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ESTIMATION OF ROCK MASS STABILITY BASED ON PROBABILITY APPROACH AND RATING SYSTEMS

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ОЦІНКА СТІЙКОСТІ МАСИВУ ГІРСЬКИХ ПОРІД, ЩО БАЗУЄТЬСЯ НА ЙМОВІРНІСНОМУ ПІДХОДІ ТА РЕЙТИНГОВИХ КЛАСИФІКАЦІЯХ

Methodology. Different types of rock mass classifications are analyzed and discussed. Advantages and disadvantages of rating systems are clarified to improve the system based on the structural factor. Statistical strength theory is used to determine the structural factor considering rock mass heterogeneity. A correspondence is set up between the structural factor and the factors reducing computed strength of rock mass obtained using Hoek-Brown's and Palstrom's approaches.

Findings. Quantitative comparison of the structural factors which approximate laboratory strength evaluation to real rock mass strength and are based on different approaches is carried out. This allowed comparing the different classifications of rocks and conducting geomechanical calculations using alternative methods. The correspondence between structural factor and geological strength index is specified. This provides a possibility of designing mining excavations using a good proven generalized Hoek-Brown strength criterion considering natural and technogenic disturbance of rocks.

Originality. For the first time the structural factor is represented as a piecewise monotonic function. The relationship between structural factor and Geological Strength Index and Rock Mass Index classifications for geomechanical calculations is established. The significant influence of discontinuities and the type of filler between joints on mechanical properties of a rock mass is shown. The need of considering the discontinuity surface conditions while structural factor defining is clarified.

Practical value. The relation between structural factor and Geological Strength Index allows creating the technique for application of GSI rating system and Hoek-Brown failure criterion in excavation design in terms of Ukrainian mines.

Keywords: *rock mass classifications, statistical strength theory, structural factor, geological strength index*

Introduction. Rock mass classification systems are extremely important in rock engineering and designing. The purpose of classifications is to group rocks with similar properties to forecast the main features of the rock mass. Rock mass classification systems are commonly used at initial stages of geotechnical projects to compensate the lack of information about the object and as a basis for physical and mathematical simulation.

This paper is focused on comparison of different kinds of rock mass classifications (Ukrainian and international ones) and adapting the rating stability systems which are the most widely used abroad to geological conditions of coal mines in Ukraine.

This issue is particularly relevant in connection with the globalization of mining, expansion of machinery and equipment market and involvement of foreign technologies into practice of Ukrainian civil engineering [1, 2].

The following parameters are commonly used to classify the rock masses: strength and deformation characteristics of the rock mass (compressive and tensile strength, M.M. Protodyakonov's hardness factor and modulus of elasticity); parameters of discontinuity (orientation and condition of joints; surface roughness, infilling and weathering); groundwater conditions; initial stress field (for deep excavation).

One of the first rock classifications is the Protodyakonov's rock hardness scale. In this scale rocks are matched to one or another category according to a complex metric called hardness coefficient. This classification is based on the assumption that the rock destruction (failure) occurs mainly due to overcoming the compressive strength. However, this classification does not consider the tensile and shear strength.

The later classifications associate the categories of rocks stability with the value of the excavation contour displacement. This approach created a foundation of construction norms and regulations described in [3].

According to the classification adopted in Ukrainian regulations any excavation could be associated with the stability category based on the absolute values of contour displacements. The displacements should be measured in the roof, floor and sides of the excavation separately and these values are constitutive geomechanical parameters for determining the rock pressure and further selection of support type.

However, displacements of the excavation contour pointed in this classification, do not associate directly with excavation depth and physical-mechanical parameters of the rock mass. It could be considered as some imperfection of the mentioned classification.

Nowadays, some classifications in Ukraine involve parameters of the rock mass structure. According to [4] these regulations divide rocks into classes with respect to stability of the immediate roof and floor of the excavation. The parameters involved are the average value of contour convergence; critical size of mined-out space provoking the roof collapse behind a longwall; the length of unsupported span; the average compressive strength and the distance between joints (spacing). Each class associates with the technological scheme of support design defining the rock pressure and support resistance. It is obvious that the required bearing capacity of lining (support) and protective means in specific geological and mining conditions should be clarified by obtaining objective information *in situ*.

The analysis of classifications shows that some of them are too general and do not contain recommendations for the excavation support. Others relate to specific technologies of mineral extraction and contain numerical values of geomechanical parameters which could not correspond to reality under other geological conditions. This issue has a special importance while introducing new mining equipment, improving mining techniques and developing innovative technical solutions.

In such cases, the design of underground constructions must be preceded by research work including visual and instrumental observation of rock behavior under natural conditions, statistical data processing and modeling the geomechanical processes [5], both physical and mathematical (numerical). When a numerical model of geotechnical process is created the rock could be associated with a particular category of stability for the correct alignment of the laboratory test results with data concerning the structural features of the rock mass *in situ*.

Thus certain quantitative indexes (ratings) should follow from the fact that rocks are attributed to the particular class of stability. These ratings should be used to identify the main geomechanical characteristics required for excavation design. This approach is widely used in international rock mass classifications (rating systems).

Analysis of rock mass rating classifications. Since the second half of the 20th century the rating systems of rocks quality assessment has been widely used in rock engineering. The Rock Mass Rating (RMR) system was developed by Bieniawski in "Engineering rock mass classification" to assess the stability and support requirements of tunnels. To classify a rock mass, the RMR system considers the following six basic parameters: uniaxial compressive strength; Rock Quality Designation (RQD); discontinuity spacing (distance between joints); condition of discontinuity surfaces; groundwater conditions; orientation of discontinuities relative by the engineered structure.

Several modifications have been made to Bieniawski's Rock Mass Rating system by Laubscher. The modified Rock Mass Rating (MRMR) adjusts the basic RMR value considering the *in situ* and induced stresses and the effects of blasting and weathering.

The RMR and MRMR systems provide a set of guidelines for the selection of rock reinforcement for tunnels and mining excavations. These guidelines depend on factors such as a depth below surface (*in situ* stress), tunnel size and shape, and method of excavation. However, as it was mentioned above, the values of geomechanical parameters under various mining conditions can vary significantly. That is why the output of the RMR system can lead to over design of support systems. Additional limitation of the RMR system is low reliably in weak rock masses.

The rating parameters were used most systemically by Hoek and Brown [6] to apply the empirical strength theory in geomechanical calculations. They proposed an empirical criterion developed through curve-fitting of triaxial test data. The original Hoek-Brown criterion is defined as

$$\sigma_1' = \sigma_3' + \sqrt{mR_c\sigma_3' + sR_c^2}, \qquad (1)$$

where m and s are constants depending on the rock mass genesis; R_c is uniaxial compressive strength of the intact rocks; σ_1 and σ_3 are major and minor principal stresses at failure.

For intact rock $s = 1$ and $m = m_i$. Values for *mi* can be calculated from laboratory triaxial testing of core samples or extracted from reported test results. The values for each of these parameters can be difficult to assess since there is no fundamental relation between the constants in the criterion equation (1) and physical characteristics of the rock mass. That is why the following empirical relations for constants m and s were presented using the RMR system (Table 1).

Hence, for jointed rock mass the values of Hoek-Brown parameters are within the inequalities $0 \le s < 1$ and $m < m_i$. For underground applications in which the confining stress would not permit the same degree of loosening as would occur in a slope, the category of "undisturbed rock mass" was introduced. This would apply to all cases in which the interlocking between particles and blocks is still significant.

So, the main idea of Hoek-Brown failure criterion development is the need of correct transition from the properties of laboratory rock sample to properties of the rock mass. It provides increasing accuracy of the geomechanical parameters determination. These parameters (the size of failure zones, the rock mass displacement, etc.) are the basis for an underground constructions design.

Considering the discontinuities according to rock mass classifications. Obtaining new observation data and laboratory test results gave an opportunity to clarify and generalize the failure criterion (1). A new parameter α was introduced in the criterion to increase an accuracy of the rock mass behavior description. The generalized Hoek-Brown failure criterion is written as [5]

$$\sigma_1' = \sigma_3' + R_c\left(m\frac{\sigma_3'}{R_c} + s\right)^{\alpha}. \quad (2)$$

For rock masses of good quality with relatively tight interlocking, the constant a is equal to 0.5, thus reducing equation (2) to the equation (1) (the original criterion).

Hoek introduced the Geological Strength Index (GSI) to facilitate the determination of rock mass properties of both hard and weak rock masses for use in rock engineering. The relationship between the rock mass structure (conditions) and rock discontinuity surface conditions is used to estimate an average GSI value represented in the form of diagonal contours. This simple, fast and reliable system represents the non-linear relationship for weak rock mass and can be tuned to computer simulation of rock structures. It can provide means to quantify both the strength and deformation properties of the rock mass. The original Geological Strength Index (GSI) chart was developed on the assumption that

Table 1

The empirical relations to determine Hoek-Brown criterion constants

Disturbed Rock Masses (slopes)	Undisturbed (or Interlocking) Rock Masses
$m = m_i e^{\frac{RMR-100}{14}}$	$m = m_i e^{\frac{RMR-100}{28}}$
$s = e^{\frac{RMR-100}{6}}$	$s = e^{\frac{RMR-100}{9}}$

observations of the rock mass would be made by qualified and experienced geologists. However, there are many situations where engineering and geological staff are less comfortable with qualitative descriptions. Hence, the issue of quantifying GSI has been given priority.

In one of the last research studies [7] the author proposed to represent discontinuity surface conditions by the Joint Condition rating (JCond89) defined by Bieniawski while the blockness of the rock mass is represented by Rock Quality Designation (RQD). The value of GSI is given by the sum of these parameters according to the following relationship

$$\mathrm{GSI} = 1.5\,\mathrm{JCond}_{89} + 0.5\mathrm{RQD}. \quad (3)$$

GSI rating system is used to estimate the Hoek-Brown failure criterion parameters

$$m = m_i e^{\frac{GSI-100}{28-14D}}; \quad (4)$$

$$s = e^{\frac{GSI-100}{9-3D}}; \quad (5)$$

$$\alpha = 0.5 + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right), \quad (6)$$

where D is disturbance factor (the degree of disturbance caused by blast damage and stress relaxation).

Another example of rock mass rating and discontinuity accounting is the Rock Mass Index system proposed by Palmstrom in "RMI – a rock mass characterization system for rock engineering purposes" to characterize the rock mass strength. This system applies input of block size, joint characteristics and strength of intact rock to express the uniaxial compressive strength of a rock mass. The rock mass index is a volumetric parameter indicating the approximate uniaxial compressive strength of a rock mass. For jointed rock it is expressed as

$$RMi = \sigma_c \cdot JP = \sigma_c \cdot 0.2\sqrt{jC} \cdot Vb^D, \quad (7)$$

where σ_c is the uniaxial compressive strength of intact rock; jC is the joint condition factor; Vb is the block volume, measured in m^3 and JP is the jointing parameter, which incorporates the main joint features in the rock mass and $D = 0.37jC^{-0.2}$.

In fact, the introduction of quantitative ratings is an attempt to estimate the rock scale effect caused by plenty of factors and revealed as a dissimilarity in mechanical properties of wide scale rock mass and a little rock sample.

Really, the uniaxial compressive strength for inhomogeneous jointed rock mass R_{cm} can be expressed by setting $\sigma_3 = 0$ in the equation (2)

$$R_{cm} = R_c s^{\alpha} = R_c k_{GSI}, \quad (8)$$

where $k_{GSI} = s^{\alpha}$ could be considered as some kind of structural factor. This factor displaces the difference between the average strength of laboratory rock samples and the inhomogeneous rock mass.

Similarly, equation (7) could be considered in the same sense

$$R_{cm} = RMi = R_c \cdot JP = \sigma_c \cdot k_{JP}, \quad (9)$$

here k_{JP} has the same interpretation but may have a different value according to the *RMI* system.

The similar approach is developed by Shashenko, Sdvyzhkova & Gapeev [8] based on statistical strength theory.

Scale effect estimation based on statistical strength theory. As we mentioned above the dissimilarity in compressive strength of real rock mass R_{cm} and average compressive strength of rock sample $\overline{R_c}$ can be defined as the scale effect and estimated quantitatively by the structural factor

$$k_c = \frac{R_{cm}}{\overline{R_c}}. \quad (10)$$

There are different approaches to evaluation of this factor. According to statistical strength theory [6] the rock mass can be considered as a unit consisting of different structural elements. The compressive strength of each structural element is supposed to be a random variable R submitted to some probability distribution $F(r)$. Let the compressive strength of the real inhomogeneous rock mass be estimated by the value R_{cm}. Then it should be accepted that the strength of each structural element is not less than this value. The probability of such event is determined by the expression

$$p(R_{cm} < r < \infty) = 1 - F(R_{cm}). \quad (11)$$

The particular aspect of the equation (11) depends on a choice of the function $F(r)$. Let us assume the hypothesis of normal distribution of structural element strength. Then the expression (11) looks like

$$p(R_{cm} < r < \infty) = 1 - F_0[(R_{cm} - a)/\sigma] = p_m, \quad (12)$$

where a, σ are parameters of normal distribution: mean and standard deviation respectively; $F_0(t)$ is the probability function of the standard normal distribution.

Then equation (12) can be written as

$$F_0\left(\frac{R_{cm} - a}{\sigma}\right) = 1 - p_m$$

and solved with respect to R_{cm}: R_{cm}: $R_{cm} = \sigma \cdot t + a$.

Here $t = \arg F_0(1 - p_m)$ is an argument of function F_0 at its value $(1 - p_m)$. Let us divide both parts of last equation by a and obtain the structural factor in the right-hand side according to (10)

$$k_c = 1 + \eta \cdot t, \quad (13)$$

where $\eta = \sigma/a$ is a variation of the random value R.

At great values of variation η the structural factor k_c becomes negative that contradicts its physical sense. This occurs because the probability of negative values becomes essential at $\eta \geq 0.33$ according to Gauss's law. So we should put forward a hypothesis of asymmetrical strength distribution for jointed rocks, in particular, logarithmically normal distribution. In this case expression for compressive strength of the rock mass R_{cm} looks like

$$R_{cm} = \exp(a + \sigma \cdot t). \quad (14)$$

Then the structural factor according to (10) considering that $\eta^2 + 1 = \exp(\sigma^2)$ is given by formula

$$k_c = \frac{\exp\left(t \cdot \sqrt{\ln\left(\eta^2 + 1\right)}\right)}{\sqrt{\eta^2 + 1}}. \quad (15)$$

At $\eta = 0$ (homogeneous material) $k_c = 1$ and compressive strength of rock mass coincides with strength of structural elements (samples). At $\eta \to \infty$ (unlimited increasing of an inhomogeneity degree) structural factor asymptotically tends to zero (Fig. 1).

The value of probability p_m depends on required reliability of the designed object. The variation η characterizes the degree of rock mass inhomogeneity on a micro-level. To consider rock heterogeneity on a macro-level some parameters of joints should be accounted.

Statistical approach to consider the rock mass heterogeneity on macro level. Let $r_1, r_2, \ldots, r_{nb}$ be the values of uniaxial compressive strength received by sample testing in a standard way. Let the examined part of rock mass be jointed and the distance between joints be l_t. While preparing samples some of them could be broken before test as they contain macro defects (joints). So, some structural elements of the rock mass do not participate in testing, but as existing ones should be included in statistics. The compressive strength of such broken specimen is supposed to be equal to zero. Let n_t be a number of the broken samples which could not be tested. The "corrected" set should include both intact and broken samples. Thus number of elements in "corrected" set is $n = n_B + n_t$. The relationship between statistical characteristics for standard tested sample set and "corrected" one has been established by Shashenko [8]. This relationship for initial moments of distribution is given by formula

$$\nu'_k = K_t \nu_k, \quad (16)$$

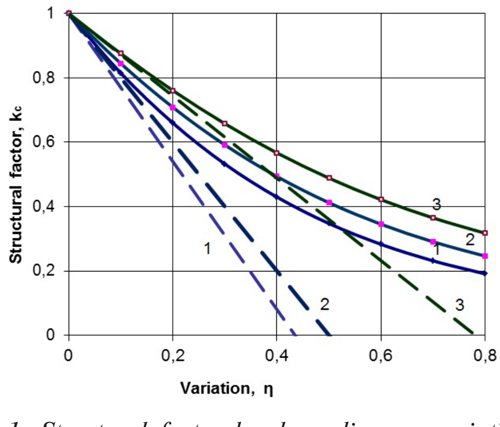

Fig. 1. Structural factor kc depending on variation of strength value η:

continuous line – logarithmic normal distribution; dotted line – normal distribution; 1 – p_m = 0.99; 2 – p_m = 0.95; 3 – p_m = 0.9 where $\nu_k = \frac{1}{n}\sum_{i=1}^{n} r_i^k$, $K_t = \frac{l_t}{l_t + l_0}$ and $l_0$is the size of a standard tested sample.

The deviation of the "corrected" sample set is connected with initial moments of standard sample set

$$D' = K_t \nu_2 - K_t^2 \nu_1^2. \qquad (17)$$

Then the variation for the "corrected" set is given by

$$\eta' = \frac{\sqrt{D'}}{\nu_1'} = \sqrt{\frac{l_t + l_0}{l_t}\left(\eta^2 + 1\right) - 1}. \qquad (18)$$

Here η *is* the variation of standard sample set. So we should use (18) in expressions (13) or (15) instead of η to account joints. The variation η' increases when a distance between joints l_t decreases. Structural factor decreases too, and strength of rock mass R_{cm} decreases respectively according to equality

$$R_{cm} = k_c \cdot \overline{R_c}. \qquad (19)$$

However, it should be taken into account that the variation of standard sample set η (initial variation) depends on joint intensity. At space between joints $l_t > 1.0m$ the initial variation η does not exceed the value $\eta = 0.1–0.25$ and reflects only the presence of micro-defects in a rock specimen. Reducing the joint space in the selected sample in the range of $0.5 < l_t < 1.0m$ results in increase in the strength variation of tested specimen up to standard value $\eta = 0.3–0.4$. The further joint space reduction provides the initial variation increasing up to $\eta = 0.5–0.6$.

Thus the variation of the "corrected" sample η' is a complex function with respect to argument l_t, since it depends on η' indirectly, through an intermediate argument η

$$\eta' = f(\eta(l_t)).$$

Since the intermediate argument η takes different values at different intervals of the argument l_t. altering, the function $\eta' = f(\eta(l_t))$ according to (18) is a piecewise function. Then the structural factor *kc* according to (15) and (18) is represented as a piecewise monotonic graph as well (Fig. 2).

Thus, the structural factor k_c determined by (13) or (15) together with (18) could be easily determined from the standard uniaxial compressive strength tests and geological data concerning the rock mass discontinuity [9].

Correlation between GSI, RMi and the structural factor kc values. As we mentioned above the equations (8, 9, 19) evaluate the rock mass strength involving the factors k_c, k_{GSI}, k_{JP} and taking into account the rock mass structure heterogeneity. Each of these factors is determined based on different approach and described by different kind of a function. But all of them provide the similar action reducing the rock strength depending on the heterogeneity level. Structural factors k_c, k_{GSI} and k_{JP} are represented in Fig. 2 depending on spacing l_t.

Analysis of Fig. 2 shows that the structural factor k_c (Kc1, Kc2, Kc3 lines in Fig. 2) based on statistical approach is close to maximum values of structural factor k_{GSI} (KGSI (very good) line in Fig. 2) based on Hoek's approach. However maximum values of k_{GSI} describe almost ideal quality of the surface conditions

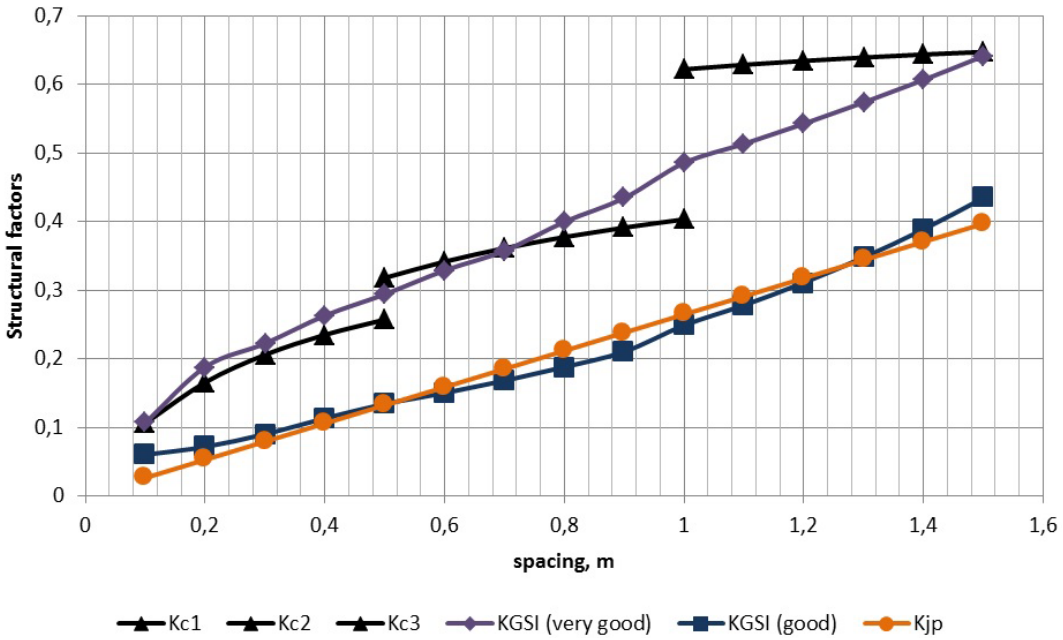

Fig. 2. Structural factors k_c, k_{GSI}, k_{JP} depending on distance between joints:

Kc1 – structural factor for $l_t < 0.5m$; Kc2 – structural factor for $0.5 < l_t < 1.0m$; Kc3 – structural factor for $l_t > 1.0m$; KGSI (very good) and KGSI (good) – structural factors based on Hoek's approach for "very good" and "good" surface conditions respectively; Kjp – structural factor based on Palmstrom's approach (conditions of discontinuities). In GSI classification such conditions are called “very good” and described as very rough and fresh unweathered surfaces. The structural factor k_{GSI} calculated considering the “good” category of surface condition in GSI classification (KGSI (good) line in Fig. 2) is similar to structural factor k_{JP} based on Palmstrom’s approach. The values of these factors are significantly less than values of *kc*. The fact of difference between the factors reducing the rock strength should be considered while defining factor k_c. In particular, conditions of discontinuities should be considered as Hoek and Palmstrom did.

The dependence between the structural factor k_c and GSI values could be established according the equation (8) considering (5) and (6). The results of comparison are represented in Table 2.

This table could be used to establish accordance between different classifications at geomechanical calculations. The structural factors k_c, k_{GSI}, k_{JP} could be used in terms of any rock strength and failure criteria to decrease the rock mass strength depending on the distance between joints.

Determination of the rock stress-strain around tunnels in a small depth below surface is carried out in terms of elastic strain. Such calculations involve only elastic modulus and Poisson ratio [10]. In this case, defining the rock elastic characteristics should be done taking into account heterogeneity of rocks as well. This could be carried out using rating classification systems or the structural factor calculated considering discontinuity characteristics.

Conclusions.

1. Analysis of rock mass rating classifications shows great attention of researchers aimed at rock mass heterogeneity accounting (in particular, influence of natural joints). Discontinuities primarily affect the rock mass strength that differs from the strength of a sample due to the presence of macrodefects. The article analyzes the approaches to assess this difference using structural factors.

2. Quantitative comparison of the structural factors based on different approaches is carried out. This allows comparing the different classifications of rocks and conducting geomechanical calculations using alternative methods. In particular, the structural factor k_c widely used in Ukrainian regulations could be matched to internationally recognized classifications Geological Strength Index and Rock Mass Index. This allows adapting internationally recognized methods and numerical simulation software for the design of excavations under condition of Ukrainian coal mines.

3. The study of various classifications shows that the conditions of discontinuities and the type of filler between joints have significant influence on mechanical properties of rock mass.

4. Analysis of the plot in Fig. 2 shows the need to consider the discontinuity surface conditions while defining structural factor k_c.

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Мета. Встановлення залежності (зв’язку) між коефіцієнтом структурного послаблення та рейтинговими покажчиками стійкості гірських порід, що використовуються у світовій практиці геомеханіки. Адаптація геологічного індексу міцності (GSI) та індексу масиву гірських порід (RMi) до ге-

Table 2

Correlation between GSI and the structural factor k_c

GSI	100	95	90	85	80	75	70	65	60	55	50	45	40
k_c	1.00	0.76	0.57	0.43	0.33	0.25	0.19	0.14	0.11	0.08	0.06	0.05	0.04

ологічних умов вугільних шахт України та класифікацій, що використовуються у практиці вітчизняного проектування.

Методика. Проаналізовані основні вітчизняні та зарубіжні класифікації гірських порід. Показані переваги та недоліки рейтингових класифікацій масиву гірських порід, що широко використовуються у практиці світового проектування підземних споруд. Використана статистична теорія міцності для визначення коефіцієнта структурного послаблення, що дозволяє враховувати масштабний ефект у гірських породах, встановлено співвідношення між коефіцієнтом структурного послаблення та коефіцієнтами, що зменшують розрахункову міцність породного масиву, одержаними на основі підходів Хока-Брауна та Палмстрома.

Результати. Виконане співставлення коефіцієнтів, що наближають лабораторні оцінки міцності до міцності реального породного масиву, які одержані на базі різних підходів до врахування масштабного ефекту, що дозволяє виконувати перехід між різними класифікаціями гірських порід та проводити геомеханічні розрахунки на основі різних підходів. Встановлена відповідність між коефіцієнтом структурного послаблення та геологічним індексом міцності (GSI), що є необхідною характеристикою узагальненого критерію міцності Хока-Брауна, який дозволяє враховувати неоднорідність породного масиву.

Наукова новизна. Уперше доведена необхідність представляти коефіцієнт структурного послаблення у вигляді кусково-неперервної функції. Встановлено співвідношення між коефіцієнтом структурного послаблення та геологічним індексом міцності. Вказана необхідність урахування впливу стану поверхні тріщин на значення коефіцієнту структурного послаблення.

Практична значимість. Співвідношення між коефіцієнтом структурного послаблення та геологічним індексом міцності дозволило розробити методику використання класифікації GSI та критерію міцності Хока-Брауна при проектуванні підземних виробок в умовах шахт України.

Ключові слова: *класифікація гірських порід, статистична теорія міцності, коефіцієнт структурного послаблення, геологічний індекс міцності*

Цель. Установление зависимости (связи) между коэффициентом структурного ослабления и наиболее используемыми за рубежом рейтинговыми показателями устойчивости горных пород. Адаптация геологического индекса прочности (GSI) и индекса массива горных пород (RMi) к геологическим условиям угольных шахт Украины и классификациям, используемым в практике отечественного проектирования.

Методика. Проанализированы основные отечественные и зарубежные классификации горных пород. Показаны преимущества и недостатки рейтинговых классификаций массива горных пород, широко используемых в практике мирового проектирования подземных сооружений. Использована статистическая теория прочности для определения коэффициента структурного ослабления, позволяющего учесть масштабный эффект в горных породах, установлено соответствие между коэффициентом структурного ослабления и коэффициентами, уменьшающими расчетную прочность породного массива, полученными на основе подходов Хока-Брауна и Палмстрома.

Результаты. Выполнено сопоставление коэффициентов, приближающих лабораторные оценки прочности к прочности реального породного массива, полученных на основе различных подходов к учету масштабного эффекта, что позволяет осуществлять переход между различными классификациями горных пород и выполнять геомеханические расчеты на основе различных подходов. Установлено соответствие между коэффициентом структурного ослабления и геологическим индексом прочности (GSI), который является необходимой характеристикой обобщенного критерия прочности Хока-Брауна, позволяющей учитывать неоднородность породного массива.

Научная новизна. Впервые доказана необходимость рассматривать коэффициент структурного ослабления как кусочно-непрерывную функцию. Установлено соответствие между коэффициентом структурного ослабления и геологическим индексом прочности. Указана необходимость учета влияния состояния поверхности трещин на значение коэффициента структурного ослабления.

Практическая значимость. Соответствие между коэффициентом структурного ослабления и геологическим индексом прочности позволило разработать методику использования классификации GSI и критерия прочности Хока-Брауна при проектировании подземных выработок в условиях шахт Украины.

Ключевые слова: *классификация горных пород, статистическая теория прочности, коэффициент структурного ослабления, геологический индекс прочности*

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