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EXPRESS METHOD FOR DETERMINING PARAMETERS OF HEAVING OF WATER-SATURATED ROCKS

The task of predicting rock heaving, quantifying its magnitude, and delineating the spatial extent of its development are critically important in the design of underground excavations, the planning of maintenance and rehabilitation measures, and the selection of effective methods to ensure the long-term stability of underground structures and utilities.

Purpose. To theoretically substantiate the mechanism by which excess pore fluid pressure in rocks influences heaving processes in underground workings.

Methodology. The study is based on a theoretical analysis of geomechanically processes developing in the rock mass surrounding horizontal underground excavations. Analytical and numerical mathematical methods were employed to describe these processes, and the resulting theoretical solutions were analyzed, generalized, and systematized.

Findings. Simple analytical relationships have been derived that enable determination of the boundaries of the basal zone in which heaving of water-saturated rocks occurs, as well as calculation of a stability coefficient for this zone. The stability coefficient is proposed as the ratio of the projection of forces restraining the rock mass from uplift to the vertical projection of forces initiating rock uplift. The obtained analytical expressions were calibrated for the conditions of the Donbas region.

Originality. It is demonstrated for the first time that, under otherwise identical conditions, an increase in pore fluid pressure leads to a reduction in the maximum depth of the basal heaving zone. It is also shown that increasing pore pressure simultaneously decreases both the stability coefficient and the maximum heaving depth.

Practical value. The results provide a mathematical basis for predicting the stability of horizontal excavations susceptible to basal heaving of water-saturated rocks, taking into account excavation depth, geometric parameters, unit weight, strength characteristics of the rock mass, and pore pressure. The proposed approach also allows determination of the boundaries of the heaving zone under specific mining and geological conditions. In addition, the theoretical conclusions are applicable to solving practical engineering problems of a technological nature, particularly in soil and rock improvement by silicification, cementation, and high-pressure grouting. This makes it possible to determine the maximum allowable injection pressure at which rock uplift or failure does not occur during the strengthening process.

Keywords: *heaving of water-saturated rocks, Coulomb strength criterion, Shashenko strength criterion*

Introduction. The construction of new underground urban engineering and technological utilities, structures, and transportation corridors, as well as underground storage facilities, industrial enterprises, civil defense shelters, and other subsurface structures, has been intensified by ongoing military activities on the territory of Ukraine and is inherently associated with the excavation of underground workings.

In addition, during urban development, the erection of new buildings, embankments, and other surface structures above existing underground excavations may lead to a significant increase in stresses acting on the rock mass.

All of the above is fully applicable to mine construction as well.

It should also be noted that, in recent years, underground excavations have increasingly been driven in weak rocks and soils, including water-saturated formations.

Heaving of rocks is a widespread phenomenon observed during the construction of underground excavations [1, 2]. Manifestations of rock heaving may occur in vertical, inclined, and horizontal underground excavations [3, 4].

This phenomenon complicates both the construction and operation of underground excavations and, as a consequence, leads to an increase in excavation time and cost, as well as in maintenance duration and expenses during their service life [5, 6].

The problem of predicting whether heaving of the rock mass will occur in a specific case, as well as determining the heaving magnitude and delineating the boundaries of the zone in which it develops, is of critical importance in the design of underground excavations, the estimation of anticipated volumes of repair work, and the selection of effective methods for ensuring the long-term stability of underground structures and utilities located within underground workings [7, 8].

Literature review. From the experience of construction and operation of underground excavations, it is known that under certain mining and geological conditions heaving of the rock mass occurs from the floor of an underground excavation.

A large volume of studies devoted to the rock heaving process has been carried out by M. M. Protod'yakonov,

P. M. Tsymbarevych, M. P. Pokrovskyi, V. V. Orlov, D. S. Rostovtsev, A. Labasse, A. V. Hurdus, I. L. Cherniak, O. M. Shashenko, and representatives of his scientific school, including O. V. Solodiankin, S. M. Hapieiev, and O. O. Sdvizhkova, as well as by other researchers [1, 5].

In particular, these and other researchers have established that rock heaving is influenced by many factors, including the following [8, 9]:

- the strength properties of the rock mass in which the excavation is constructed [9, 10];
- the load acting on the rock mass adjacent to the excavation (and, consequently, the excavation depth) and its deformations [11, 12];
- the spatial orientation of the excavation (vertical, horizontal, inclined, etc.) [4];
- the geometry of the excavation [11];
- the spatial position of the rock layers in which the excavation is formed [13];
- the spatial arrangement of structural elements of the rock mass in which the excavation is driven [14];
- the spatial arrangement of textural elements of the rock mass in which the excavation is formed [15];
- the type of support used for the roof and the floor of the underground excavation [16];
- the surface relief of the area in which the underground excavation is located [17].

The above-mentioned factors should be taken into account during engineering investigations, as well as in the calculation of deformations, strength, stability, and operation not only of underground mine workings, but also of such underground structures [18, 19] as:

- metro tunnels [20, 21];
- highway tunnels [22].

Below, the most well-known methods for determining the parameters of rock heaving are considered.

At present, the following physical models of a continuous medium are used to predict rock heaving processes:

1) models based on various strength failure criteria (as a rule, the Mohr–Coulomb or Hoek–Brown criterion) and elements of the theory of limit equilibrium (for example, Melentievich, S., Berisavljevic, Z., Berisavljevic, D., & Marañon, K. O. (2024), as well as [23, 24], and others);

2) rheological models (for example, Frenelus, W., Peng, H., & Zhang, J. (2022), and others);

3) elasto-plastic models (for example, Cui, L., Yang, W., Sheng, Q., et al. (2024));

4) based on the listed models, constitutive equations are formulated, which must be supplemented by equilibrium equations, as well as boundary and initial conditions (the latter only when the dynamic component and/or rheological behavior of the process is taken into account).

A representative result of determining the basal heaving zone of the floor of an underground excavation using a continuous medium model was obtained by the authors of [25].

This study is of interest from the following perspectives:

- it is typical in terms of the application of numerical methods and continuous medium models;
- it considers the influence of water on the heaving of the floor of underground excavations, while employing the Hoek–Brown strength criterion.

It should be emphasized that the mechanism of groundwater influence on floor heaving considered in that study fundamentally differs from the mechanism analyzed in the present article. In the former case, water infiltration into the floor occurs under pore pressures close to zero, which leads to degradation of the properties of clayey soils. In contrast, the present study examines the effect of excess pore fluid pressure in water saturating the rock pores, which may significantly differ from zero and can induce rock failure.

A drawback of continuous medium models is the difficulty of generalizing and systematically analyzing the results obtained using them. This is due to the fact that, in such models, the calculation results are tied to specific values of material constants. In addition, there is a limitation in applying this approach to optimization problems, which is associated with the large computational effort required at each optimization step, especially when iterative procedures are used to predict heaving processes.

Empirical methods occupy a special place in predicting heaving processes. These methods are based on simple analytical relationships or nomograms (for example, Mark, C. (2016), and Sakhno, I., Liashok, I., Sakhno, S., & Isaienkov, O. (2022)).

The advantage of this approach lies in its simplicity and low computational cost, whereas its limitation is the dependence on the specific regions from which the statistical data were obtained.

In particular, for the geological and mining conditions of the Donbas region, the following simple formula was obtained to assess the susceptibility of rocks to heaving

$$2 \cdot a > \frac{1.22 R_c}{(\gamma \cdot h)}, \tag{1}$$

where a is the half-width of the excavation span; R_c is the uniaxial compressive strength of the rock mass; γ is the weighted-average unit weight of the rock mass over the depth interval 0...h; h is the depth of the excavation floor below the ground surface. This formula is presented, in particular, in the works by O. Shashenko.

Despite the diversity of existing studies and the variety of methodological approaches employed, a common limitation can be identified. In particular, the influence of excess pore fluid pressure on the heaving of the rock mass has not been sufficiently considered in previous research.

In other words, the works cited above entirely omit the effect of pore fluid pressure on the heaving process of the floor of underground excavations, although the presence of pore pressure in the rock mass is a widespread phenomenon [12, 23].

Excess pore pressure may arise in the following situations:

1. In the presence of a water column above the point of interest at the excavation floor (hydrostatic pressure). The water column can induce a phenomenon related to rock heaving – hydraulic uplift of the rock – which must be accounted for in the design of hydraulic structures.
2. Due to fluid movement within the pores of the rock mass, resulting in hydrodynamic pressures, for example, during pumping or injection of water, or when excavating workings in water-bearing formations.
3. As a result of anthropogenic factors, such as explosions in water-saturated rock, injection of solutions for rock silicification, cementation, bituminization, etc.

These factors may act individually or in combination.

In this study, the approach presented in [24] was applied to systematically investigate the influence of excess pore fluid pressure on rock heaving, with a focus on the following aspects:

1) the occurrence of heaving in water-saturated rock under conditions of excess pore fluid pressure;

2) differences in the heaving behavior of water-saturated and unsaturated rocks under the influence of pore fluid pressure;

3) the spatial extent and shape of the heaving zone in water-saturated rock;

4) the effect of pore fluid pressure on the strength and stability of the rock mass during heaving.

Purpose. The aim of this study is to theoretically determine how excess pore fluid pressure affects the heaving of rock masses in underground excavations.

To achieve this aim, the following research objectives were formulated:

1. To establish whether, and how, the heaving process of water-saturated rock depends on the magnitude of pore fluid pressure.

2. To determine the zones within an excavation driven in water-saturated rock where heaving occurs, depending on the magnitude of excess pore fluid pressure.

3. To identify and quantify the differences between the heaving processes of saturated and unsaturated rocks at the excavation floor.

Methods. The answers to these questions can be obtained using the arching effect hypothesis, which was proposed by M. M. Protod'yakonov and presented in [24].

The study was carried out in the following sequence.

At the first stage, an analysis was performed of the solution presented in [24], which was obtained for a foundation with zero pore pressure. It was concluded that, in the first approximation, the foundation area in which heaving occurs can be represented as a pointed arch, with semi-arches subjected to forces that both restrain and displace the rock mass.

To determine the restraining forces, the Mohr-Coulomb strength criterion was used, which describes the strength of the rock mass under the influence of excess pore fluid pressure [23, 26].

Next, a functional was constructed, numerically equal to the ratio of the forces restraining the rock to the forces displacing it (i.e., the stability coefficient of the rock prone to heaving).

Subsequently, the value of the arch rise was determined at which the stability coefficient reaches its minimum. This allowed the final solution of the problem to be obtained.

Finally, the solution obtained was validated using asymptotic estimates.

The research task was formulated as follows:

1. At the design depth, a long horizontal excavation is driven in a rock mass with known physico-mechanical properties.

2. The mining pressure at the upper part of the excavation and the lateral pressure are supported by a compliant support, which is capable of vertical movement. Horizontal displacement of the support and its elements is excluded. This is achieved either by anchoring the support into the floor rock or by installing horizontal struts (Fig. 1).

3. No support is provided at the excavation floor (Fig. 1).

4. A pre-determined equation describing the shape of the heaving zone beneath the excavation is known; however, the parameters of this equation are unknown and must be determined during the solution of the problem.

5. The rock failure mechanism is shear, and therefore its failure is governed by the Mohr–Coulomb strength criterion.

6. The horizontal and vertical stresses at the design depth are known (Fig. 1).

7. The pore fluid pressure, P in the rock mass is known.

8. It is necessary to determine:

- whether heaving of the rock mass occurs under these specific conditions;
- the maximum depth below the excavation floor at which heaving begins;
- how excess pore fluid pressure affects the heaving of the rock mass.

The following is given

$$\left.\begin{aligned}
&\mathrm{tg}(\alpha)=\frac{dY(x)}{dx}=\frac{f}{a}\\
&\alpha=\mathrm{arctg}\left[\frac{dY(x)}{dx}\right]=\mathrm{arctg}\left(\frac{f}{a}\right)\\
&dy=dx\cdot\mathrm{tg}(\alpha)=dx\cdot\frac{f}{\alpha}\\
&ds=\sqrt{d^2x+d^2y}=\frac{dx}{\alpha}\cdot\sqrt{\alpha^2+f^2}
\end{aligned}\right\}.$$

Results. As a result of solving the problem, it is necessary to determine whether heaving of the rock occurs in this specific case and to identify the boundary separating the failed and intact rock in the excavation roof.

In order to define the zone where rock heaving occurs, the following assumptions are made:

1. The strength of the rock is governed by the Mohr-Coulomb criterion [23]

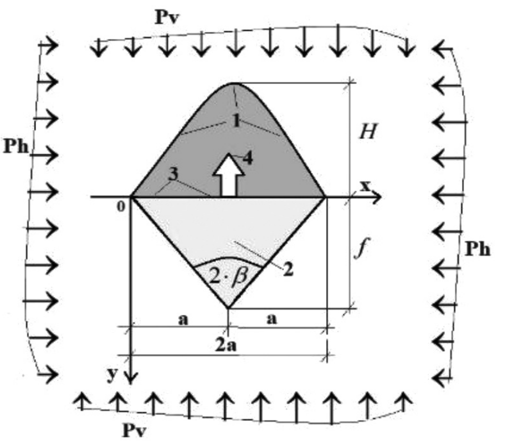


Fig. 1. Prior to determining the failure zone of the rock mass at the floor of a horizontal excavation:

1 – restraining structure; 2 – heaving zone of the rock; 3 – unloaded surface of the excavation floor; 4 – direction of heaving deformations (P_v *– vertical stress;* P_h *– horizontal stress; f – rise of the support arch; a – half-width of the excavation; x, y – coordinates)*

$$\tau = (\sigma - P)\cdot \mathrm{tg}(\varphi) + c =$$
$$= \frac{(\sigma - P)}{2}\cdot\frac{R_c - R_p}{\sqrt{R_c\cdot R_p}} + \frac{1}{2}\sqrt{R_c\cdot R_p} = \qquad (2)$$
$$= \frac{(R_c - R_p)\cdot(\sigma - P) + R_c\cdot R_p}{2\cdot\sqrt{R_c\cdot R_p}}.$$

Here τ is shear stress; σ is normal stress; for R_c see explanation for (1); R_p is the same, for tensile stress; P is pore fluid pressure.

2. In the first approximation, the equation for the boundary of the heaving zone, using Heaviside functions U_x, can be represented as a pointed arch [27]

$$Y(x) = \frac{f}{a}\cdot x\cdot\left[1 - U(a - x)\right] +$$
$$+ f\cdot\left(2 - \frac{x}{a}\right)\cdot\left[U(a-x)U(2\cdot a - x)\right].$$

Next, let us consider the forces acting at point M – the restraining and shear forces (Fig. 3).

Since the cause of rock failure is shear, the forces restraining and displacing the rock are directed tangentially to the curve $M(x)$at point M (Fig. 2). It should also be noted that (Fig. 2)

$$\beta = \frac{\pi}{2} - \alpha.$$

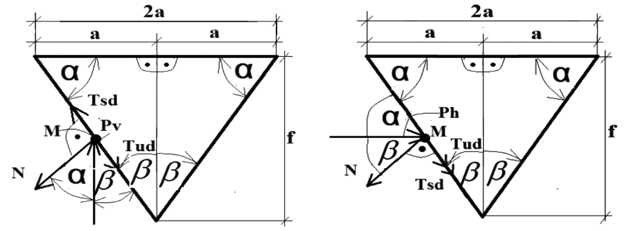
Considering the symmetry, only the left side of the arch is further analyzed. First, the differentials of the forces acting at point $M$are determined. To do this, infinitesimal increments of the abscissa dx, ordinate dy, and arc length $ds$are considered (Figs. 2 and 3).

Next, the differentials of the shear and restraining forces are determined. The differential of the shear force, dT_{sd}, is equal to

$$dT_{sd} = P_v\cdot\cos(\beta)\cdot dx = P_v\cdot\sin(\alpha)\cdot dx = P_v\cdot\frac{f}{\sqrt{\alpha^2+\beta^2}}\cdot dx.$$

The differential of the normal force, dN, is equal to

$$dN = P_v\cdot dx\cdot\cos(\alpha) = P_v\cdot\frac{a}{\sqrt{\alpha^2+f^2}}\cdot dx. \qquad (3)$$



a *b*

Fig. 2. Determination of shear and restraining forces:

a – diagram for determining the restraining and shear forces due to vertical load P_v; b – the same, due to horizontal load; α – inclination angle of the semi-arch relative to the abscissa axis; $\beta = \pi/2 - \alpha$; $\alpha = \pi/2 - \beta$; P_v and P_h – vertical and horizontal components of mining pressure acting per meter of excavation length (measured in kN/m^2 · m); T_{sd} and T_{ud} – loads that, respectively, displace and restrain the rock at a point on the surface of the semi-arch

Substituting (3) into the Mohr–Coulomb strength criterion (2), we obtain

$$dT_{ud} = (dN - dP)\cdot\mathrm{tg}(\varphi) + c\cdot ds =$$
$$= \left[P_v\cdot\cos(\alpha) - P\right]\cdot\mathrm{tg}(\varphi)\cdot dx + \frac{c\cdot dx}{\cos(\alpha)}.$$

Here, $dP = P\cdot dx$ is the differential of the force due to pore fluid pressure (measured in t/m^2).

Next, the projections of the restraining and shear forces onto the vertical axis are determined.

The projection of the shear forces onto the vertical axis, $T_{sd,y}$, is equal to

$$T_{sd,y} = 2\cdot\int_0^\alpha dT_{sd}\cdot\cos(\beta) =$$
$$= 2\cdot\int_0^\alpha P_v\cdot f\Big/\sqrt{\alpha^2\cdot f^2}\cdot dx\cdot\cos(\beta) = \qquad (4)$$
$$= 2\cdot P_v\cdot\alpha\cdot f^2\big/(f^2+\alpha^2).$$

The projection of the restraining forces onto the vertical axis, $T_{ud,y}$, is equal to

$$T_{ud,y} = 2\cdot\int_0^\alpha dT_{ud}\cdot\cos(\beta) + Q_{op} =$$
$$= -2\frac{\left[\sqrt{\alpha^2+f^2}\cdot\mathrm{tg}(\varphi)\cdot\alpha^2 P_v - (\alpha^2+f^2)\cdot c\right]}{(\alpha^2+f^2)}\times \qquad (5)$$
$$\times f + \gamma\cdot\alpha\cdot f.$$

Here, $Q_{op} = \gamma\cdot\alpha\cdot f$ is the weight of the rock contained within the heaving zone, and γ is its unit weight.

To simplify (5) and provide a certain safety margin, we set $Q_{op} = 0$. In this case, a slight reduction in the stability coefficient is obtained.

Equations (4 and 5) allow the strength of the rock in the heaving zone to be assessed by introducing a stability coefficient in the form of the vertical-axis projection of the ratio of restraining forces to shear forces. The following is obtained

$$k_u = \frac{T_{ud,y}}{T_{sd,y}} =$$
$$= \frac{-\sqrt{\alpha^2+f^2}\cdot P\cdot\alpha\cdot\mathrm{tg}(\varphi) + P_v\cdot a^2\cdot\mathrm{tg}(\varphi) + c(\alpha^2+f^2)}{f\cdot P_v\cdot a}. \qquad (6)$$

This approach allows a straightforward determination of the state of the foundation zone beneath the ex-

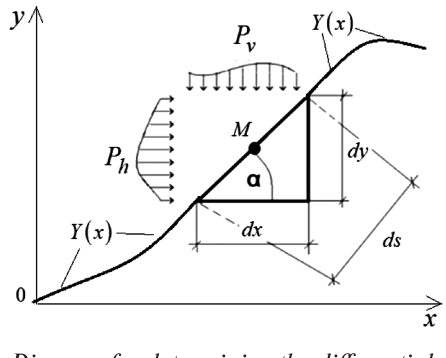


Fig. 3. Diagram for determining the differentials of shear and restraining forces

cavation (stable, neutral, or unstable). Specifically, if the rock in the heaving zone is in a stable state, the $k_u = \frac{T_{ud,y}}{T_{sd,y}} > 1$. If the rock in the heaving zone is in a neutral state, then $k_u = \frac{T_{ud,y}}{T_{sd,y}} = 1$. Finally, if the rock in the heaving zone is in an unstable state, then $k_u = \frac{T_{ud,y}}{T_{sd,y}} < 1$.

From equation (6), it follows that by assigning different values to the arch rise f, different values of the stability coefficient k_u can be obtained. However, of interest are those values of the arch rise f for which the stability coefficient reaches a minimum. From a mathematical point of view, the above statements correspond to the formulation of a problem for finding the minimum of a certain functional.

To find the extremum, the first partial derivative of functional (6) with respect to the variable f is taken and set equal to zero. The following is obtained

$$\frac{\partial k_u}{\partial f} = -\frac{\left[-P_v \cdot a^2 \cdot \mathrm{tg}(\varphi) - c \cdot a^2 + c \cdot f^2\right] \cdot \sqrt{\alpha^2 + f^2} + P \cdot a^3 \, \mathrm{tg}(\varphi)}{f^2 \sqrt{\alpha^2 + f^2} P_v \cdot \alpha} = 0,$$

from which

$$\left.\begin{aligned}
&f_{sat} = -\sqrt{\alpha^2 + z_0^2} \\
&K_{st,sat} = k_u = \frac{-\sqrt{\alpha^2 + f^2} \cdot P \cdot \alpha \cdot \mathrm{tg}(\varphi) + P_v \cdot a^2 \cdot \mathrm{tg}(\varphi) + c(\alpha^2 + f^2)}{f \cdot P_v \cdot a} \\
&z_0 = \left(\frac{z_1}{6 \cdot c} + 2\frac{\mathrm{tg}(\varphi) \cdot P_v + 2 \cdot c}{z_1}\right) \cdot a \\
&z_1 = \left(\left(-108 \cdot P \cdot \mathrm{tg}(\varphi) + 12\sqrt{-3\frac{z_2 + z_3}{c}}\right) \cdot c^2\right)^{\left(\frac{1}{3}\right)} \\
&z_2 = 4 \cdot \mathrm{tg}(\varphi)^3 \cdot c \cdot P_v^2 - 27 \cdot \mathrm{tg}(\varphi)^2 \cdot P^2 \cdot c \\
&z_3 = 24 \cdot \mathrm{tg}(\varphi)^2 \cdot c \cdot P_v^2 + 48 \cdot \mathrm{tg}(\varphi) \cdot c^2 \cdot P_v + 32 \cdot c^3
\end{aligned}\right\}. \tag{7}$$

In equation (7), $K_{st,sat}$ and f_{sat} are, respectively, the minimum value of the stability coefficient for water-saturated rock and the corresponding maximum heaving depth.

Next, the critical values of the maximum depth at which heaving occurs and the stability coefficient for non-water-saturated rock are determined. From this, we obtain

$$\left.\begin{aligned}
f_{dry} &= \lim_{P \to 0}(f_{sat}) = \frac{\alpha \cdot \sqrt{\mathrm{tg}(\varphi) \cdot P_v \cdot c + c^2}}{c} \\
K_{st,dry} &= \lim_{P \to 0}(K_{st,sat}) = \frac{2 \cdot \sqrt{\mathrm{tg}(\varphi) \cdot P_v \cdot c + c^2}}{P_v}
\end{aligned}\right\}. \tag{8}$$

Here, $K_{st,dry}$ and f_{dry} are, respectively, the minimum value of the stability coefficient for non—water-saturated rock, k_u, and the corresponding maximum heaving depth, f.

It is worth noting that results fully coinciding with (8) for a non-water-saturated foundation were previously obtained by the authors of [25] using a completely different physical model of the foundation.

In rock mechanics, it is customary to operate with strength characteristics such as the uniaxial compressive strength R_c and the uniaxial tensile strength R_p. Therefore, it is appropriate to consider the results obtained above in terms of these strength characteristics R_c and R_p.

To transition to these strength characteristics, we use relationships (9), which relate the compressive and tensile strength of the soil (rock) to its internal friction angle and specific cohesion (i.e., the material constants of the Mohr-Coulomb strength criterion).

From this, we obtain

$$\left.\varphi = \arcsin\left(\frac{R_c - R_p}{R_c + R_p}\right); \quad c = \frac{1}{2} \cdot \sqrt{R_c \cdot R_p}.\right\}. \tag{9}$$

Assuming $P = 0$ in (7), and taking into account (9) for a non—water-saturated foundation, we find:

1) for the maximum depth of the foundation where heaving occurs

$$f_{dry} = \alpha \cdot \sqrt{1 + P_v \cdot \left(\frac{1}{R_p} - \frac{1}{R_c}\right)}; \tag{10}$$

2) for the minimum value of the stability coefficient

$$K_{st,dry} = \frac{\sqrt{R_c \cdot R_p + P_v \cdot (R_c - R_p)}}{P_v}. \tag{11}$$

It is worth noting that the results fully coinciding with (10 and 11) for non-water-saturated rock were previously obtained by the authors of [24] using a completely different physical model.

Next, we compare the dependencies of the maximum depth of the heaving zone and the rock stability coefficient on the pore fluid pressure. To illustrate the obtained results, we consider the problem of determining the dimensions of the heaving zone in the foundation of horizontal excavations with a span of 6 meters, located at different depths h equal to: 10, 100, 200, 250, and 500 meters.

The following input data are used: the unit weight of the rock $\gamma = 20$ kN/m^3; compressive strength of the rock

R_c = 5,000 kPa; tensile strength R_p = 900 kPa. These strength values were used by the authors of [24] and are characteristic of subsiding rocks such as siltstone, argillite, marl, chalk, and weak limestone in a fully water-saturated state.

During the calculations, the pressure at the design depth was determined using the formula

$$P_v = \gamma \cdot h. \qquad (12)$$

After that, using formulas (7 and 9), the maximum depth at which heaving occurs and the stability coefficient of the rock involved in the heaving process were determined.

The calculation results obtained using formulas (7 and 9) are summarized in Table 1.

Next, to compare our results (first column in Table 1) with the data of other authors, formula (1) was rewritten in the form

$$K_{st,emp} = \frac{1.22 \cdot R_c}{2 \cdot a \cdot \gamma \cdot h}, \qquad (13)$$

where $K_{st,emp}$ is the empirical stability coefficient.

The stability coefficient values calculated using formulas (11 and 13) are shown in Fig. 4.

The curves shown in Fig. 5 coincide, indicating successful calibration. Therefore, in order to apply our results to the conditions of the Donbas region in formulas (7, 10 and 11), the actual vertical stress P_v should be replaced with the so-called calibrated stress. This stress should be determined using the following formula

$$P_v^* = 1.4691 \cdot P_v + 0.0038 \cdot P_v^2. \qquad (16)$$

The adjusted values of the maximum heaving depth f_{sat}^* and the stability coefficient $K_{st,sat}^*$ of water-saturated rock, calculated taking into account (16), are presented in Table 2.

The data shown in Fig. 4 indicate that the stability coefficient values calculated using formulas (1 and 11) differ. This is because in formula (1) the stability coefficient is determined under the condition of a 20 cm heaving of the excavation floor, whereas in formula (11) it is determined under the condition of complete failure of the foundation in the heaving zone.

Therefore, when applying formula (11) to the conditions of the Donbas region, it requires adjustment (sometimes referred to as calibration). Formula (11) can be rewritten in the form

Table 1

Results of determining the rock stability coefficient and the depth of the heaving zone

No	h, m	$b = 2 \cdot a$, m	Parameter name	Pore fluid pressure P, kPa				
				0	250	500	750	1,000
1	10	6	f_{sat}, m	3.26	3.03	2.76	2.42	1.94
			$K_{st,sat}$, units	11.53	9.86	8.13	6.28	4.23
2	100	6	f_{sat}, m	5.04	4.93	4.82	4.70	4.57
			$K_{st,sat}$, units	1.78	1.64	1.50	1.36	1.21
3	200	6	f_{sat}, m	6.47	6.40	6.33	6.26	6.18
			$K_{st,sat}$, units	1.14	1.08	1.01	*0.94*	*0.88*
4	250	6	f_{sat}, m	7.07	7.01	6.96	*6.90*	*6.83*
			$K_{st,sat}$, units	1.00	*0.95*	*0.90*	*0.84*	*0.89*
5	300	6	f_{sat}, m	*7.63*	*7.58*	*7.53*	*7.48*	*7.43*
			$K_{st,sat}$, units	*0.90*	*0.86*	*0.81*	*0.77*	*0.73*

Notes: 1. In Table 1, the following notations are used: h – depth of the excavation floor; $b = 2 \cdot a$ – excavation width; f_{sat} – maximum heaving depth of water-saturated rock; $K_{st,sat}$ – stability coefficient of water-saturated rock in the heaving zone. 2. Critical values of the stability coefficient and heaving depth are indicated in italics

Table 2

Adjusted values of the rock stability coefficient and the depth of the heaving zone

No	h, m	$b = 2 \cdot a$, m	Parameter name	Pore fluid pressure P, kPa				
				0	250	500	750	1,000
1	10	6	f_{sat}^*, m	3.56	3.36	3.13	2.85	2.53
			$K_{st,sat}^*$, units	5.64	4.93	4.19	3.42	2.61
1*	49.5	6	f_{sat}^*, m	7.07	*7.01*	*6.96*	*6.90*	*6.83*
			$K_{st,sat}^*$, units	1.000	*0.945*	*0.895*	*0.842*	*0.790*
2	100	6	f_{sat}^*, m	*12.56*	*12.54*	*12.52*	*12.50*	*12.48*
			$K_{st,sat}^*$, units	*0.49*	*0.48*	*0.46*	*0.45*	*0.44*

Notes: In Table 2, the following notations are used: h – depth of the excavation floor; $b = 2 \cdot a$ – excavation width; f_{sat}^* – maximum heaving depth of water-saturated rock; $K_{st,sat}^*$ – stability coefficient of water-saturated rock in the heaving zone. See the explanation for Table 1. 3. Some critical values of the heaving depth and stability coefficient have been excluded from the table, as they have neither scientific nor practical significance

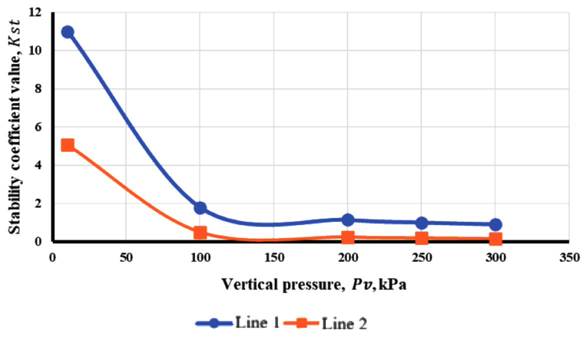


Fig. 4. Dependence of the stability coefficient K_{st} on the vertical stress P_v:

Line 1 – calculation performed using (11); Line 2 – the same, using (13)

$$K^*_{st,dry} = \frac{\sqrt{R_c \cdot R_p + K_{cor} \cdot P_v \cdot (R_c - R_p)}}{K_{cor} \cdot P_v}, \qquad (14)$$

where $K^*_{st,dry}$ is adjusted stability coefficient; K_{cor} is correction factor, which should be determined by comparing (14 and 13). It was found that this factor is equal to

$$K_{cor} = 1.4691 + 0.0038 \cdot P_v. \qquad (15)$$

The stability coefficient values calculated using formulas (14 and 13) are shown in Fig. 5.

It follows from Table 2 that, for the conditions of the Donbas region and the rock properties considered, rock heaving begins at a depth of approximately 49 meters.

The analysis of the data presented in Tables 1 and 2 made it possible to draw the following conclusions:

1. The data presented in the fifth column of Table 1 fully coincide with the results obtained by the authors of [9] for rocks with zero pore fluid pressure.

2. At the same time, the data presented in the fifth column of Table 2 fully coincide with those calculated using formula (13). If excess pore fluid pressure is present in the foundation, the following regularities are observed:

2.1. There is a pronounced tendency toward a decrease in the stability coefficient, and consequently in the rock stability within the heaving zone.

2.2. With increasing pore fluid pressure, the maximum depth from which rock heaving begins decreases.

Conclusions. The obtained research results allow the following conclusions to be drawn:

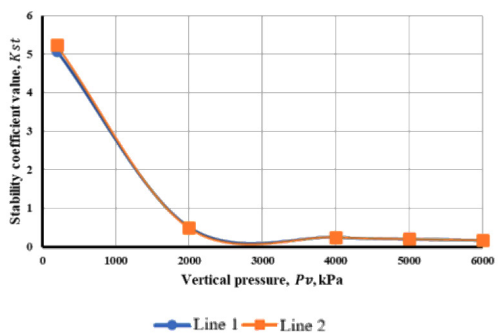


Fig. 5. Dependence of the stability coefficient K_{st} on the vertical stress P_v:

Line 1 – calculation performed using (14); Line 2 – the same, using (13) Lines 1 and 2 coincide

1. It is shown that the modification of the arching effect theory of M. M. Protodyakonov, proposed by the authors of [24], can be effectively used to determine the geometric dimensions of the floor rock heaving zone in underground excavations under conditions of excess pore pressure.

2. To solve the problem, it is proposed to introduce the stability coefficient $K_{st,sat}$, which is numerically equal to the ratio of the forces restraining the rock in the heaving zone to the forces causing its sliding. In this case, if $K_{st,sat} > 1$, the rock in the heaving zone is in a stable state; $K_{st,sat} = 1$, in a neutral state; $K_{st,sat} < 1$, in an unstable state.

3. It is shown that:

3.1. At zero pore fluid pressure, the results obtained fully coincide with the data reported in [24] for non-water-saturated rocks.

3.2. If excess pore fluid pressure is present in the floor rocks, there is a clear tendency toward a decrease in the stability coefficient, and consequently in the rock stability within the heaving zone.

3.3. An increase in pore fluid pressure leads to a decrease in the maximum depth from which rock heaving begins.

4. An algorithm for calibrating the obtained results to specific engineering and geological conditions is presented.

5. Scope of application of the obtained results:

5.1. Prediction of the state of the rock mass in the floor of a mining excavation (i.e., whether heaving occurs or not).

5.2. Approximate determination of the boundaries of the floor rock zone in which rock heaving occurs.

5.3. Determination of initial input data for the first approximation when solving problems related to the determination of heaving zone parameters using numerical methods with iterative procedures.

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Експрес-метод визначення параметрів здимання водонасиченої гірської породи

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Задача прогнозування здимання гірських порід, визначення його величини й окреслення меж області, де воно проявляється, є надзвичайно важливим під час проєктування підземних виробок, планування обсягів ремонтних робіт, а також вибору ефективних методів забезпечення тривалої стійкості підземних споруд і комунікацій, що розташовані у виробках.

Мета. Теоретичне встановлення механізму впливу надлишкового тиску рідини в порах гірської породи на процеси її здимання в умовах підземних виробок.

Методика. Дослідження виконано теоретичним шляхом на основі аналізу геомеханічних процесів, що розвиваються в зоні навколо горизонтальних гірничих виробок. Для цього застосовані аналітичні й чисельні математичні методи, а також проведено аналіз і систематизацію отриманих таким чином теоретичних результатів.

Результати. Встановлені прості аналітичні співвідношення, що дозволяють обчислювати межі області основи, де проявляється здимання водонасичених порід, і визначати коефіцієнт стійкості цієї області. В якості такого коефіцієнту запропоновано використовувати відношення проєкції сил, що утримують породний масив від здимання, на вертикальну вісь до проєкції сил, які ініціюють здимання породи. Виконана калібровка отриманих у дослідженні формул до умов Донбасу.

Наукова новизна. Уперше доведено, що зі збільшенням тиску порової рідини за однакових умов відбувається зменшення максимальної глибини області здимання основи. Показано також, що зростання порового тиску спричиняє одночасне зменшення коефіцієнта стійкості й максимальної глибини здимання.

Практична значимість. Отримані результати дозволяють із використанням математичних методів виконувати прогноз стійкості горизонтальних виробок, схильних до здимання порід підошви, ураховуючи глибину їхнього закладення, геометричні параметри, питому вагу й міцнісні характеристики порід, а також тиск у порах. Крім того, вони дають можливість визначати межі області здимання в конкретних гірничо-геологічних умовах. Теоретичні висновки роботи поширюються й на вирішення прикладних завдань технологічного характеру, зокрема під час силікатизації, цементації й високонапірної цементації ґрунтових основ. Це забезпечує визначення максимально допустимого тиску у водно-силікатних розчинах, що нагнітаються в породу, за якого не відбувається руйнування основи у процесі її зміцнення.

Ключові слова: *здимання водонасиченої породи, критерій міцності Кулона, критерій міцності Шашенка*

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