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Selection of optimal strength criteria for the terrigenous Pashiyan horizon of the Romashkinskoe field Tashliyarskaya area

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Abstract. In the process of oil reserves' development, the in-situ stresses change. Knowledge of rock failure constraints will allow prediction of behavior of rock when subject to subsurface stress change. In this study, we used the results of studies of the Pashiyan sandstone core samples recovered from the Tashliyarskaya area well No. 14403. Six sets of samples, each consisting of three samples taken from the homogeneous intervals at the same depth, were used to determine the ultimate tensile strength, uniaxial and triaxial compressive strength in the in-situ conditions. An analysis of the methods for constructing a rock strength certificate, and comparison of the strength criteria described in State Standard 21153.8-88, the Mohr-Coulomb linear strength criterion and the non-linear Hoek-Brown criterion are provided. The Hoek-Brown criterion has the advantage of describing a non-linear increase in strength with an increase in overburden pressure and more adequately reflects the properties of rock. For the first time, a comparison of applicability of strength criteria obtained by different methods and based on the laboratory core analysis was made to determine their practical applicability. Comprehensive studies of the strength characteristics have never been previously conducted, and the results obtained will serve as the basis for further analysis and application in order to improve the development of the terrigenous Devonian Romashkinskoe field.

Keywords: failure criterion, strength certificate, Mohr-Coulomb failure criterion, Hoek-Brown failure criterion, ultimate stress, tensile, uniaxial compression, triaxial compression

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In the process of oil reserves' development, the in-situ stresses change. Production enhancement operations (e.g., waterflooding, hydraulic fracturing) cause the reservoir pressure, the effective stress, stress regimes, and the reservoir temperature change both locally, and over the entire field.

Change of the in-situ stresses can cause failure of subsurface rock and alteration of the pore volume, activate the pre-existing flaws, change permeability of natural fractures, etc.

Knowledge of rock failure constraints will allow prediction of behavior of rock when subject to subsurface stress change. Ability of rock to withstand an external applied load without failure is referred to as the strength of a material. Ultimate strength of rock is determined by laboratory methods when rock samples are subject to tensile and compression loadings.

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The rock strength is defined by two components: the strength of the rock matrix and the strength of discontinuity interfaces (natural fractures, inclusions, flaws, etc.). The conditions that cause failure of the subsurface rocks can be described by the stress criteria, also known as the strength criteria. When discussing failure of rock it should be remembered that compressive strength of the geological material exceeds the tensile strength.

To describe strength criteria, the subsurface rock mechanics usually uses strength criteria defined via stresses, whereby, minimum and maximum principal stresses are only used, while the intermediate principal stress is ignored, as rule. A curve enveloping the ultimate stress circles built in the coordinates of the normal effective stress-shear stress (σ, τ) is a criterion known as a certificate of rock strength.

According to GOST R 50544-93 (State Standart, 1993), a certificate of rock strength is the relationship between the ultimate shear breaking stress and the normal stress acting on the subsurface rocks. Graphically, it is expressed as a curve enveloping stress circles.

The procedure to obtain a certificate of rock strength described in GOST 21153.8-88 (State Standart, 1988), is based on the ultimate triaxial compressive strength of not less than three core samples at different lateral pressures, and the ultimate tensile and uniaxial compressive strength of two more core samples. Five circles in *σ-τ* coordinates are built and a smooth curve enveloping all five, or more, semicircles is drawn. The described procedure requires that at least five core samples should be used, which often cannot be satisfied in practice, because of scarcity of core material.

Another procedure based on the ultimate strength under compression shear loading described in GOST 21153.5-88 (State Standart, 1988) fails to meet production and research goals.

Both procedures mentioned above use circles in the *σ-τ* coordinates based on the ultimate strength of core samples. Presented below is an alternative procedure to obtain a certificate of rock strength. This is a calculation procedure that uses an empiric equation to determine the coordinates of the points of the enveloping curve:

$$\tau = \tau_{max}\left(\frac{\sigma_к^2}{\sigma_к^2+a^2}\right)^{3/8}, \qquad (1)$$

where τ_{max} – maximum shear strength of the subsurface rock, MPa. It is assumed that fractures and pores are completely closed under the action of pressure; $\sigma_к$ – normal stress relative to the origin of coordinates transposed to the point of intersection of the enveloping curve and x-axis, MPa; *a* – parameter related to the shape of the enveloping curve.

Results of laboratory experiments on determination of the ultimate tensile and uniaxial compressive strength, as well as tabulated data given by GOST 21153.8-88 (State Standart, 1988) are used to calculate *τ* by Eq. (1).

In this study, we used the results of studies of the Pashiyan sandstone core samples taken from the Tashliyarskaya Area Well No. 14403. Six sets of samples, each consisting of three samples taken from the homogeneous intervals at the same depth, were used to determine ultimate tensile, uniaxial, and triaxial compressive strength corresponding to in-situ conditions. Table 1 summarizes the results of the laboratory experiments. Figure 1 illustrates building of a certificate of rock strength according to GOST 21153.8-88 (State Standart, 1988) procedure.

It is evident that the strength criterion does not describe accurately the circle built based on the ultimate strength and confining pressure values from the triaxial test. The compression created during the triaxial test increases the ultimate strength of a material compared to the ultimate strength obtained from the uniaxial test performed at the atmospheric pressure and zero confining pressure. It follows that it would be problematic to use GOST 21153.8-88 (State Standart, 1988) to obtain

No. of sample set	No. of sample	Core recovery depth, m	Sample size, diameter × length, mm	Test type	Ultimate strength, MPa
1	32	1625.6	30 × 60	triaxial compression	87.5
	31	1625.55	30 × 60	uniaxial compression	43.20
	33	1625.6	30 × 15	tension	2.90
2	35	1626.3	30 × 60	triaxial compression	61.0
	37	1626.35	30 × 60	uniaxial compression	25.58
	36	1626.3	30 × 15	tension	2.70
3	43	1628.43	30 × 60	triaxial compression	119.1
	41	1628.4	30 × 60	uniaxial compression	43.48
	42	1628.4	30 × 15	tension	3.50
4	45	1629.33	30 × 60	triaxial compression	144.9
	47	1629.36	30 × 60	uniaxial compression	61.27
	46	1629.33	30 × 15	tension	5.70
5	51	1630.43	30 × 60	triaxial compression	119.0
	49	1630.4	30 × 60	uniaxial compression	75.69
	50	1630.,4	30 × 15	tension	3.90
6	54	1632.25	30 × 60	triaxial compression	78.0
	56	1632.28	30 × 60	uniaxial compression	41.45
	55	1632.25	30 × 15	tension	4.40
Average			30 × 60	triaxial compression	102.0
			30 × 60	uniaxial compression	48.45
			30 × 15	tension	3.85

Table 1. Ultimate strength of core samples under tension, uniaxial and triaxial compression

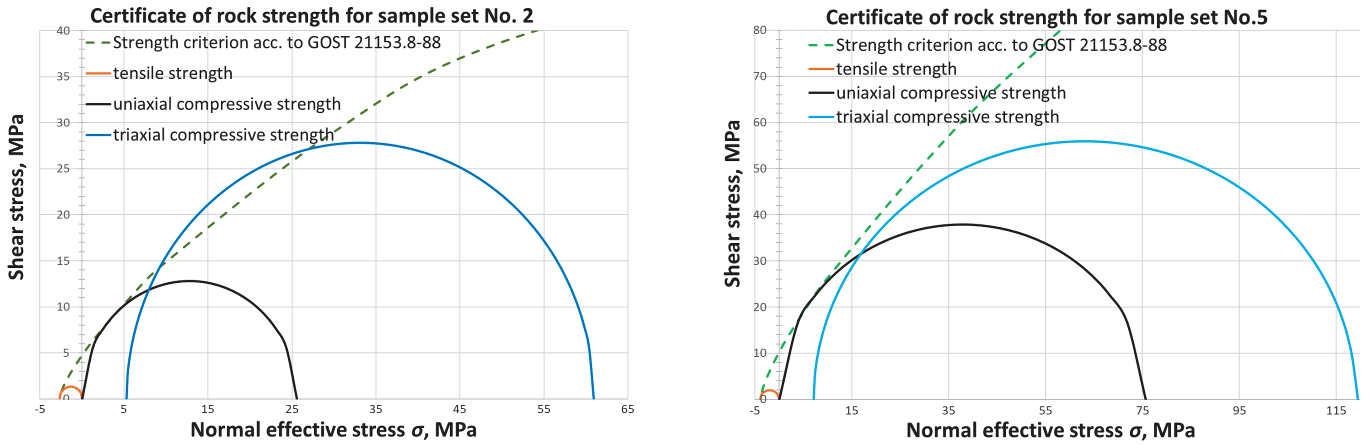

Fig. 1. Certificate of rock strength according to GOST 21153.8-88 for sample sets Nos. 2 and 5

certificates of rock mass strength based on tests under in-situ conditions.

The Mohr-Coulomb linear strength criterion (Coulomb, 1776) does not satisfy the requirements to description of the rock mass ultimate strength:

$$\tau = C + \sigma \cdot tg\varphi, \qquad (2)$$

where τ – shear stress, MPa; C – cohesion, MPa; σ – normal stress, MPa; tgφ – slope of strength criterion curve.

Mohr-Coulomb failure criterion is based on the Mohr' hypothesis of shear stress dependence upon normal stress, and the Coulomb's hypothesis of cohesion force.

Figure 2 illustrates the Mohr-Coulomb linear failure criterion. One can see that in the region of the tensile strength the criterion is not tangent to the circle, while the region beyond the ultimate strength is overestimated.

As an alternative to the GOST 21153.8-88 (State Standart, 1988) procedure and the Mohr-Coulomb equation, we have considered the Hoek-Brown failure criterion. In contrast to the Mohr-Coulomb linear failure criterion, the Hoek-Brown criterion is a non-linear relationship and has a parabolic form. It is an empirical failure criterion that describes non-linear increase of the ultimate strength of rock at increase of the effective stress.

The Hoek-Brown criterion is based on the Evert Hoek's brittle rock failure tests and the parabolic Mohr envelope obtained from the Griffith theory to determine the relationship between the shear and normal stresses at failures in rock masses. Having connected appearance of fractures with propagation of fractures and failures of rock masses, Hoek and Brown offered correction factors to adapt different parabolic curves to the laboratory triaxial tests (Hoek, Brown, 1980). So, the Hoek-Brown criterion has an advantage over the considered earlier approaches in describing the non-linear increase of strength of rock with increase of confining pressure.

Kumar P. (1988) gives the following form of the Hoek-Brown equation:

$$\sigma_1 = \sigma_3 + C_0 \cdot \sqrt{m \cdot \frac{\sigma_3}{C_0} + s}, \qquad (3)$$

where σ_1 – maximum confining load at triaxial failure test, MPa; σ_3 – minimum confining load at failure, MPa; C_o – strength at uniaxial compression, MPa; *m, s* – material parameters.

The parameter *s* varies from 1 for intact rocks to 0 for disturbed rocks. The values of the parameter *m* are derived from laboratory tests. This parameter relates to the rock brittleness, the less the *m* parameter, the more plastic the rock mass.

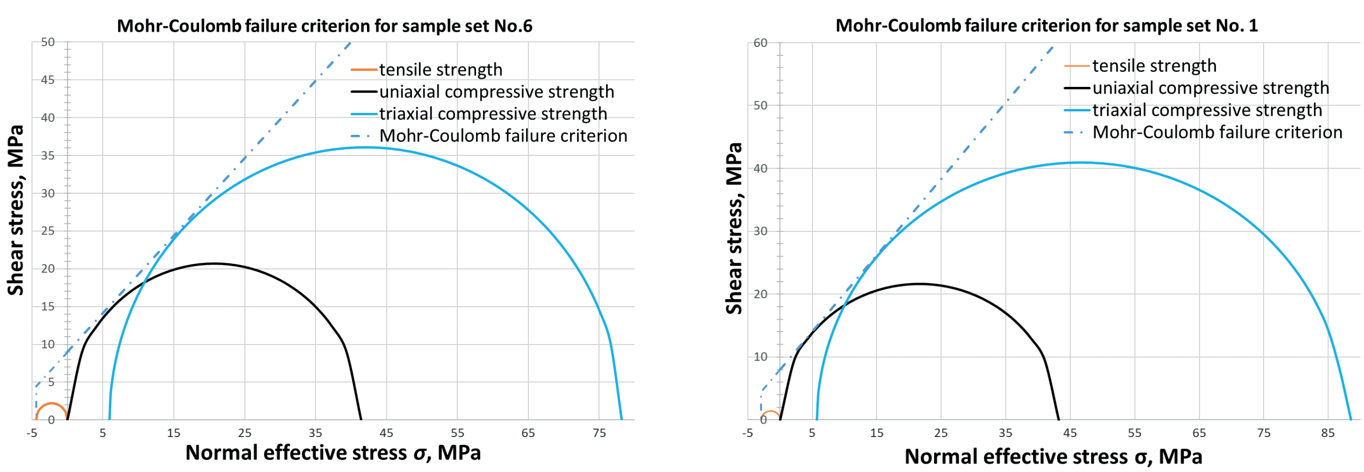

Fig. 2. Linear Mohr-Coulomb failure criterion for sample sets Nos. 1 and 6

When laboratory data is not available, reference data obtained by Hoek and Brown are used. For example, the *m* parameter may vary from 15 to 24 for sandstones (Zoback, 2010).

The Hoek-Brown criterion relates the major and the minor stresses, and to transform it to the parameters of normal and shear stresses, Kumar offered a method to calculate the tangent points in the coordinates (σ, τ) (Kumar, 1998):

$$\sigma = \sigma_3 + \frac{\sigma_1 - \sigma_3}{1+\sigma'}, \quad (4)$$

$$\tau = \frac{\sigma_1 - \sigma_3}{1+\sigma'} \cdot \sqrt{\sigma'}, \quad (5)$$

$$\sigma' = 1 + ma\left(m\frac{\sigma_3}{\sigma_c} + s\right)^{a-1}, \quad (6)$$

where σ – normal stress, MPA; τ – shear stress, MPa; σ_1 – maximum confining load at failure, MPa; σ_3 – minimum confining load at failure, MPa; σ' – differentials σ_1 and σ_3 relationship; σ_c – strength at uniaxial compression, MPa; *a, m, s* – material parameters.

The parameters *a, m,* and *s* are selected using the following relationship:

$$\frac{\sigma_1 - \sigma_3}{\sigma_c} = \left(m\frac{\sigma_3}{\sigma_c} + s\right)^a. \quad (7)$$

For the Pashiyan sandstone formation, we took the *a* parameter as 0.5, the *s* parameter as 1, for the purpose of unification. As for the *m* parameter, we determined it using Eq.(7). The results are summarized in Table 2. For the sample set No. 1, the *a* parameter was set to 0.54 to achieve a good description of the stress circles and to satisfy the conditions of Eq. (7).

No. of sample set	Parameter		
	a	*s*	*m*
1	0.54	1	17
2	0.5	1	18.2
3	0.5	1	33
4	0.5	1	33.4
5	0.5	1	12.7
6	0.5	1	14.3
Average rock parameters	0.5	1	21.4

Table 2. Values of a, m, s parameters for sample sets

From Figure 4, one can see that for the sample set No. 6, full description of stress circles in the region of tensile stress was not achieved, and the missing portion was approximated.

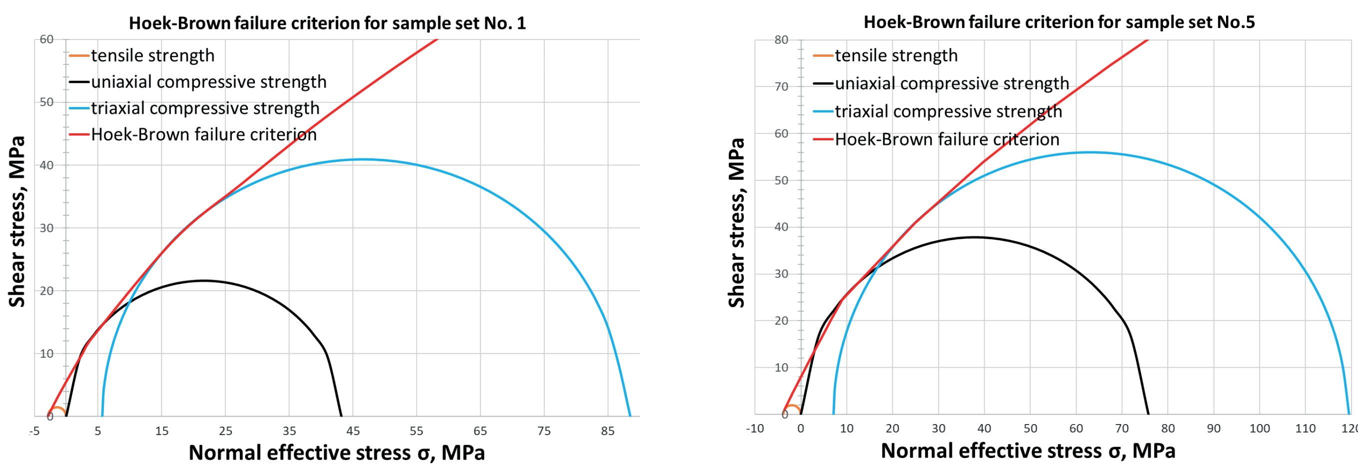

Fig. 3. Hoek-Brown failure criterion for sample sets Nos. 1 and 5

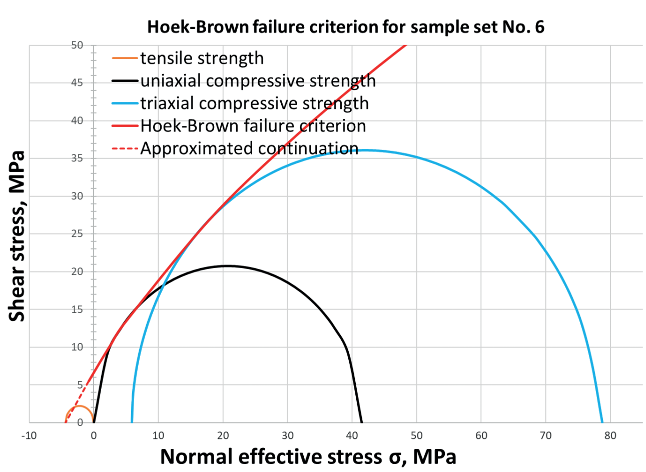

Fig. 4. Hoek-Brown failure criterion for sample sets No. 6

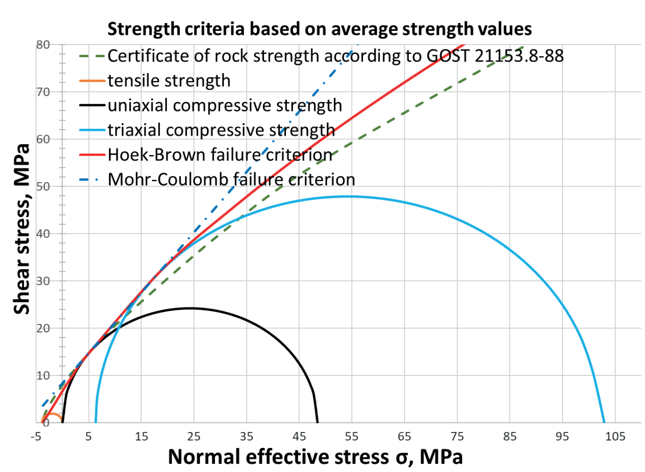

Fig. 5. Strength criteria based on average compressive strength values obtained from all sample sets under study

To determine the generalized parameters of the Pashiyan sandstone formation, we used average values of the ultimate strength determined from laboratory experiments. The parameters *a* and *m* were set to 0.5 and 1, respectively, while the s parameter was taken as an average of all values from the tests and equals to 21.4. Figure 5 shows the obtained strength criteria according to the GOST 21153.8-88 (State Standart, 1988) procedure, the linear Mohr-Coulomb failure criterion, and the Hoek-Brown failure criterion based on the average values for the Pashiyan sandstone formation.

From Figure 5 it is evident that the Mohr-Coulomb linear failure criterion overestimates the region of the tensile strength, the strength criteria according to the GOST 21153.8-88 (State Standart, 1988) procedure underestimates the boundary in the region of the compressive stress, while the Hoek-Brown criterion satisfactorily describes the stress circles in all stress regions-tensile, uniaxial, and triaxial compressive stresses.

Conclusion

For the first time, a comparison of applicability of strength criteria for the Pashiyan formation of the Romashkinskoe oil field obtained by different methods and based on the laboratory core analysis was made to determine their practical applicability.

Comprehensive studies of the strength characteristics have never been previously conducted, and the results obtained will serve as the basis for further analysis and application in order to improve the development of the terrigenous Devonian Romashkinskoe field.

The Mohr-Coulomb linear failure criterion is an empirical relationship based on data obtained by experiment, and, as such, is not reliable. The drawback of the Mohr-Coulomb failure criterion consists in its linearity, which compromises accuracy and affects results of calculations of, e.g., wellbore stability.

The Hoek-Brown criterion because of its non-linear nature more adequately reflects the properties of rock in the region of the tensile stress, and the region beyond the ultimate strength in the in-situ conditions. Being analytical, the Hoek-Brown criterion is convenient for practical application and numerical modeling of rock behavior in the Pashiyan formation of the Romashkinskoe oil field Tashliyarskaya area.

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