appropriate failure criteria are modified Lade, modified Wiebols-Cook and Mogi-Coulomb [3]. In comparison with the above mentioned the Tresca, von Mises and Drucker-Prager criteria yield higher values of rock strength and, as a consequence, the need for large amount of mud.

A. Elyasi and K. Goshtasbi presented a comparative analysis of the failure criteria to assess the stability of boreholes at two oil fields in Iran [4]. The maximum and minimum allowable values of the drilling fluid pressure in two wells were calculated in Fish module of the FLAC engineering software using the Mohr-Coulomb, Mogi-Coulomb, Hoek-Brown criteria. According to the results of calculations, the Mogi-Coulomb criterion is recommended to evaluate the stability of the wells, since it most adequately describes the geomechanical processes during drilling, which satisfactorily agrees with the data of field research.

The parameters of the Hoek-Brown criterion (σ_{ci}, m_i and s) are significantly affected by the strength of anisotropic intact rock mass. In the study of H. Saroglou and G. Tsiambaos the criterion was modified to include a new parameter ($K\beta$) for taking into account the impact of the strength of anisotropic rock under load at different positions of the plane anisotropy (Saroglou & Tsiambaos). The range of $K\beta$ parameter for specimens of metamorphic rocks (gneiss, schist, marble) has been tested analytically and investigated by triaxial testing with different orientations of the layers.

Many types of natural rocks have inherent anisotropic plane, such as bedding, layering, etc. These structural features are responsible for the anisotropy of rock strength and deformation properties. The Hoek-Brown failure criterion takes into account the anisotropy through the special parameter $K\beta$. M. A. Ismael et al. proposed a simplified method of accounting for the influence of the rock anisotropy directly from uniaxial compression tests instead of triaxial tests to determine the anisotropy parameter $K_{\min}$ and reduce the volume of experimental work [5].

In the case when several parameters influence the total stability of the rock mass or geotechnical constructions, the application of probabilistic analysis with a random distribution of the rock massif properties to improve the durability of geotechnical design is proper. So, the probability of the rock failure and effectiveness of technology during underground mining depends on the depth of coal layers and moisture of the rock massif [6].

Currently, there are a number of studies devoted to the impact of intermediate stress σ_2 on the stress-strain state of the tested solid medium, which conceptually can be divided into 2 directions. Thus, A. D. Alekseev and N. V. Nedodayev have proved that the influence of σ_2 on the material deformation behavior does not exceed 8.5 % (Alekseev & Nedodayev). However, there are studies that show a significant impact of σ_2 on development of deformations in the solid. Thus L. B. Colmenares and M. D. Zoback consider seven different failure criteria, comparing them with available test data for five different rock types in volumetric compression ($\sigma_1 > \sigma_2 > \sigma_3$) under various stress conditions (Colmenares & Zoback). It was established that modified criteria Wiebols and Lode showed good agreement for the most of the test data with high dependence on intermediate stress σ_2 (dolomite, limestone). This statement requires serious analysis, since it does not fit well confirmed Mohr's hypothesis. These results give rise to logical questions concerning the design of the machine (device) for measuring the stress-strain state of the tested specimen and the experiment procedure.

However, for some rocks (sandstone, slate) intermediate stress has almost no effect on the failure process and the Mohr-Coulomb and Hoek-Brown criteria reflect the test data under volumetric compression better than other criteria [7].

The presented above analysis of various failure criteria shows their ambiguous application for description of the rock massif behavior.

The objective of this paper is to analyze two failure criteria of Hoek-Brown and A. N. Shashenko to describe the behavior of a rock mass under the conditions of volumetric stress-strain state.

Comparative analysis of two criteria. All known failure criteria can be divided into two major groups according to the way of their obtaining: analytical and empirical ones. Thus, the analytical criteria include those proposed by A. Griffith, Tresca-Saint Venant, Yu. I. Yagna, P. P. Balandin, I. N. Mirolyubov, A. N. Shashenko, and many others (Shashenko et. al.).

The analytical failure criterion proposed by A. N. Shashenko in many respects is similar to the A. Griffith failure criterion (Griffith), the general formula for which looks as follows

$$4\tau^2 - 2a\sigma R_c - bR_c^2 = 0. \tag{1}$$

When $a = 1 - \psi$ and $b = \psi$, we obtain the dependence proposed by A. N. Shashenko, but when $a = 1$ and $b = 0.25$ we obtain the formula proposed by A. Griffiths. Here $\psi = \dfrac{R_p}{R_c}$ is brittleness index, R_p and R_c are uniaxial tensile and uniaxial compression strengths respectively, $\tau = \dfrac{\sigma_1 - \sigma_3}{2}$, $\sigma = \dfrac{\sigma_1 + \sigma_3}{2}$, σ_1, σ_3 are maximum and minimum principal stresses.

Transition from the R_c strength of rock specimens to the $R_м$ strength of objects much larger in size (rock massif) is carried out through the coefficient of structural attenuation (Shashenko et al.)

$$k_c = \frac{R_м}{R_c} = 1 - \sqrt{0.5\eta}\exp(-0.25\eta),$$

where η is coefficient of rock mass strength variation, determined according to the formula

$$\eta = \sqrt{\frac{l_t - l_0}{l_t}(\eta_0^2 + 1) - 1}.$$

Here l_t is the average distance between the cracks; l_0 is a typical rock sample size; η_0 is uniaxial compression strength variation coefficient for rock specimens.

Among the most popular and well-known empirical failure criteria, those proposed by O. Mohr, Z. T. Bieniawski, Hoek-Brown and others are widely used in geotechnical calculations. The empirical strength criteria are obtained through the processing results of laboratory testing of rocks under complex stress-strain states and related field measurements. In the strict sense, their use should be limited to those rocks and geological conditions of the experiment, which are subsequently subjected to generalization on the basis of statistical and mathematical analysis of measurement results.

Let us consider the empirical failure criterion proposed by Evert Hoek and Edwin T. Brown (Hoek), which is very popular in geomechanics. Its generalized formula looks as

$$\sigma_1 = \sigma_3 + R_c\left(m_b\frac{\sigma_3}{R_c} + s\right)^a, \tag{2}$$

where m_b is the Hoek-Brown constant, taking into account the genesis and state (quality) of the rock mass, s and a are constants arising from the approximation of the power function of the envelope of limit stress circles obtained by bulk sample compression.

For intact (undisturbed) rock mass, the dependence (2) is transformed into the following formula

$$\sigma_1 = \sigma_3 + R_c\left(m_i\frac{\sigma_3}{R_c} + 1\right)^{0,5}. \tag{3}$$

Here the m_i constant, unlike the m_b constant, considers the genesis and structure of the rock mass ($0 \le\le m_i \le 33$). A larger *mi* value corresponds to fragile rocks, and a smaller *mi*, value corresponds to plastic rocks respectively. The stress state when m_i = 0 corresponds to the state of perfect plasticity ($R_c = R_p$). The equivalents of the m_i constant in the failure criterion (1) are the variation coefficient η_0 and brittleness coefficient ψ. In fact, the equation (3) reflects the pattern of rock specimen failure in laboratory testing.

For disturbed rock mass m_b constant is defined as follows

$$m_b = m_i\exp\left(\frac{GSI - 100}{28}\right),$$

where GSI (Geological Strength Index) presents a parameter that takes into account geological features of rock mass, in particular, its structure and occurrence of cracks ($5 \le \text{GSI} \le 100$). The GSI parameter is very similar to the RMR parameter (Rock Mass Rating) proposed by Z. T. Bieniawski.

For the rock mass of "good quality" (GSI > 25) we obtain the following values of s and a

$$s = \exp\left(\frac{GSI - 100}{9}\right), \quad a = 0.5.$$

Respectively, for the rock mass of "poor quality" (GSI < 25) the values are

$$s = 0, \quad a = 0.65 - \frac{GSI}{200}.$$

In order to smooth the transition from the solid rocks (with "good quality") to very poor (with "poor quality"), an additional parameter D (as "disturbance factor") has been taken into consideration. D takes into account the disturbance of the rock mass, for example, as a result of blasting. Thus, the constants m_b, s and a can be expressed through the D parameter by the following relationships

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right);$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right);$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right).$$

The D parameter possesses the values in the range from 0 for an intact rock to 1.0 for a very disturbed rock. The values of the D parameter are selected on the basis of visual observation of rocks "in situ" and assessment of their quality and disturbance degree.

It should be noted that the failure criterion (1), the characteristics of the rock mass (genesis, disturbance, structure, etc.) are considered by introducing three parameters of ψ, η_0, l_t. The values of these parameters are obtained quite objectively through the results of laboratory testing and geological surveys. The generalized Hoek-Brown failure criterion suggests introduction of five parameters of m_b, s, a, GSI, and D, the definition of which has to some extent a subjective procedure. In an attempt to take into account all peculiarities of the rock mass in analytical expression inevitably makes empirical relations more cumbersome and less accurate due to the value dispersion for each of the input parameters. The error inevitably arises in the calculation of geotechnical constructions due to uncertainty when choosing initial parameters. Its value is smaller for more plastic rocks, and respectively larger for more brittle rocks.

The considered above criteria, even though they were obtained from various preconditions, are very similar. Equations (1) and (3) in relation to the rock specimen failure made of very brittle materials ($\psi = 0$, $m_i = 33$), can be reduced to the following form

$$\sigma_1 = \sigma_3 + \sqrt{0.25\left(2\sigma_3 + R_c k_c\right)^2 - \sigma_3^2 + \sigma_3 k_c} + 0.5 R_c k_c; \quad (4)$$

$$\sigma_1 = \sigma_3 + R_c \sqrt{\frac{m_i \sigma_3}{R_c} + 1}. \quad (5)$$

In order to compare criteria (4) and (5), the two of their parts should be divided by R_c.

As a result, the following expressions are obtained

$$\bar{\sigma}_1 = \frac{\sigma_1}{R_c} = \frac{\sigma_3}{R_c} + \sqrt{0.25\left(\frac{2\sigma_3}{R_c} + k_c\right)^2 - \frac{\sigma_3^2}{R_c^2} + \frac{\sigma_3 k_c}{R_c}} + 0.5 k_c;$$

$$\bar{\sigma}_1 = \frac{\sigma_1}{R_c} = \frac{\sigma_3}{R_c} + \sqrt{\frac{m_i \sigma_3}{R_c} + 1},$$

or

$$\bar{\sigma}_1 = \bar{\sigma}_3 + \sqrt{0.25\left(2\bar{\sigma}_3 + k_c\right)^2 - \bar{\sigma}_3^2 + \bar{\sigma}_3 k_c} + 0.5 k_c; \quad (6)$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 + \sqrt{m_i \bar{\sigma}_3 + 1}. \quad (7)$$

For plastic materials (e.g., wet clays) under $\psi = 1$ and $mi = 0$ the expressions (1) and (3) become identical

$$\sigma_1 - \sigma_3 = R_c k_c;$$

$$\sigma_1 - \sigma_3 = R_c.$$

For brittle rocks $\psi = 0$ and $k_c = 1$ in the criteria (1), and $m_i = 33$ for the criteria (3) should be accepted. Then from the equations (6) and (7), we obtain

$$\bar{\sigma}_1 = \bar{\sigma}_3 + \sqrt{0.25\left(2\bar{\sigma}_3 + 1\right)^2 - \bar{\sigma}_3^2 + \bar{\sigma}_3} + 0.5; \quad (8)$$

$$\bar{\sigma}_1 = \bar{\sigma}_3 + \sqrt{33\bar{\sigma}_3 + 1}. \quad (9)$$

The Figure presents the results of laboratory testing of various rocks in the volumetric stressed state obtained by A. N. Stavrogin and A. G. Protosenya (Stavrogin & Protosenya). These curves correspond to the dependencies (8) and (9). It is implied that both criteria fully coincide for plastic rocks ($\psi = 1$, $m_i = 0$). For brittle rocks ($\psi = 0$, $m_i = 33$), the curve corresponding to the dependence (8), coincides with the results of laboratory tests with an accuracy of $R_2 = 0.90$. The curve corresponding to equation (8), when $m_i = 33$ lies far away from the results of rock testing and agrees fairly well with them when $m_i = 3$. Since there is no reason not to trust the test data obtained by A. N. Stavrogin and A. G. Protosenya for rocks in volumetric stressed state, this circumstance requires additional analysis. Perhaps the reason lies in an incomplete account of the components of spherical stress tensor in the basic equation (2).

The difference in the structure of formulas (1) and (3) can be explained by the fact that the first case completely considers the fact of material failure by shear ($\sigma_1 - \sigma_3$) and separation ($\sigma_1 + \sigma_3$), while in the second case the failure is not entirely considered by its shear ($\sigma_1 - \sigma_3$) and the value of σ_3. The extent to adequacy of analytical criteria to the failure of such structurally inhomogeneous materials as rocks can be determined only on the basis of compliance with the results of laboratory testing of rocks in non-uniform stressed state ($\sigma_1 > \sigma_2 = \sigma_3$).

The Figure presents such comparison of two criteria for adequacy, from which it follows that taking into

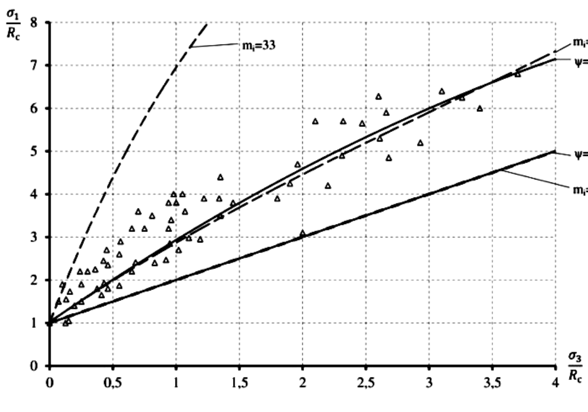

Fig. Comparison of analytical equations of failure criteria (4) and (5) with results of laboratory testing of rocks (by A. N. Stavrogin A. G. Protosenya) account the scattering experimental points obtained as a result of testing rock specimens in volumetric stressed state at $0 \le m_i \le 4.0$, they reflect quite well the fact of the destruction of such structurally inhomogeneous materials as rocks. However, at the values of $m_i > 4.0$ the curves that correspond to the Hoek-Brown failure criterion are located substantially above the experimental points [8, 9]. Considering the popularity of this criterion, this fact requires further investigation.

Conclusions.

1. Two failure criteria (A. N. Shashenko and Hoek-Brown) for compliance with the results of rock specimen laboratory testing in the volumetric stressed state has been analyzed.

2. Matching both criteria with analytical expressions for a particular rock type and geological conditions is carried out by introducing special coefficients: m_b, s, a, D, GSI – for the Hoek-Brown criterion; ψ, η_0, l_t – for the A. N. Shashenko criterion. In this case, the D parameter takes into account the disturbance of rock mass during blasting operations ($0 \le D \le 1$).

3. It is shown that the structure of analytical expressions of both criteria is similar; however, the Hoek-Brown criterion does not fully consider the components of the stress spherical tensor ($I = \sigma_1 + \sigma_3$).

4. Both criteria match the results of laboratory tests provided the *m*i coefficient, which takes into account the structure and genesis of rocks in analytical equation of the Hoek-Brown criterion, does not exceed 4 ($m_i \le 4$).

5. Guidelines of Hoek and Brown in the case for $m_i > 4$ require further study and substantiation.

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Сдвижкова Е. А. Анализ закономерностей формирования нагрузки на крепь при проектировании монтажных камер струговых лав в условиях шахт Западного Донбасса / Е. А. Сдвижкова, Д. В. Бабец, А. В. Смирнов // Науковий вісник Національного гірничого університету. – 2014. – № 5. – С. 26–32.

Мета. Аналіз двох критеріїв руйнування гірських порід, що знаходяться в об'ємному напружено-деформованому стані.

Методика. Дослідження базуються на комплексному підході з використанням аналізу та узагальнення літературних джерел з тематики руйнування гірських порід з неоднорідною структурою, застосуванні аналітичних і емпіричних критеріїв руйнування для оцінки міцності гірських порід.

Результати. Виконано аналіз двох критеріїв руйнування на предмет їх відповідності результатам випробувань гірських порід в об'ємному напруженому стані. Встановлено, що аналітичні вирази обох критеріїв відображають процес руйнування гірських порід за допомогою введення коефіцієнтів, які враховують гірничо-геологічні умови та технологію розробки родовища: у критерії Хоєка-Брауна – m_b, s, a, D, GSI; у критерії О. М. Шашенка – ψ, η_0, l_t. Результатам лабораторних випробувань у повній мірі відповідають обидва критерії за умови, що коефіцієнт m_i, який враховує структуру та генезис порід в аналітичному виразі Хоєка-Брауна, не повинен перевищувати 4 ($m_i \le 4$).

Наукова новизна. Аналітичне порівняння двох критеріїв показало, що, з урахуванням розкиду експериментальних точок, отриманих у результаті лабораторних випробувань гірських порід в об'ємному напруженому стані при

$0 \le m_i \le 4.0$, вони досить добре відображають факт руйнування структурно неоднорідних гірських порід. Однак, критерій Хоєка-Брауна не в повній мірі враховує компоненти кульового тензора напружень ($I = \sigma_1 + \sigma_3$) і при $m_i > 4$ його застосування вимагає додаткового обґрунтування.

Практична значимість. Порівняння аналітичних критеріїв з результатами лабораторного тестування зразків гірських порід в об'ємному напруженому стані дозволяє з точністю 90 % прогнозувати руйнування гірських порід у масиві.

Ключові слова: *критерій руйнування гірських порід, критерій Хоєка-Брауна, критерій О. М. Шашенка, межа міцності на одновісне стискання, коефіцієнт геологічної міцності, коефіцієнт структурного ослаблення, коефіцієнт крихкості*

Цель. Анализ двух критериев разрушения горных пород, находящихся в объемном напряженно деформированном состоянии.

Методика. Исследования базируются на комплексном подходе с использованием анализа и обобщения литературных источников по тематике разрушения горных пород с неоднородной структурой, применении аналитических и эмпирических критериев разрушения для оценки прочности горных пород.

Результаты. Выполнен анализ двух критериев разрушения на предмет их соответствия результатам испытаний горных пород в объемном напряженном состоянии. Установлено, что аналитические выражения обоих критериев отражают процесс разрушения горных пород посредством введения коэффициентов, учитывающих горно-геологические условия и технологию разработки месторождения: в критерии Хоека-Брауна – m_b, s, a, D, GSI; в критерии А. Н. Шашенко – ψ, η_0, l_t. Результатам лабораторных испытаний в полной мере соответствуют оба критерия при условии, что коэффициент m_i, учитывающий структуру и генезис пород в аналитическом выражении Хоека-Брауна, не должен превышать 4 ($m_i \le 4$).

Научная новизна. Аналитическое сравнение двух критериев показало, что, с учетом разброса экспериментальных точек, полученных в результате лабораторных испытаний горных пород в объемном напряженном состоянии при $0 \le m_i \le 4.0$, они достаточно хорошо отражают факт разрушения структурно неоднородных горных пород. Однако, критерий Хоека-Брауна не в полной мере учитывает компоненты шарового тензора напряжений ($I = \sigma_1 + \sigma_3$) и при $m_i > 4$ его применение требует дополнительного обоснования.

Практическая значимость. Сравнение аналитических критериев с результатами лабораторного тестирования образцов горных пород в объемном напряженном состоянии позволяет с точностью 90 % прогнозировать разрушение горных пород в массиве.

Ключевые слова: *критерий разрушения горных пород, критерий Хоека-Брауна, критерий А. Н. Шашенко, предел прочности на одноосное сжатие, коэффициент геологической прочности, коэффициент структурного ослабления, коэффициент хрупкости*

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