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STUDY OF STABILITY OF ROCKS BEYOND THEIR STRENGTH LIMIT UNDER EXTERNAL LOCAL INFLUENCES

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Abstract. The subject of the research is deformation processes in rocks beyond their strength limits under targeted external local influences. The aim of the work is to determine the influence of individual physical and technological factors on the stability of the system "rock massif - mine working - support" beyond the strength limit of rocks to substantiate the parameters for controlling the geomechanical state of the specified system at great depths. Research methods: laboratory experimental studies, mathematical modeling, analysis and generalization of results. A set of experimental studies was carried out on high-rigidity testing equipment. Research modes: one-, two- and three-axial compression up to and beyond the strength limit with different options of external local impact. The object of research is specimens of sedimentary and rocky rocks. It is confirmed that beyond the strength limit, even local impacts with low energy are capable of changing the nature of rock deformation from pseudo-brittle fracture to pseudo-plastic flow. When deformation occurs beyond the strength limit, the most significant changes occur with the residual bearing capacity of rocks. The value of residual strength depends on the minimum stress component and the relative area of application of the local impact. To develop recommendations for the practical use of local impact effects, an elastic-plastic problem was solved, taking into account the "inclusion in work" of the marginal rock massif. It is shown that for mine workings at great depths or in conditions of fractured rocks, the active involvement of the marginal massif in the work and blocking the process of rock loosening are of primary importance. Control of rock massif destruction can be achieved by its spatial reinforcement with a system of anchors, injection of bonding solutions, spray concreting and plugging of cavities behind the support. To prevent loss of stability of the mine workings, local impact must be continuous. Therefore, reinforcement must be carried out using flexible anchors with constant or smoothly changing resistance. Since the maximum displacement occurs on the contour, it is advisable to locate the anchor flexibility unit on the surface of the mine working.

Keywords: rocks, deformation beyond the strength limit, stability of mine workings, support, targeted external local impact, types of impact, recommendations.

1. Introduction

During recent decades, there is the world tendency to increase the depth of mineral development. The depth of mines already exceeds 4 km. Even individual coal mines mine at depths of 1.0–1.5 km. Under such conditions, rock pressure significantly exceeds the strength limit of rocks. Rocks at smaller depths also “work” beyond the strength limit due to their low strength or high fracturing of the massif’s marginal zone. Under such conditions, the problem of rock pressure management becomes of primary importance for the safety of mining operations. To counteract the pressure, various types of supports are used, the efficiency of which is far from 100%, and the cost is often economically unjustified [1–5].

However, it is known that sometimes minor resistance to vertical compression of rocks causes significant changes in the nature of their destruction [6–9]. The problem is that in mine conditions it is impossible to create such counteraction along the entire contour. However, it is possible to create separate zones counteracting pressure by the known methods and supports. For example, the load-bearing capacity of the most common frame support is (150–250) kPa. While blocking free destruction is only possible at a value of ≥1 MPa [7,10,11]. Therefore, this can be achieved by localizing the counteraction by reducing the area of impact by 4–7 times.

In a number of works [7–12], the patterns of change in strength and the nature of deformation of rocks were experimentally established for the conditions $\sigma_1 > \sigma_2 = \sigma_3$,

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where σ_1 is the vertical, and σ_1, σ_2 are the lateral components of the load of cubic specimens. The results were obtained under a fairly high lateral pressure, which is difficult to implement in practice. This is due to the use of testing machines of normal or insufficient rigidity. For the same reasons, the behavior of rocks beyond their strength limit with an unevenly distributed load and incomplete coverage of the lateral face was not studied. There is also no comparative assessment of the effectiveness of various types of external action on the geomechanical state of the "rock massif - mine workings - support" system. External influences such as heating, cooling, moistening, drying, cementation, etc., are studied most for undisturbed materials. As for for disturbed material, which is rock, external influences change the bond strengths between individual particles. Therefore, the spectrum of their influence is wider. Depending on the type of external influence, the behavior of the rock in the limit state can change from brittle failure to plastic flow and vice versa.

For many years, much attention has been paid to the study of the patterns of rock deformation beyond the strength limit. This is evidenced by a significant number of publications, for example [13–24]. But from a practical point of view, specific behavior of rocks under unequal-component triaxial loading with additional local action of lateral pressure presents the greatest interest, since these are the conditions that are realized in the rock massif near the surface of a mine working with support.

The aim of the work is to determine the influence of physical and technological factors on the stability of the system "rock massif - mine working - support" beyond the strength limit of rocks to substantiate the parameters for controlling the geomechanical state of the specified system at great depths.

2. Methods

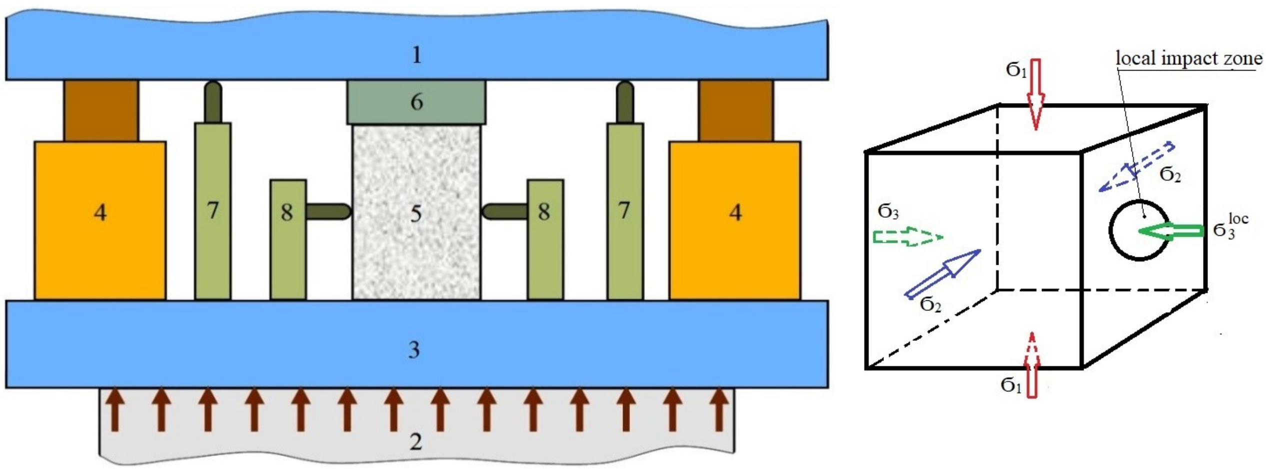
Experimental studies of deformation properties and strength of rocks were carried out in laboratory conditions on a high-rigidity testing machine. The machine is based on a hydraulic press with a force of 5 MN (PSU-500), two hydraulic jacks to counteract compression and a system for synchronous continuous recording (or photo recording) of longitudinal and transverse deformations. The diagram of the testing machine and the directions of application of loads to the specimen are shown in Fig. 1.

The machine is designed in such a way that during the testing process up to 90% of the load falls on the counteracting jacks. This allows the "load-deformation" process to be controlled even beyond the strength limit of the specimen.

The size of the specimens for deformation studies was 40x40x60 mm. The choice of sizes was determined by the technical capabilities of the testing equipment. With a maximum nominal compressive force of the press of 5 MN, the actual capabilities are limited by the hydraulic system and range from 3.5 MN to 4.0 MN. The maximum compression load in the strict test mode was not more than 35 kN [7,8,25].

During the research, the following local impacts were simulated: water saturation, injection, plugging and spraying of cementing solutions, as well as rock bolting.

For the studies of rocks strengthened by solutions, specimens were made from granular sands or crushed rocks with different granulometric compositions. To determine the influence of fracturing and delamination, strengthened rock specimens with a given degree of disturbance were studied. Specimens were made from sandstone, argillite, siltstone and granite in the form of cubes with size 50x50x50 mm, which were destroyed to residual strength.



a) b)

1 – upper thrust plate of the press, 2 – press piston, 3 – lower movable press plate,
4 – anti-compression hydraulic jack, 5 – specimen, 6 – strain gauge force meter,
7, 8 – longitudinal and transverse strain meters

Figure 1 – Schematic illustration of a high-rigidity testing machine (a) and directions of application of loads on the specimen (b)

Strengthening of damaged specimens was carried out in a vacuum unit with solutions of polyester resin. A solution of methyl ethyl ketone peroxide was used as a curing initiator, and a solution of cobalt naphthenate was used as an accelerator.

The experiments were carried out at different levels of local lateral load and the area of its application. The sample surfaces, where stresses σ_1, σ_2, σ_3, acted, were completely covered by the equipment elements. The coverage of the sample surface, where stress σ_3^{loc} acted, was changed stepwise in the range of (25–100)% of the face area. The stress was regulated by changing the characteristics of the compliance node from rigid to compliant, where $\sigma_3^{loc} = \sigma_3$.

In all cases, the load (σ_1) near and beyond the tensile strength limit was applied cyclically, which allowed controlling the process of specimen failure.

To ensure correct comparison of results, specimens for each series of tests were prepared from the same sample.

The parameters of some test conditions are given in Table 1.

Table 1 – Test conditions

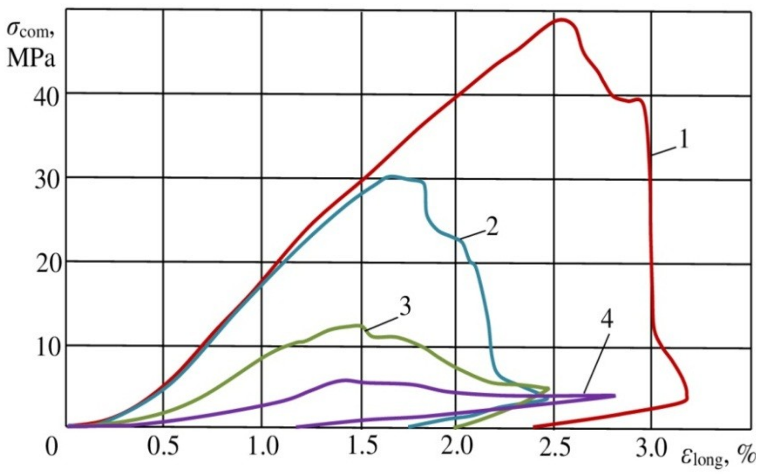
Test element	Parameters				
Specimen number	1	2	3	4	5
Support washer diameter, mm	0	10	15	22	22
Rock bolt lock shear force, kN	0	2.5	2.5	2.5	5.0

Lateral load, MPa	0	32	14.1	6.5	13

3. Results and discussion

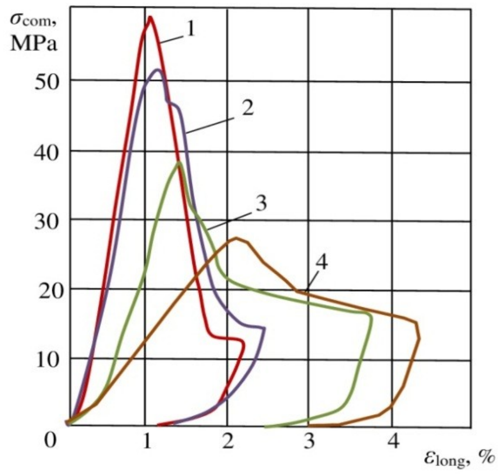
One of the main types of external action that significantly affects the strength properties of rocks is a change in their humidity. It was experimentally established that water saturation affects rocks containing clay minerals and fractured rocks with cracks filled with clay minerals. This confirms the known results of studies [26–28]. This phenomenon is caused by the ability of clay particles to be enveloped by a hydrate shell, which disrupts the bond between them. In addition, the tests showed that changing the humidity (*W*) of rocks changes not only their strength and deformation properties, but also the nature of their behavior beyond the strength limit. As an example, Fig. 2 shows the "stress-longitudinal strain" (σ_{com} - ε_{long}) diagrams of argillite samples with different humidity (where, from here on, $\sigma_{com} = \sigma_1$ is the current value of the vertical component of compressive stress).

In this and all subsequent diagrams, the trend lines near and beyond the tensile strength are smoothed envelopes of cyclic loadings of the specimen.



1 – *W* = 0,5%; 2 – *W* = 2%;
3 – *W* = 5%; 4 – *W* = 8%

Figure 2 – Stress-strain diagrams of argillite specimens with different moisture content *W*



1, 2 – with intact structure;
3, 4 – broken and strengthened samples

Figure 3 – "σ_{com} - ε_{long}" diagrams of undisturbed and strengthened rocks

It is evident from the graphs that at a humidity of more than 2%, clayey rocks are destroyed in the form of intermittent slip or pseudoplastic flow during deformation in limit states. As such rocks dry out, they acquire a tendency to pseudobrittle destruction. In this case, the recession modulus increases tens of times and, accordingly, the value of inelastic limit deformations decreases.

The behavior of the marginal rocks is significantly changed by the injection of various bonding solutions into them. Usually, the greatest attention is paid to strengthening rocks with cement-sand and cement mortars. At the same time, studies show that their bearing capacity does not reach the limit of uniaxial compression strength σ^0_{com} of undisturbed rocks, but depends on the lithological composition of the rocks and the grade of the solution. The effect of strengthening rocks is manifested in the fact that such rocks allow significantly greater extreme deformations without a significant reduction in bearing capacity. Increasing the water-cement ratio reduces the bearing capacity of rocks. When the water-cement ratio is twice increased, the strength of, for example, argillites and siltstones is reduced by almost half. Therefore, in complex mining and geological conditions, disturbed rocks must be strengthened by injecting synthetic resins, which have increased strength and penetrating ability compared to cement mortars. The test results showed that such a solution penetrates cracks with an opening of up to 1 μm. It was also found that after the solution hardens, the specimen is tightly connected microblocks. At the same time, with the introduction of the polymer, the tendency to separation into blocks is reduced and the strength and deformation characteristics of the rocks are significantly restored. An example for siltstone is shown in Fig. 3.

Studies showed that the use of various types of resins (polyester, urea, epoxy) as a binder with an injection of up to 5% of the specimen volume does not change the nature of rock deformation, and the strength does not exceed 0.8 σ^0_{com}. The similarity of the stress-strain diagrams of undisturbed and strengthened rocks indicates that the destruction of strengthened specimens occurs in the rock mass, and not in the structure formed from the hardened resin. This indicates that the recession modulus parameter can be reduced by selecting the composition of the strengthening solution. However, an increase in the limiting region of destruction of strengthened rocks occurs with a simultaneous decrease in their hardness.

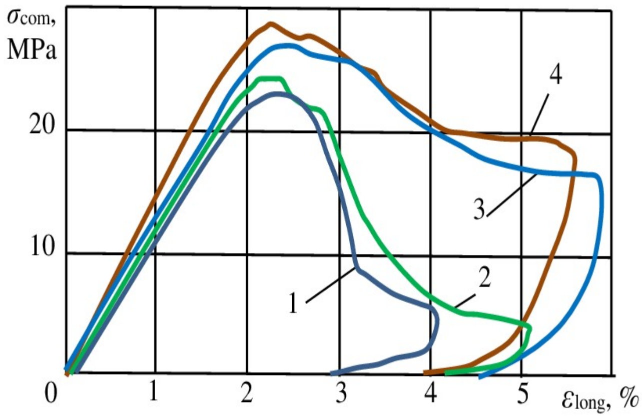
For the implementation of local impacts on the marginal massif, bolting is of particular interest, including in combination with other support. The study examined the influence of rock bolts on the behavior of rocks during their deformation after reaching the strength limit. It was found that different types of boits have different effects on the behavior of rocks beyond the strength limit. Bolts with chemical fixation increase the strength of rocks the most. However, when reaching the limit states, they create conditions for a sharper loss of their bearing capacity.

The greatest influence on the behavior of rocks beyond the strength limit is exerted by compliant anchors with a compliance node located near the surface. As an example, Fig. 4 shows the results of tests on triaxial unequal-component compression of siltstone specimens reinforced with rock bolts. It is evident that the influence of anchors, even with a relatively small bearing capacity, can change the pseudo-brittle nature of destruction to pseudo-plastic flow.

At the same time, for hard rocks, bolting does not change the nature of the destruction process, but also leads to an increase in their residual bearing capacity. In addition, with an increase in the shear force of the flexible rock bolt and the value of the initial thrust, the value of transverse deformations decreases.

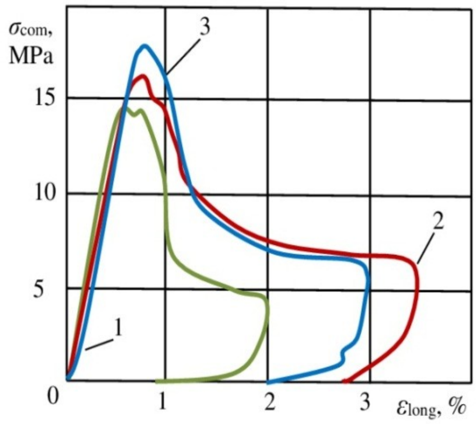
An example of the experimental result on local action by lateral pressure for the conditions $\sigma_1 > \sigma_2 > \sigma_3$ (Fig. 1,b) on argillite specimens is shown in Fig. 5.

Curve 1 was obtained during testing of a specimen without a rock bolt with a free lateral face. Curve 2 – with a bolt operating in a rigid mode. Curve 3 – with a bolt operating in a flexible mode. Under the condition σ_3 = 0, after reaching the strength limit, the destruction process actively develops.



1, 2 – specimens without rock bolt;
3, 4 – specimens with flexible rock bolt

Figure 4 – Nature of the influence of reinforcement on the deformation of siltstone specimens beyond the strength limit



1 – σ_3 = 0; 2 – σ_3 = 16 MPa;
3 – σ_3 = 20 MPa

Figure 5 – "σ_{com} - ε_{long}" diagrams of argillite specsmens at different values of lateral support with coverage ΔS = 4%

The loss of bearing capacity occurs proportionally to the increase in the ultimate deformations. The residual strength hardly changes. Such conditions are not favorable for the preservation of workings. The pressure on a part of the lateral surface ($\Delta S \leq$4%) by installing a rock bolt significantly affects the development of the process of destruction beyond the strength limit. The residual strength changes and the transverse deformation decreases. For a sample with a rigid bolt, it is greater than the deformation with a flexible bolt. This is explained by the fact that a rigid bolt "slips" when reaching its maximum load, reducing lateral pressure, which allowed lateral deformations to occur. A flexible bolt smoothly extends at a given force, maintaining constant lateral support.

Figure 6 shows a series of stress-strain curves for blocking free rock fracture by local actions. The results were obtained for siltstone specimens under various conditions, as given in Table 1.

It follows from the graphs that up to the strength limit, the rock bolt parameters have little effect on the nature and magnitude of rock deformation. However, beyond the strength limit, the destruction process, even at low values of local action, differs from destruction both under free surface conditions and under triaxial unequal-component compression. Beyond the strength limit, the yield zone does not form, but the recession modulus decreases. The process smoothly transits to the residual strength stage. Its value depends on both the minimum component σ_3 and the relative area of load application. At values of σ_3 = 13 MPa, the residual strength reaches 75–80% of σ_{com}, and the nature of the destruction approaches the conditions of triaxial unequal-component compression. Note that the intermediate stress σ_2 does not affect the transverse deformation. It is insignificant or absent. However, the transverse deformation increases due to the influence of stress σ_3. This indicates the formation of a stress field (blocked fracture zone) with a characteristic destruction of the sample, which occurs without loosening the rock.

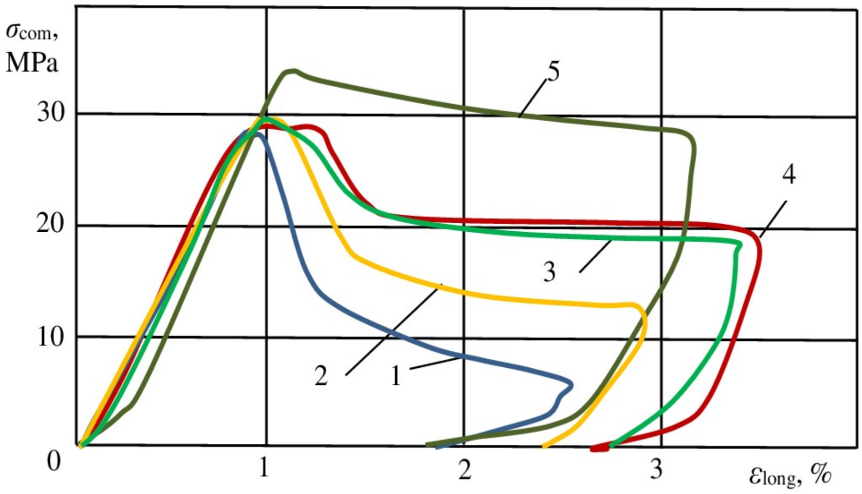


Figure 6 – Diagrams of "σ_{com} - ε_{long}" when blocking free destruction of rocks by local actions in accordance with the conditions of Table 1

A similar nature of rock destruction is also observed in natural conditions in sections of mine workings with frame-bolt combined support [29]. During the observations, two stages of support operation were identified (before and after the failure of rigid rock bolts). Before the failure of the bolts, the displacement of the workings roof was 45% less compared to sections of workings secured with frame support. After the failure of the rigid bolts, a sharp jump in displacements occurred, as a result of which the total displacements decreased by 25–30%.

To develop recommendations for the practical use of local impact effects, a corresponding elastic-plastic problem was solved. The problem was considered in one section. The flat strain state was modeled by an infinite plate with a round hole in its middle part. Around the hole in the near-edge zone, the stress was maintained not lower than the threshold.

The mathematical model, which includes the work of near-edge rock mass, has the following form:

$$\sigma_\theta - A\sigma_r \le \sigma_{com} = ¿\{ \qquad \sigma^0_{com} ¿\} ¿\{\} \frac{for\ r \ge r_L}{for\ 1 \le r \le r_B}, \tag{1}$$

where σ_θ, σ_r - circular and radial stress components; $A = \frac{1+\sin\rho}{1-\sin\rho}$ – parameter depending on the angle of internal friction ρ; σ^0_{com} – current strength value; σ^0_{com} - ultimate strength of rock under uniaxial compression; σ^{res}_{com} – residual bearing capacity of disturbed rock; r_L – radius of the zone of destroyed rocks; r_B – radius of destruction blocking zone; $K > 0$ - coefficient determining the degree of involvement of marginal rocks in the work $0 \le K \le 1$; $\sigma^{¿} = K_i \sigma^0_{com}$ (K_i = 0.2–0.4) – ultimate stress.

The stress function F is related to the components of the stress ratio as follows:

$$\sigma_r = \frac{F}{r}; \quad \sigma_\theta = \frac{dF}{dr}. \tag{2}$$

Having solved the obtained differential equation, we determine the value of the stress component in the impact zone:

$$\sigma^B_r = Pr^{B-1} + \frac{\sigma^{res}_{com}}{B-1}\left(Br^{B-1} - 1\right); \quad \sigma^B_\theta = BP\,r^{B-1} + \frac{\sigma^{res}_{com}}{B-1}\left(Br^{B-1} - 1\right), \tag{3}$$

where $B = A + K\left(\sigma^0_{com} - \sigma^{res}_{com}\right)/\sigma^{¿}$; P – resistance of the support to the rock massif.

To eliminate the sensitivity of rocks to weak impacts, the radius of mining and technical local impacts should be such that outside this zone the value of the minimum stress component is greater than the threshold value:

$$\sigma^B_r \ge \sigma^{¿} \ \text{at} \ r \ge r_B \tag{4}$$

From (4) from (3) we obtain the relationship for the minimum size of the zone of strengthening action that blocks the free destruction of the marginal rocks:

$$r_B = \left\{ \frac{\dfrac{\sigma^{res}_{com} + \sigma^{¿}}{B-1}}{P + \dfrac{\sigma^{res}_{com}}{B-1}} \right\}^{\frac{1}{B-1}} \tag{5}$$

From the first relation of condition (1) using (2) and taking into account the limit condition, we obtain the distribution of stress components outside the blocked fracture zone (pseudoplastic flow zone):

$$\sigma^H_r = \frac{\sigma^{res}_{com}}{A-1} + \left[\frac{\sigma^0_{com}}{A-1} - \frac{\sigma^{res}_{com}}{B-1}\right]\frac{r^{A-1}}{r_B^{A-1}} + \left[P + \frac{\sigma^{res}_{com}}{B-1}\right]\gamma_B^{B-A} R^{A-1};$$

$$\sigma^H_\theta = -\frac{\sigma^0_{com}}{A-1} + A\left[\frac{\sigma^0_{com}}{A-1} - \frac{\sigma^{res}_{com}}{B-1}\right]\frac{\gamma^{A-1}}{\gamma_B^{A-1}} + A\left[P + \frac{\sigma^{res}_{com}}{B-1}\right]\gamma_B^{B-A}\gamma^{A-1}. \tag{6}$$

Since at the boundary of the zone of elastic and inelastic deformations $r = r_L^B$ the condition $\sigma_r^H = P_L$ is satisfied, then after transformations we obtain:

$$\gamma_L^B = \gamma_B \left\{ \frac{P_L + \sigma_{com}^0 / A - 1}{\sigma^{¿} + \sigma_{com}^0 / A - 1} \right\}^{\frac{1}{A-1}}, \tag{7}$$

where $P_L = \frac{2\rho gH - \sigma_{com}^0}{A+1}$ the value of the radial component of stresses at the boundary of the zone of inelastic and elastic deformations; H – depth of mine workings; ρ – bulk density of rocks; g – acceleration of gravity.

By comparing the value of the radii of inelastic deformation zones during free and blocked destruction processes, we obtain a ratio for determining the coefficient of reduction of the zone of disturbed rocks:

$$K_{Zn} - \frac{r_L^B}{r_L^C} = \frac{\left\{ \left[P_L + \frac{\sigma_{com}^{res}}{B-1} \right] / \left[P + \frac{\sigma_{com}^{res}}{B-1} \right] \right\}^{\frac{1}{B-1}}}{\left\{ \left[P_L + \frac{\sigma_{com}^{res}}{A-1} \right] / \left[P + \frac{\sigma_{com}^{res}}{A-1} \right] \right\}^{\frac{1}{A-1}}}. \tag{8}$$

The rock stability category, which is the basis for selecting the types of supports and their parameters, is determined by the displacement of the mine workings contour. The expected displacement of the workings contour during the formation of a zone of inelastic deformations is equal to:

$$U_r = \frac{\rho gH - P_L}{2G} \gamma_L^{A+1}, \tag{9}$$

where G – shear modulus of rocks.

Taking into account (8) and (9), the displacement of the rock contour during blocked failure, expressed through the displacement during free failure, is:

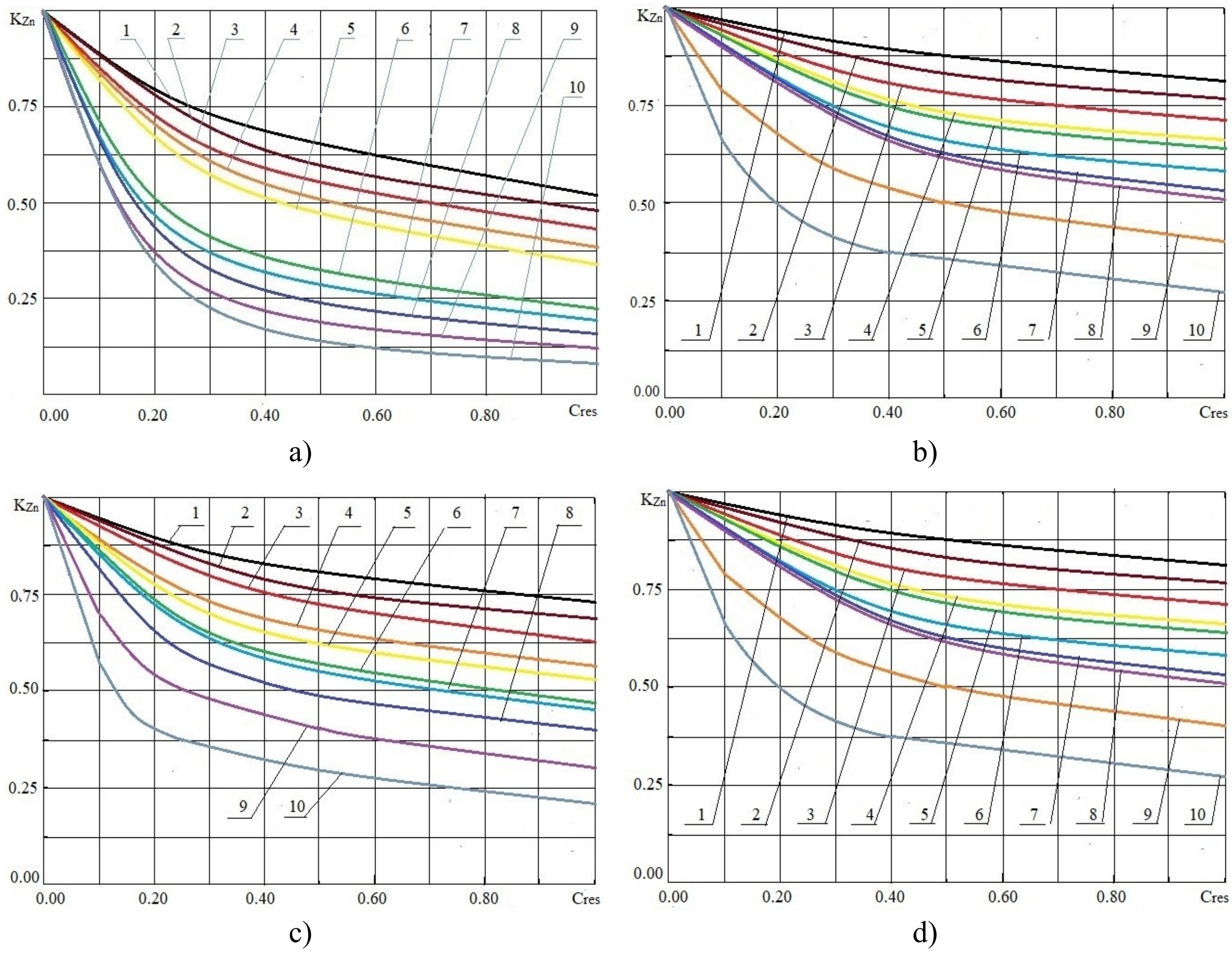
$$U_r^B = K_{res}^{A+1} U_r^C. \tag{10}$$

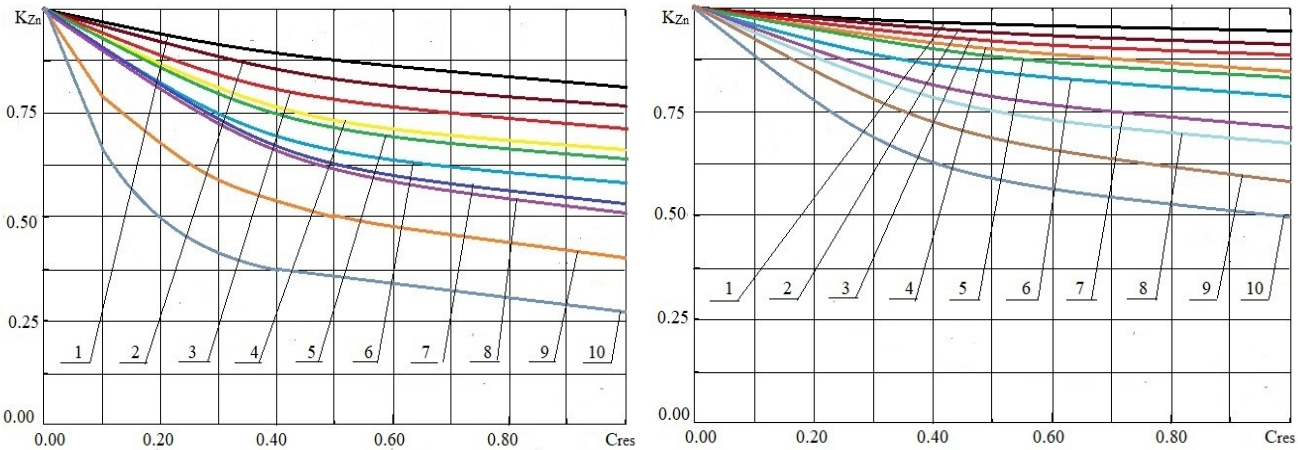
Dependence (10) allows one to select the magnitude of local impact, the implementation of which will lead to the transfer of rocks in the marginal zone to a higher stability category.

To simplify engineering calculations of expected displacements of the rock contour, Fig. 7 shows graphs of the dependence of the coefficient of reduction of the zone of disturbed rocks K_{Zn} on the coefficient of blocking of the massif $c_{res} = \sigma_{com}^{res} / \sigma_{com}^{0}$.

Curves 1, 2, 3, 4, 5 correspond to the value A = 2; curves 6, 7, 8, 9, 10 – A = 3. Curves 1 and 6, 2 and 7, 3 and 8, 4 and 9, 5 and 10 – blocking coefficient c_{res}, equal to 0.05; 0.10; 0.15; 0.20; 0.25, respectively.

The blocking coefficient depends on the ratio $\sigma_{com}^{res} / \sigma_{com}^{0}$. That is, it is determined by the change in the value of the residual bearing capacity of rocks, which is achieved by the adopted mining and technological processes and can take values from 0 to 1. The conditions of the mine working construction are taken into account by the coefficient $c = \dfrac{\sigma_{com}^{0}}{\gamma H}$ and the parameter A.





e) f)

a – c = 0.25; b – c = 0.5; c – c = 0.75; d – c = 1.0; e – c = 1.25; f – c = 1.5

Figure 7 – Diagrams of the dependence of the coefficient of the zone of disturbed rocks on the blocking coefficient for different values of the parameter $c=\frac{\sigma^0_{com}}{\gamma H}$

For example, mine workings is laid under the following conditions: $c = 0.75$, $A = 2$, $c_{res} = 0.15$. Let's use our calculations. From formula (10) we get that the expected displacement of the rock contour during free destruction is 350 mm. According to the accepted classification [30], this corresponds to category III of rock stability (very unstable).

We apply a local action, for example, bolting rocks with coverage of part of the workings surface. For the given conditions this will allow increasing c_{res} up to 0.6. Using curve 3, Fig. 7, we find that $K_{Zn} = 0.47$, and using formula (10) – that U^B_r = 76 mm. This already corresponds to the second category of stability (medium-resistant), which significantly reduces the requirements for the choice of technology and the parameters of the workings support.

4. Conclusions

For mine workings constructed at great depths or in conditions of fractured rocks, the reliability and operability of the support are of primary importance. However, practical experience shows that the effectiveness of traditional support schemes is insufficient. These conditions are best met by the support technology that actively involves the marginal massif in the work, blocking the process of crack formation and loosening of rocks.

Beyond the strength limit of rocks, even small local impacts can change the geomechanical state of the rock mass from pseudo-brittle failure to pseudo-plastic flow. During deformation beyond the strength limit, the most significant changes occur with the residual bearing capacity of rocks. The value of this parameter depends on the minimum stress component and the relative area of application of local resistance. This force prevents lateral displacement of the support and loss of its bearing capacity. According to the results of experimental studies a constant passive resistance (lateral support) with a relative area of application of 4% allows increasing the residual strength by 40%, and with 24% by 80% of the strength limit.

Rock destruction control can be achieved by spatial reinforcement of the rock mass, for example, by combined frame-bolt support, injection of bonding solutions, spray concreting, plugging, etc.

The maximum rock displacement occurs on the workings contour. Therefore, the rock mass reinforcement must be performed by flexible support with constant or smoothly changing resistance. In this case, it is advisable to locate the support flexibility node as close as possible to the workings contour.

Since local impacts are effective even with partial coverage of the mine workings surface, it is optimal to use the effect of the combined operation of rock bolt and frame supports or bolts with grabs.

Taking into account the results of analytical studies for practical application, a simplified method for calculating local impacts is presented for the purpose of blocking disturbed marginal rocks.

Conflict of interest

Authors state no conflict of interest.

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ДОСЛІДЖЕННЯ СТІЙКОСТІ ГІРСЬКИХ ПОРІД ЗА МЕЖЕЮ МІЦНОСТІ ПРИ ЗОВНІШНІХ ЛОКАЛЬНИХ ВПЛИВАХ

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Анотація. Предмет досліджень – деформаційні процеси в гірських породах за межею міцності при цільових зовнішніх локальних впливах. Мета роботи – визначити вплив окремих фізичних і технологічних факторів на стійкість системи «породний масив – гірнича виробка – кріплення» за межею міцності порід для обґрунтування параметрів керуванням геомеханічним станом вказаної системи на великих глибинах. Методи досліджень - лабораторні експериментальні дослідження, математичне моделювання, аналіз і узагальнення результатів.

Комплекс експериментальних досліджень виконано на випробувальному обладнанні підвищеної жорсткості. Режими досліджень: одно-, двох- та трьохвісьовий стиск до і за межею міцності з різними варіантами зовнішньої локальної дії. Об'єкт досліджень – зразки гірських порід осадового і скельного типів. Підтверджено, що за межею міцності навіть локальні впливи з малою енергією здатні змінити характер деформування порід від псевдокрихкої руйнації до псевдопластичної течії. При позамежному деформуванні найбільш істотні зміни відбуваються з залишковою несучою здатністю порід. Її значення залежить від мінімальної компоненти напружень і відносної площі застосування локальної дії. Для розробки рекомендацій з практичного використання ефектів локального впливу вирішено пружно-пластичну задачу, що враховує "включення в роботу" приконтурного масиву гірських порід. Показано, що для гірничих виробок на великих глибинах або в умовах тріщинуватих порід першочергове значення набувають активне залучення до роботи приконтурного масиву і блокування процесу розпушування порід. Показано, що керування руйнуванням породного масиву можна досягти його просторовим армуванням системою анкерів, ін'єкцією скріплюючих розчинів, набризкбетонуванням і тампонажем порожнин за кріпленням. Для запобігання втрати стійкості гірничої виробки локальний вплив має бути безперервним. Тому армування необхідно виконувати піддатливими анкерами з постійним або плавно змінним опором. Оскільки максимальне зміщення відбувається на контурі, вузол піддатливості анкера доцільно розташовувати на поверхні виробки.

Ключові слова: гірські породи, деформування за межею міцності, стійкість гірничих виробок, кріплення, цільовий зовнішній локальний вплив, види впливу, рекомендації.