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GEOMECHANICAL MODEL OF FLOOR HEAVE IN LONG DEEP UNDERGROUND EXCAVATIONS

Purpose. To develop an adequate geomechanical model of floor heave in long-term underground excavations and to describe the transition from displacement-controlled support loading to load-controlled support loading.

Methodology. This involves a step-by-step modelling of the heaving process, taking into account the results obtained at the previous modelling stages and comparing the obtained results with in-situ measurements. The research tool used is the finite element method, implemented in the Phase2 software product from the Canadian company Rocscience.

Findings. A multi-stage geomechanical model of floor heave development in long underground openings was developed, taking into account the dilatancy effect under conditions of constrained volumetric deformation. A qualitative pattern of rock mass displacement around the excavation was obtained, showing a 90 % agreement with in situ monitoring data. The process of rock movement at the bottom of a working is approximated by an analytical dependence, on the basis of which it is possible to predict the operational state of capital workings and plan measures for their maintenance.

Originality. A digital geomechanical model of the rock mass heaving process around a single, extended working, constructed using the finite element method, is proposed. Unlike the existing models, it simulates the loss of elastic-plastic stability of the rock as a limit state, dividing the lining loading process into two stages: specified deformation and specified load. It has been proven that the process of rock deformation at the base of a permanent working with a cross-sectional area of 15–20 m^2 extends to a depth of 10–15 m. It has been established that maximum vertical displacements occur at the base of the working and decrease exponentially with increasing distance from it.

Practical value. The identified regularities of rock mass deformation around horizontal long underground openings under conditions of elastoplastic instability provide a basis for developing methods to control this process in both time and space. This can be achieved through the regulation of mining and maintenance operations during excavation development and service using a dynamic geomechanical model.

Keywords: *underground excavations, floor rock heave, stress-strain state, geomechanical model*

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Introduction. Under the current conditions of intensive mining at great depths, a substantial increase in deformation manifestations within the floor zone of underground excavations has been observed. One of the most complex and widespread phenomena is floor heave, which results in the loss of excavation stability, increased maintenance costs, and a reduction in both the safety and efficiency of mining operations. Despite more than a century of research, the physical nature of floor heave and the mechanisms governing its development remain insufficiently understood.

The problem has become particularly relevant because many underground excavations operate under the combined influence of high in-situ stresses, weak surrounding rocks, mine water, and technological factors associated with excavation driving and support maintenance. Floor heave is especially pronounced under dynamic impacts acting on support systems. In such cases, the phenomenon may be considered a transient process

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characterized by a transition from frame loading under prescribed displacements at the initial stage to loading under prescribed forces at the subsequent stage.

Under these conditions, traditional approaches to floor heave assessment and control are often inadequate. This necessitates the development of new conceptual models capable of describing the heave process not only during gradual deformation accumulation but also during the stage of sudden stability loss that is typical of deep underground excavations.

An important scientific challenge is the quantitative evaluation of critical displacements corresponding to the transition from the elastic-plastic state to the dilatancy stage, as well as the determination of the depth of the floor-heave development zone required to optimize design and technological solutions. Therefore, the development of numerical geomechanical models capable of reproducing the multi-stage nature of floor heave in accordance with modern instability theories is of particular relevance.

Literature review. A comprehensive review of studies devoted to floor heave in underground excavations is presented in [1]. The first references to rock floor heave in underground workings date back to the late nineteenth century. This period was characterized by the rapid development of the mining industry and the construction of extensive transportation tunnels in Asia, Western Europe, and the United States. Publications of that time, primarily authored by American researchers, covered the period from the late nineteenth to the early twentieth century. These studies examined in detail the influence of moisture on the strength of clay shales and the effects of the mine atmosphere. Particular attention was paid to the relationship between mechanical processes occurring simultaneously in the roof and floor strata.

At the beginning of the twentieth century, studies addressing floor heave problems in the coal mines of the Donbas region began to emerge, reflecting the growing practical significance of this phenomenon. A notable example is the paper by engineer A. Rohalevych, published in Gornyi Zhurnal, which presented observations of floor-rock behaviour in inclined roadways under different protection and maintenance methods.

Further development of scientific understanding of floor heave was driven by the accumulation of field observations and theoretical investigations conducted in various mining regions worldwide, as well as by the significant contribution of the Ukrainian geomechanics school. These efforts substantially advanced the understanding of rock-mass deformation mechanisms and the factors governing the stability of underground excavations.

In his work, M. Yevdokymov-Rokotovskyi summarized the experience gained during the construction of tunnels in Western Europe, Asia Minor, North and South America, Japan, and the London Underground. Concerning floor-rock uplift, the author emphasized the influence of lateral stresses under conditions of elevated and uniformly distributed rock pressure. This phenomenon was found to be most characteristic of weak rocks; however, substantial upward pressures may also develop when competent rocks occur in the floor. The floor heave observed in the Czernitz Tunnel (Upper Silesia), described by the author, essentially represents a case of instability of the excavation-rock mass system, manifested by a rapid increase in floor deformations after critical boundary displacements had been reached.

One of the first truly comprehensive studies of floor heave was conducted by A. Hurdus, who generalized both mine-operating experience and laboratory research results. He identified two principal factors responsible for floor heave development. The first factor is moisture, which causes the swelling of clay-bearing rocks. The second factor is the action of rock pressure generated by the overlying strata. A. Hurdus considered the former to be of secondary importance, whereas geostatic pressure was regarded as the primary factor controlling the initiation and development of floor heave.

Significant contributions to the study of floor heave were also made by F. Bielaienko, V. Hlushko, O. Maksymov, Yu. Zaslavskyi, I. Cherniak, V. Lytkin, and other researchers. Their approaches differed substantially from the concepts described above. More complex geological environments were considered, rheological effects were incorporated into the analysis, and advanced mathematical tools based on the theories of plasticity and creep were employed.

The data accumulated over several decades demonstrated that floor heave in underground excavations is considerably more complex than initially assumed. Consequently, its investigation required either rheological approaches or more sophisticated mechanical models. A large-scale research programme in this field was carried out at the mines of the Central and Western Donbas, as well as the Lviv-Volyn Coal Basin, by a scientific group led by O. Maksymov at Dnipropetrovsk Mining Institute.

Using the Bingham-Shvedov rheological model, O. Maksymov developed a mathematical description of the floor-heave process. The Navier-Stokes equations were adopted as the governing relations. The resulting solution provided analytical dependences for estimating floor displacements and support loads. For the first time, it was shown that floor heave is not confined to the immediate vicinity of the excavation contour but may extend over distances exceeding 10 m, which was consistent with field observations. A significant limitation of this approach was the inability to determine certain mechanical characteristics of the rock mass, including the Reynolds number.

Numerous studies devoted to floor heave and rock-mass deformation under geomechanical loading have subsequently been conducted in different countries, including Poland and China, where numerical modelling has become a widely used tool for analysing rock-mass behaviour in deep underground excavations. The reported findings confirm the necessity of further improving geomechanical models for predicting rock deformation and failure processes [2, 3].

Shashenko O., Khoziaikina N., and Tereshchuk R. [4] investigated the regularities of displacement distribution around a single excavation in a stratified rock mass, which enabled a more accurate description of deformation processes within the near-contour zone.

In the study by R. Tereshchuk, N. Khoziakina, and D. Babets [5], numerical modelling was employed to substantiate rational parameters of rock-bolt support systems and to evaluate their influence on the stress-strain state of the surrounding rock mass.

In recent years, increasing scientific attention has been paid to floor heave processes in underground excavations, as evidenced by numerous publications by both international and Ukrainian researchers.

In study [6], a mechanical model for controlling floor heave in deep underground excavations was proposed using a closed U-shaped steel support system incorporating an inverted floor arch. The authors combined analytical limit-equilibrium calculations with numerical modelling to evaluate the stress-strain state around the excavation. The results demonstrated that the introduction of a closed support contour significantly reduces the magnitude of floor heave and promotes the redistribution of plastic zones within the surrounding rock mass. The proposed approach has become one of the fundamental theoretical models frequently cited in subsequent floor-heave studies.

Field investigations conducted at an underground coal mine in Western Kentucky (USA) were reported in [7], where intensive floor heave developed due to the increased thickness of a weak clay-bearing layer. The authors analysed the sequence of failure events, geological conditions, support configuration, and the effectiveness of an integrated support strategy involving supplementary supports, pressure grouting, and modifications to the support scheme. The study also presented results of three-dimensional numerical modelling that were in good agreement with field observations. The value of this work lies in the detailed description of engineering measures that successfully mitigated floor heave under specific mining and geological conditions.

The authors of [8] investigated the mechanism of floor heave in water-saturated weak rocks and evaluated the effectiveness of floor reinforcement measures. Numerical simulations performed using the ANSYS software package demonstrated that water saturation causes a nonlinear increase in floor heave due to the reduction in the elastic modulus of the rock mass. To control deformation, reinforcement of the floor by means of an inverted arch was proposed. A stable correlation between moisture content and floor-heave magnitude was established, and an effective reinforcement depth was determined. These findings provide a basis for predicting the behaviour of water-saturated rocks and substantiating floor-support parameters in underground excavations.

Polish researchers [9] proposed a detailed numerical procedure for modelling floor heave under both dry and water-saturated conditions using the Hoek-Brown failure criterion and the Phase2 software environment. The mechanical properties of floor rocks were considered time-dependent variables affected by weathering and moisture ingress, allowing floor heave to be represented as an evolutionary process. Numerical predictions were compared with convergence and floor-heave measurements obtained in Polish mines, yielding a correlation level of 90–99 %. This study represents a valuable reference methodology for constructing numerical models of floor heave while accounting for moisture-induced degradation of rock properties.

Studies [10, 11] examined the mechanisms of floor heave in excavations driven through water-saturated soft rocks and assessed the effectiveness of modern stabilization techniques. The authors combined laboratory testing, numerical modelling, and field observations to analyse the development of plastic zones, stress redistribution, and floor deformation processes. It was shown that increasing moisture content results in reductions in rock strength, internal friction angle, deformation modulus, and cohesion, thereby significantly intensifying floor-heave manifestations. To control deformation, integrated engineering solutions were proposed, including the DS-IBA approach (deformation suppression and improved bolt-arch system) and grouting-based reinforcement of the surrounding rock mass. Numerical analyses revealed the existence of a "vertical swelling core" within the floor strata and established that the minimum effective grouting depth should fully encompass this zone. The obtained results demonstrate the high efficiency of combining structural support measures with geotechnical reinforcement techniques to mitigate floor heave and ensure the long-term stability of underground excavations.

In study [12], the mechanism of floor heave in deep underground excavations was investigated, and technological approaches for its control were proposed. Based on numerical modelling and field observations, the influence of the rock-mass stress state on floor deformation development was established. The authors also substantiated the effectiveness of employing a dense system of boreholes to reduce floor-heave manifestations and improve excavation stability.

A comprehensive approach to floor-heave control in deep underground excavations was presented in [13]. The proposed methodology was based on stress redistribution and the rational design of support systems aimed at limiting floor deformation and enhancing excavation stability.

The study reported in [14] investigated floor-heave development in a coal-mine haulage roadway using the discrete-element method implemented in the PFC2D software package. The results demonstrated that floor heave represents a progressive failure process within a stratified floor, governed by the stiffness, strength, and moment of inertia of individual layers. A control technique based on floor slotting and stress-relief measures was proposed. The optimum slotting depth was determined through numerical modelling to intersect fractured and weakened strata and to form a stable load-bearing floor. This study provides valuable information regarding stress-relief technologies for floor-heave mitigation.

In recent years, considerable attention has been focused on investigating the mechanisms of floor heave in excavations driven through water-saturated weak rocks and on developing effective mitigation techniques. Numerical modelling has demonstrated that both moisture content and rock mechanical properties strongly influence floor-heave intensity, whereas targeted support measures, including reinforcement and grouting of the near-excavation rock mass, significantly reduce deformation [10]. At the same time, the stabilization of deep underground excavations requires careful consideration of geological conditions and proper support-system design, since optimal support placement and timely monitoring are critical for preventing progressive floor heave [12]. Numerical models further indicate that high in-situ stresses may induce complex failure mechanisms involving localized shear failure and floor heave, thereby confirming the necessity of integrating geomechanical modelling with practical ground-support measures [3]. In-

vestigations of progressive excavation failure have shown that mining-induced stress redistribution can initiate time-dependent deformation processes, which are essential for predicting floor heave and planning preventive measures [15]. Long-term observations and numerical analyses indicate that floor displacement develops gradually, while moisture conditions and rheological properties significantly affect both the rate and extent of deformation [16]. Finally, studies of floor-instability mechanisms have demonstrated that the interaction among stress conditions, lithology, and support systems determines the effectiveness of reinforcement measures and the long-term stability of underground excavations [17].

Considering the operating conditions of deep underground excavations and drawing upon extensive time-dependent measurements of floor-heave development under various mining and geological conditions, O. Shashenko proposed a bifurcation model of floor heave [18]. According to this concept, a zone of plastic deformation develops around a deep excavation, and its dimensions increase as a function of the ratio between excavation depth, rock-unit weight, and the uniaxial compressive strength of the surrounding rocks. Once the plastic zone reaches a critical size corresponding to a critical displacement of the excavation boundary, the volume of plastically loosened rock can no longer be accommodated within the plastic deformation zone. As a result, a sudden change in the excavation floor contour occurs, accompanied by the upward movement of loosened rock into the excavation. This stage is identified as dilatancy-induced floor heave and is associated with an instantaneous reduction in stresses within the near-contour zone.

Subsequently, the flow of the loosened rock mass into the excavation may develop according to one of two scenarios: stable or unstable. This duality forms the basis of the term bifurcation theory of floor heave. At present, this concept appears to provide the most comprehensive interpretation of the physical nature of floor-heave development in deep underground excavations.

The idea underlying O. Shashenko's bifurcation theory is conceptually close to the hypothesis previously proposed by Yu. Zaslavskyi, who considered floor heave as a consequence of buckling instability within a rock layer subjected to lateral compressive forces. He suggested that a floor uplift of 0.2 m could be adopted as a criterion for floor-heave occurrence. This value is close to the criterion of 0.3 m proposed by O. Shashenko and O. Solodiankyn. Since floor heave represents the transition from the first loading stage of the support system to the second, this criterion should be verified and determined individually for specific mining and geological conditions.

Unsolved aspects of the problem. Based on the literature review, it has been established that the existing approaches to the investigation of floor heave in underground excavations are characterized by several limitations:

- insufficient quantitative assessment of the critical displacements preceding the loss of rock-mass stability;
- limited consideration of the long-term influence of the rheological properties of weak rocks;
- the absence of comprehensive numerical models capable of describing the two-stage mechanism of floor heave, comprising deformation accumulation followed by dilatancy and sudden floor uplift;
- limited information regarding the depth of the floor-heave development zone and its variation under different mining and geological conditions;
- insufficient agreement between field measurements and numerical modelling results obtained under actual mine conditions.

Therefore, despite the substantial amount of research conducted to date, the mechanisms governing the two-stage stability loss of the excavation-rock mass system, as well as the regularities of the multi-stage development of the floor-heave zone, require further investigation using advanced numerical geomechanical modelling techniques.

Purpose. The purpose of this study is to develop and investigate a numerical geomechanical model capable of reproducing the multi-stage process of floor heave within the zone of influence of a long horizontal underground excavation at great depths.

To achieve this purpose, the following objectives were defined:

1. To analyse the deformation behaviour of the rock mass under the action of rock pressure.
2. To determine the depth of the floor-heave development zone and the pattern of vertical displacement distribution.
3. To implement a numerical model based on the finite element method using the Phase2 software package.
4. To evaluate the consistency of the obtained results with contemporary floor-heave concepts, particularly the bifurcation model.
5. To determine the influence of the governing parameters on the onset of instability and the transition to the dilatancy-controlled stage of multi-stage floor-heave development.

Materials and methods. The study employed methods of deformable body mechanics, numerical modelling, scientific literature analysis, and the processing and interpretation of field and numerical investigation results.

Numerical modelling was performed using the Phase2 software package (Rocscience, Canada). The developed model incorporates:

- rheological properties of weak rocks;
- a two-stage deformation mechanism (deformation accumulation followed by dilatancy);
- changes in the stress-strain state within a zone extending 10–15 m beneath the excavation floor;
- nonlinear rock-mass behaviour when critical displacement values are reached.

The methodology was developed in accordance with O. Shashenko's concept of the bifurcation nature of floor heave. Numerical simulation of the floor-heave process was previously implemented in study [19]; however, a different approach was adopted for identifying the transition from the limiting stress-strain state of the rock mass weakened by the excavation to the loss of elastic-plastic stability of the floor rocks.

Findings. The geomechanical process is considered to be multi-stage in nature. Taking into account the excavation development technology, geomechanical processes evolve over time in a stepwise manner as the working face advances away from the considered cross-section. The excavation face acts as a barrier that restrains the development of the inelastic deformation zone and, consequently, the displacement of the excava-

tion boundary. As the face advances, its restraining effect gradually decreases, resulting in the expansion of the inelastic deformation zone around the excavation and a corresponding increase in boundary displacements. Finally, when the radius of the inelastic deformation zone reaches $3R_0$ and the boundary displacement attains 0.3 m, the first stage of geomechanical process development is completed [20].

At this stage, the stress-strain state of the rock mass weakened by the excavation can be described using the equations of elasticity theory. This stage can be readily simulated in the Phase 2 software package [21].

According to the bifurcation theory of floor heave, the second stage begins almost instantaneously and is characterized by the elastic-plastic instability of the excavation-weakened rock mass. This process is accompanied by large deformations that require consideration within the framework of elasticity theory supplemented by additional assumptions based on the laws governing rock deformation beyond the peak strength state [22, 23]. These laws permit the application of elasticity equations at the failure limit while incorporating a plastic stress component. In this case, displacements are calculated through a multi-stage procedure accounting for dilatancy, i.e., the increase in relative volume caused by plastic loosening of the rock. Dilatancy parameters may be determined from laboratory tests performed on stiff loading machines operating in a displacement-controlled mode beyond the peak strength limit [24, 25].

The subsequent stage of geomechanical process development can also be implemented in the Phase 2 software package, which allows modelling of elastoplastic behaviour based on the Hoek-Brown failure criterion. In this approach, the phenomenon of dilatancy, namely plastic rock expansion induced by critical stress conditions, is incorporated together with staged modelling procedures. The software retains the results obtained at previous stages and uses them as input data for subsequent calculations. This approach provides a more realistic representation of rock-mass behaviour by accounting for local stress concentrations and the gradual accumulation of deformation in critical zones. The numerical model also makes it possible to evaluate the influence of variations in strength parameters and elastic modulus on excavation stability, which is essential for assessing the risk of floor heave and local collapses. Such a comprehensive modelling framework enables prediction of rock-mass behaviour during excavation service life and provides a basis for selecting effective measures to improve excavation stability.

Fig. 1 presents the calculation scheme and finite-element mesh used for the first-stage analysis, during which the inelastic deformation zone develops from its initial dimensions to the limiting size preceding the onset of elastic-plastic instability in the rock mass surrounding the excavation. The model reproduces the mining and geological conditions of the Zakhidno-Donbaska Mine operated by PJSC DTEK Pavlohradvuhillia. The adopted rock properties are as follows: Young's modulus $E = 1{,}334$ MPa, Poisson's ratio $\nu = 0.3$, uniaxial compressive strength UCS = 31 MPa, uniaxial tensile strength UTS = 5.9 MPa, and rock density (siltstone) $\gamma = 2.3$ t/m^3. The excavation depth is 480 m.

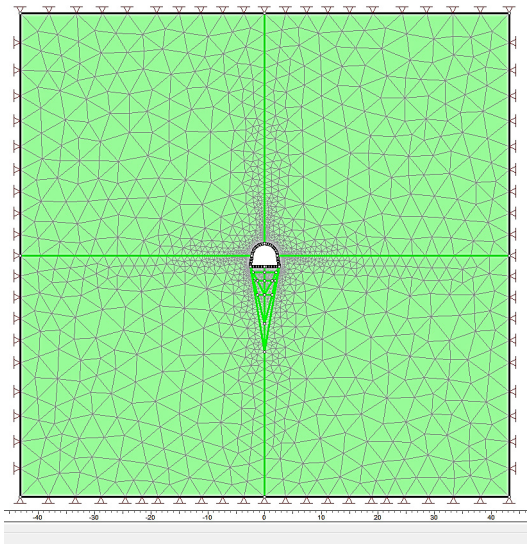

Fig. 1. Calculation scheme and finite-element mesh for the analysis of the inelastic deformation zone

The modelling scheme assumes that the elastic-plastic instability of the rock mass surrounding the excavation has already occurred. Under these conditions, the radius of the inelastic deformation zone is approximately equal to $3R_0$, while the displacement of the excavation boundary reaches 0.3 m. This state corresponds to the completion of the first stage of stress-strain state formation in the rock mass surrounding the excavation.

Subsequently, the excavation-rock mass geomechanical system loses its elastic-plastic stability, resulting in significant floor displacements. The zone of disturbed rocks affected by inelastic deformation extends up to 15 m beneath the excavation floor, which corresponds to approximately $6R_0$. This zone may be conventionally represented as an inverted triangular region with its apex directed downward and its base coinciding with the excavation floor. The finite-element mesh was refined in the vicinity of the disturbed rock mass located within this triangular zone.

The numerical simulation was performed in four stages, assuming that the rocks adjacent to the excavation boundary were subjected to the highest degree of disturbance. In this zone, the rock strength was assumed to be reduced to 10 % of the initial uniaxial compressive strength R_c, while plastic loosening reached its maximum value corresponding to a dilatancy coefficient of 0.15. Toward the outer boundary of the disturbed zone, the rock strength gradually approached its intact value R_c, the intensity of rock loosening decreased, and the dilatancy coefficient was reduced to 0.05, corresponding to the in-situ mechanical properties of the rock mass. The variation of both the dilatancy coefficient and the uniaxial compressive strength from their maximum values to minimum ones was assumed to follow a linear relationship.

Stage 1. A rock zone located within the excavation floor at a depth of $h = 0{-}1$ m was considered. The rock strength was assumed to be 10 % of R_c, while the dilatancy coefficient varied from 0.15 to 0.10.

Stage 2. The analysed floor-rock zone corresponded to depths of $h = 1{-}2.5$ m. The rock strength was taken as 25 % of R_c, and the dilatancy coefficient varied from 0.13 to 0.11.

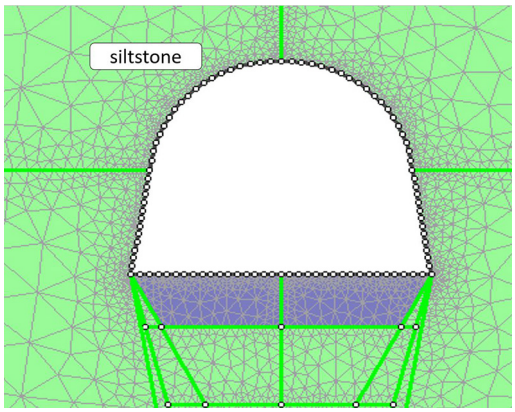

Fig. 2. Calculation scheme used in the first stage of numerical modelling

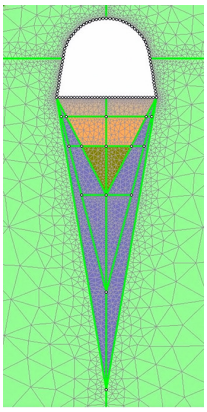

Fig. 3. Calculation scheme used in the fourth stage of numerical modelling

Stage 3. A triangular rock zone within the excavation floor at depths of $h = 2.5–5$ m was considered. The rock strength was assumed to be 50 % of R_c, and the dilatancy coefficient varied from 0.10 to 0.07.

Stage 4. A triangular rock zone within the excavation floor at depths of $h = 5.0–15$ m was considered. The rock strength was assumed to be 75 % of R_c, and the dilatancy coefficient varied from 0.07 to 0.05.

The successive stages of numerical modelling are illustrated in Figs. 2 and 3.

The modelling procedure was performed sequentially for each stage. Initially, the displacement field and the extent of the failed rock zone were determined for Stage 1. Subsequently, the results obtained at Stage 1 were stored and incorporated into the simulation of Stage 2. The same procedure was applied to each subsequent stage, with the results of the previous calculation step being used as input for the next stage. The calculation schemes adopted for Stages 1 and 4 are shown in Figs. 2 and 3, respectively.

The results obtained at the first and second modelling stages were accumulated and stored in the computer memory. A similar procedure was applied during the third and fourth modelling stages. The results of the fourth stage, which incorporate the calculations performed at all preceding stages, are presented in Figs. 4–6.

Fig. 4 shows the resulting zone of failed rocks surrounding the excavation after the fourth modelling stage. The results indicate that the dimensions of the failed (weakened) rock zone in the roof and sidewalls increased by 15–20 % compared with those obtained at the first modelling stage. In the floor strata, the disturbed zone extends to a depth exceeding 10 m. At depths ranging from 10 to 15 m, deformation of the rock mass remains predominantly within the elastic range.

As follows from Fig. 5, the displacement of the excavation boundary reaches approximately 0.7 m, whereas at a depth of 15 m within the floor strata the displacement does not exceed 0.01 m. These results are consistent with both analytical calculations and field observations.

An upward movement of the floor strata into the excavation by 0.48 m weakens the surrounding rock mass beneath the floor and along the sidewalls, creating favourable conditions for the development of roof falls (Fig. 6).

In addition to the imposed deformation from the surrounding rock mass during the first stage of critical inelastic zone formation, the steel arch support is subjected to an additional load from the weight of the rocks within the roof-fall zone. This fact should be taken into account when analysing the stress-strain state and determining the support parameters.

Fig. 7 shows the gradual increase in floor displacements with the progression of the modelling stages.

It can be seen that displacements attain their maximum values at the excavation boundary, where they reach 0.48 m during the final modelling stage. The displacement magnitude then gradually decreases with depth, approaching nearly zero values at distances of

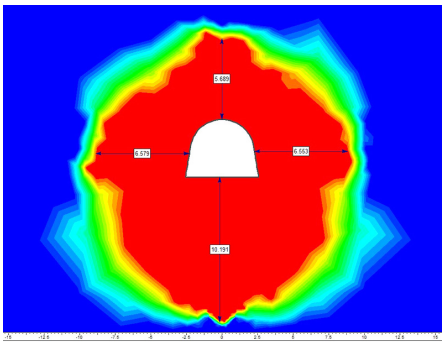

Fig. 4. Inelastic deformation zone around the excavation after the fourth stage of numerical modelling

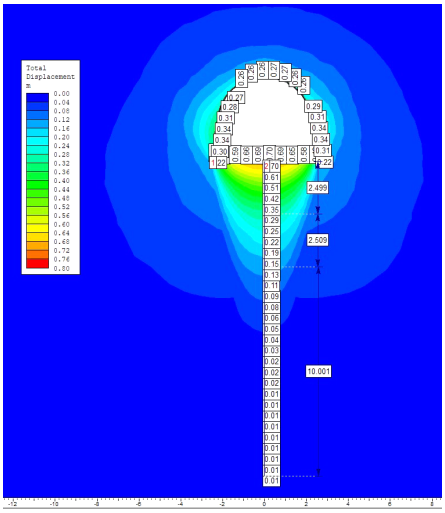

Fig. 5. Displacement distribution at the excavation boundary and within the rock mass to a depth of 15 m after the fourth modelling stage

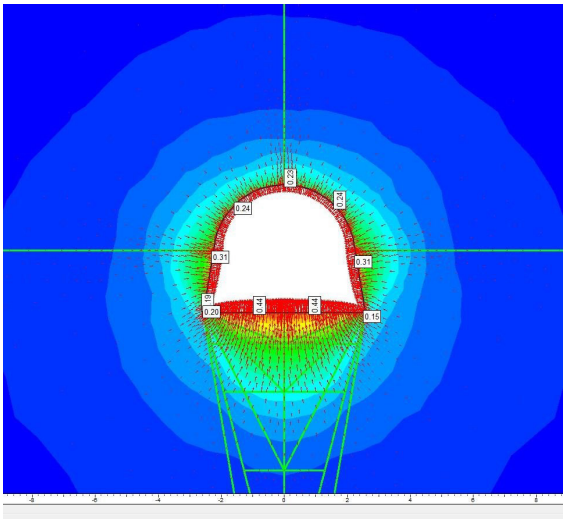

Fig. 6. Displacement vectors in the rock mass surrounding the excavation during floor heave

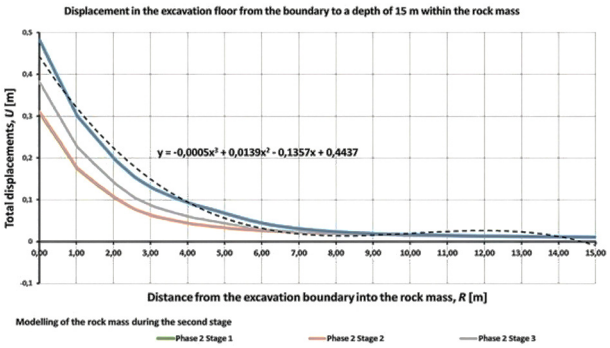

Fig. 7. Variation of vertical displacement with depth below the excavation floor

11–15 m below the excavation floor. The most active zone of plastic deformation is located at a distance of 1.5-2.5 m from the excavation boundary.

The decrease in displacement with depth below the excavation boundary follows an exponential law. The relationship can be expressed as follows

$$U = -0.0005R^3 + 0.139R^2 - 0.1357R + 0.4437.$$

Here, U is displacements, m; R is the depth below the excavation floor, m.

To verify the adequacy of the proposed geomechanical model, the results of field measurements of floor-rock displacements summarized in the monograph [1] were used. These measurements were obtained from investigations conducted in the mines of the Central Donbas. The corresponding experimental data are presented in Fig. 8. In the figure legend, numbers 1–9 denote individual field observation sites, the characteristics of which are given in [1]. A comparison of these data with the results of numerical modelling (Fig. 7) indicates a qualitative agreement in the pattern of displacement variation with depth within the rock mass. This confirms the adequacy of the proposed model for describing the floor-heave process.

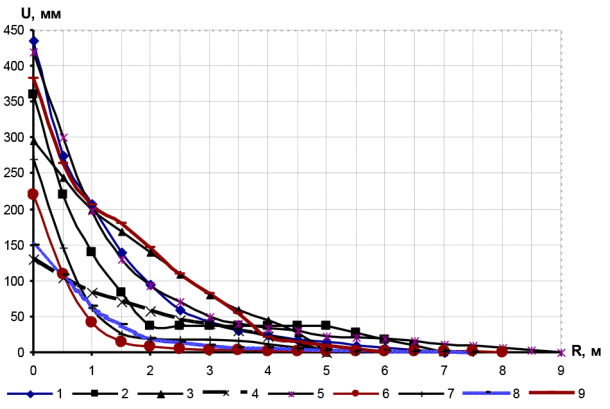

Fig. 8. Distribution of floor-rock displacements with depth below the excavation floor [1]

It follows from the graphs that the pattern of rock displacement within the excavation floor obtained by numerical modelling (Fig. 7) is consistent with that observed in field measurements. Both approaches show similar displacement profiles, comparable displacement magnitudes at the excavation boundary, and a similar depth of influence within the rock mass.

Thus, based on the results of numerical modelling, the following findings were established:

- the floor-deformation zone extends to a considerable depth of 10–15 m, which is consistent with field observations;
- the maximum vertical displacements occur directly at the excavation floor surface;
- the magnitude of floor heave decreases exponentially with increasing depth within the rock mass;
- the onset of instability corresponds to the attainment of critical displacement values, after which a dilatancy zone is formed;
- a significant stress reduction is observed within the near-contour zone following the transition of the rock mass to an unstable state;
- the developed model confirms the two-stage mechanism proposed by O. Shashenko: 1) the stage of plastic deformation accumulation and 2) the stage of dilatancy accompanied by sudden floor uplift.

Conclusions. A review of the available literature on the physical nature of floor heave in deep underground excavations was performed. This made it possible to identify the theoretical concepts that most adequately describe the floor-heave process in underground workings. The bifurcation model proposed by O. Shashenko was found to provide the most realistic representation of the geomechanical processes occurring around deep excavations, while the hydrodynamic model developed by O. Maksymov adequately reflects the geomechanical mechanisms responsible for floor heave.

A methodology for modelling floor heave in deep underground excavations was developed. The proposed approach is based on multi-stage simulation of deformation processes within a heterogeneous rock mass located in the floor-heave zone.

During the second stage of plastic deformation development in the rocks underlying the excavation floor, the modelling procedure was performed in four consecutive stages from top to bottom. At each successive stage, the mechanical properties of the rocks involved in the previous stages, including uniaxial compressive strength and dilatancy, were modified according to a linear law, which affected the resulting stress and displacement fields.

It was shown that displacements within the floor-heave zone attain their maximum values at the excava-

tion boundary and decrease exponentially with depth into the rock mass, approaching negligible values at depths of approximately 11–15 m.

The obtained relationships make it possible to control the mechanical properties of the rock mass through the application of grouting materials and rock bolts, as well as to manage the floor-heave process in underground excavations through the spatial and temporal optimization of excavation-support technologies using the principles of digital-twin-based geomechanical monitoring.

Future research. Future studies should focus on improving the proposed model by considering the effects of water saturation and long-term creep behaviour, applying three-dimensional modelling for the analysis of complex excavation geometries, investigating the influence of support parameters and technological solutions on floor-heave mitigation, and developing engineering approaches for predicting critical displacements in underground excavations at different depths.

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Геомеханічна модель пучення порід підошви у протяжних виробках глибокого закладення

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Мета. Розробка адекватної геомеханічної моделі процесу втрати стійкості порід підошви капітальних протяжних виробок, що є перехідним від стадії заданого деформування кріплення до стадії заданого навантаження.

Методика. Передбачає поетапне моделювання процесу пучення з урахування впливу моделей попередніх етапів і порівнянням отриманих результатів із натурними вимірами. В якості інструмента

досліджень використовується метод скінчених елементів, реалізований у програмному продукті Phase2 канадської компанії Rocscience.

Результати. Побудована багатоетапна геомеханічна модель процесу пучення порід підошви у протяжних виробках, що враховує явище дилатансії в умовах жорсткого об'ємного деформування. Отримана якісна картина переміщення породної маси навколо виробки, що на 90 % збігається із результатами натурних вимірів. Процес переміщення порід у підошві виробки апроксимовано аналітичною залежністю, на основі якої можна прогнозувати експлуатаційний стан капітальних виробок і планувати заходи щодо їх підтримання.

Наукова новизна. Запропонована цифрова геомеханічна модель процесу пучення породного масиву навколо одиночної протяжної виробки, побудована методом скінчених елементів. На відміну від відомих моделей, вона імітує втрату пружно-пластичної стійкості порід як граничний стан, що розділяє процес навантаження кріплення на дві стадії: заданого деформування й заданого навантаження. Доведено, що процес деформування порід у підошві капітальної виробки площею перерізу 15–20 м^2 поширюється на глибину 10–15 м. Встановлено, що максимальні вертикальні переміщення виникають у підошві виробки та експоненційно зменшуються зі збільшенням відстані від неї.

Практична значимість. Закономірність деформування породного масиву навколо горизонтальних протяжних виробок, у разі втрати ним пружно-пластичної стійкості, дозволяє розробити способи управління цим процесом у часі та просторі, регулюючи на основі динамічної геомеханічної моделі технологічні процеси під час проведення й експлуатації самої виробки.

Ключові слова: *капітальні виробки, пучення порід, напружено-деформований стан, геомеханічна модель*