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Prerequisites for studying a two-phase model of steel reinforcement loading in deep excavations

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Abstract. Deep underground excavations at great depths are characterised by increased rock pressure, floor heave, and instability phenomena that significantly alter the loading conditions of steel arch supports. Conventional quasi-static calculation models do not fully account for the transition to impulsive loading associated with the loss of elastic-plastic stability of the surrounding rock mass. The objective of this study was to develop and substantiate a two-phase loading model for steel arch supports in deep underground excavations, accounting for nonlinear interaction with the rock mass. The methodology combined analytical modelling based on the theory of a curved beam on an elastic foundation, finite element simulations, and comparison with in situ monitoring data. This integrated approach enabled analysis of the stress-strain evolution of the "support-rock mass" system and identification of a phase transition criterion. The results demonstrate significant differences in the stress-strain state of the steel support between quasi-static and impulsive phases. A phase transition criterion based on a critical excavation contour displacement is proposed and substantiated. In the first phase, maximum bending moments are concentrated at the arch crown and depend on displacement amplitude and flexural stiffness. In the second phase, loading becomes impulsive, and peak bending moments shift towards the support zones depending on the roof collapse configuration. The redistribution of internal forces under nonlinear contact interaction, as well as the influence of anchor stiffness, are analysed. It is shown that impulsive loading may lead the support to its ultimate limit state even when quasi-static conditions appear acceptable. The findings can be applied in the design and assessment of steel supports in deep mine workings and in the development of monitoring systems for early prediction of stability loss

Keywords: stress-strain state; heaving; geomechanical interaction; bifurcation model; bending moments

Introduction

Ensuring the stability of underground workings at great depths remains a critical issue in geomechanics and mining engineering due to increasing in situ stress levels and the complex behaviour of the rock mass. It has been established that at depths of 500 m and greater, stress redistribution leads to intensified plastic deformation, the formation of cracking zones, and loss of stability in excavation roofs. Floor heave and the

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associated displacements of the rock mass significantly affect the loading conditions of support systems. Under such conditions, conventional quasi-static approaches are often insufficient to describe the evolution of the stress-strain state of the "support-rock mass" system.

Recent studies indicate that floor heave remains one of the primary mechanisms of instability in deep underground excavations. Statistical analyses of underground transport tunnels demonstrated that floor uplift is controlled by geological structure, stress redistribution, and the mechanical properties of the surrounding rock mass. Modern support strategies therefore combine reinforcement of the floor zone with improvements in support-rock interaction in order to limit progressive deformation (Liu *et al.*, 2024). Additional investigations of deep soft-rock tunnels confirmed that deformation control requires not only strengthening of support systems but also pressure-relief measures and optimisation of excavation support schemes (Ling & Ying, 2024). M. Ma *et al*. (2022) investigated optimal support solutions for soft rock roadways using numerical modelling based on the Drucker-Prager yield criterion and demonstrated that appropriately designed combined support systems substantially reduce roadway deformation and improve excavation stability. Field investigations and numerical analyses conducted by Z. Tao *et al*. (2020) further showed that severe deformation in deep soft-rock tunnels results from the combined effects of high in situ stress, weak rock properties, and time-dependent behaviour, while integrated support systems comprising high-strength rock bolts, steel arches, and shotcrete provide the most effective deformation control. In addition to the immediate effects of stress redistribution, the long-term behaviour of deep underground excavations is strongly influenced by time-dependent deformation processes. Progressive deterioration of rock mass properties, creep, and stress relaxation may gradually bring the surrounding rock mass to a critical state even after excavation and support installation. K. Arora & M. Gutierrez (2021) demonstrated that viscous-elastic-plastic deformation mechanisms play a fundamental role in the evolution of squeezing ground conditions and significantly affect tunnel behaviour over time. Similar conclusions were reported by G. Li *et al*. (2020), who showed that deformation and failure processes in deep soft-rock roadways develop progressively and substantially influence support performance. The importance of time-dependent deformation was also confirmed by Q. Zhu *et al*. (2024), who demonstrated that creep significantly accelerates failure evolution in deep high-stress roadways and contributes to the progressive deterioration of surrounding rock stability. Recent studies have increasingly emphasised that excavation stability should be analysed within the framework of a coupled rock-support system rather than by considering support structures separately. T. Yang *et al*. (2023) demonstrated that instability development and support efficiency are strongly controlled by the interaction between reinforcement systems and the surrounding rock mass. Analytical investigations performed by K.-H. Ahn & A. Pouya (2023) further showed that contact interaction between tunnel support and surrounding rocks significantly affects load transfer mechanisms and stress redistribution around underground openings. Similarly, R.-C. Fu *et al*. (2024) developed an exact analytical solution for deep tunnels considering viscoelastic-plastic rock behaviour and the actual excavation loading path, demonstrating that loading history has a substantial influence on stress redistribution and tunnel deformation. Because direct observation of stress redistribution and support loading mechanisms in deep underground excavations is difficult, numerical modelling has become one of the principal tools for investigating geomechanical behaviour. Numerical studies conducted by I. Sakhno & S. Sakhno (2023) demonstrated the effectiveness of computational approaches in analysing floor heave development and evaluating reinforcement efficiency. Similarly, R. Fan *et al*. (2024) showed that numerical models provide valuable insight into deformation evolution and the identification of conditions leading to excavation instability. Geomechanical investigations by M. Kimour *et al*. (2023) likewise confirmed that numerical modelling integrated with rock mass classification methods provides a reliable basis for excavation design and stability assessment under complex geological conditions. Such approaches create a methodological basis for analysing the stress-strain state of the "support-rock mass" system and for developing advanced loading models. Another important feature of deep underground excavations is the possibility of abrupt changes in the stress-strain state during instability development. Experimental investigations performed by H. Shi *et al*. (2023) demonstrated that fractured rock masses exhibit complex mechanical behaviour under dynamic loading conditions, leading to significant changes in strength characteristics and failure mechanisms. Such observations indicate that instability development cannot always be adequately represented by conventional quasi-static approaches alone, particularly when critical deformation states are reached. Physical simulation experiments conducted by S. Zhang *et al*. (2026) further demonstrated that large deformation of surrounding rocks in deep high-geostress roadways develops through progressive damage accumulation, highlighting the importance of incorporating realistic deformation mechanisms into support design and stability assessment.

Despite significant advances in the investigation of floor heave mechanisms, deep roadway stability, support technologies, and support-rock interaction, existing studies are primarily focused either on gradual deformation processes or on individual manifestations of instability. Considerably less attention has been devoted to the transition between these states and its influence on the loading conditions of steel arch supports. In particular, the evolution of support loading from deformation-controlled behaviour to a regime characterised by rapid stress redistribution remains insufficiently formalised within existing geomechanical models. Therefore, the development of a two-phase approach capable of describing both gradual deformation accumulation and its subsequent transformation into intensified support loading remains an important scientific and engineering problem. The aim of this study was to develop and validate a two-phase loading model for steel arch supports in deep underground excavations that accounts for the nonlinear interaction between the support system and the surrounding rock mass.

Materials and Methods

The performance of underground excavations supported by steel arch systems was analysed taking into account the interaction in the "support-rock mass" system under increasing geostatic pressure and contour deformation. A two-phase loading scheme was adopted: a phase with controlled deformation, characterised by a gradual increase in the displacements of the excavation contour, and a phase of suddenly applied static load, which occurs after the loss of stability of the excavation bottom. The transition between the phases was governed by a predetermined critical displacement criterion corresponding to the onset of instability. The computational domain was represented as a fragment of the excavation under plane strain conditions, explicitly taking into account the geometry of the cross-section. The model dimensions were selected to eliminate the influence of boundaries in the central analysis zone. Anchor elements were located in accordance with the design configuration of the support system.

The research methodology was based on an integrated combination of analytical methods of deformable solid mechanics, numerical modelling using the finite element method (FEM) or the discrete element method (DEM), and comparative analysis with field observations. This approach ensured consistency between theoretical assumptions and field monitoring data. The analytical model of the steel arch support and its behaviour was described using the classical model of a curved beam on an elastic foundation (Winkler foundation model). Within this framework, the surrounding rock mass was represented as a system of distributed elastic springs, the stiffness of which characterised the resistance of the rock mass to deformation. The governing differential equation describing the deformation of the steel arch under the influence of external loading and the reaction of the rock mass was written in the following general form:

$$Id4u(x)/dx4 = R(x, t), \tag{1}$$

where $u(x, t)$ – the vertical displacement of the support element at point x and time t; EI – the bending stiffness of the steel element; $R(x, t)$ – the distributed load exerted by the surrounding rock mass The internal forces in the model are determined by the combined stiffness of the surrounding rock mass, the anchor reinforcement system, and the steel arch support, which is consistent with contemporary approaches to support-rock interaction modelling (Ahn & Pouya, 2023; Yang *et al.*, 2023). For rock bolt support:

$$N = k_a(u_n - u_{nop}) = E \cdot \varepsilon, \tag{2}$$

where k_a – the anchor stiffness; u_a and u_r – the displacements of the anchor and the surrounding rock mass, respectively; and ε is the anchor strain. For the steel arch support, the normal forces, shear forces, and bending moments are considered.

Normal force:

$$N = EA \cdot \frac{du}{ds}. \tag{3}$$

Shear force:

$$Q = \frac{dM}{ds}. \tag{4}$$

Bending moment:

$$M = -EI \cdot d^2w/ds^2, \tag{5}$$

where E – the modulus of elasticity of the arch; A – the cross-sectional area; I – the moment of inertia of the projected profile; u and w are the longitudinal and transverse displacements, respectively, and s – the coordinate along the arch. These expressions make it possible to describe the stress-strain state of the combined support system under complex external loading and to evaluate the interaction between support elements and the surrounding rock mass under varying geomechanical conditions (Ahn & Pouya, 2023). The analytical framework combines methods of deformable solid mechanics with numerical modelling based on the finite element and continuum approaches. Similar methodologies have been successfully applied in the analysis of floor heave development, support performance, and excavation stability in deep underground workings (Sakhno & Sakhno, 2023; Fan *et al.*, 2024). A comparative analysis was performed between the measured excavation contour displacements obtained from field monitoring and the corresponding analytical and numerical predictions, which is consistent with modern practices in geomechanical research.

Ensuring the stability of deep underground excavations remains a critical issue in modern geomechanics and geotechnical engineering. Studies of deep soft-rock roadways demonstrated that increasing depth is associated with higher stress concentrations, enhanced deformation of the surrounding rock mass, and deterioration of excavation stability conditions (Li *et al.*, 2020). Furthermore, investigations of floor heave mechanisms confirmed that deformation of the excavation floor represents one of the principal manifestations of instability and therefore should be explicitly considered in geomechanical calculations and numerical analyses (Mo *et al.*, 2020; Sakhno & Sakhno, 2023). The operational performance of underground excavations is ensured through the installation of steel arch support systems. Two loading phases are considered: a deformation-controlled phase under prescribed, gradually increasing displacements; a phase of impulsive distributed loading occurring after the loss of stability of the excavation floor. The proposed model is based on the concept of piecewise-continuous loading, with phase transition governed by a predefined critical displacement criterion.

The computational domain represents a fragment of the excavation under plane strain conditions, explicitly accounting for the geometry of its cross-section. The size of the modelling domain is selected to eliminate the influence of boundary conditions on results obtained in the central region. Anchor elements are positioned according to the design specifications. The geomechanical parameters of the support-rock system include rock mass mechanical properties, excavation depth, cross-sectional dimensions, support type, and steel profile characteristics. Rock mass properties are evaluated based on laboratory test results, field investigations, and numerical modelling. Key parameters include the elastic modulus, Poisson's ratio, uniaxial compressive strength, uniaxial tensile strength, internal friction angle, cohesion, and specific adhesion. These parameters govern the resistance of rocks to compression and shear and are critical for assessing the geomechanical conditions surrounding the excavation and for selecting a rational support system. The contact between the support and the surrounding rock mass is mathematically described as a nonlinear interaction between two structural elements. At the initial stage, load transfer occurs elastically, followed by a transition to plastic contact behaviour as deformation progresses. Consideration of contact friction characteristics, support stiffness, and non-uniform load distribution is essential in numerical simulations. These effects are modelled using specialised contact elements implemented in software packages such as FLAC3D, ABAQUS, or UDEC.

Depending on excavation depth and rock mass structure, the following primary failure mechanisms are considered: brittle failure with crack formation, plastic deformation, shear failure, and combined mechanisms. At great depths, elevated stress levels promote predominantly plastic and combined failure modes. Failure zones are identified using the Mohr-Coulomb criterion or alternative strength criteria, depending on the adopted material model and research objectives. During interaction with the rock mass, normal and shear forces develop within the steel arch elements and vary along the excavation perimeter. Their analysis is performed using limited equilibrium methods, the finite element method, or semi-analytical approaches. Internal forces-bending moments, shear forces, and axial forces-define the load-bearing capacity of the support elements and serve as key criteria for optimising support profiles. Three main stages of support performance during rock mass deformation are distinguished: an initial elastic stage, where the support operates within elastic limits; an intermediate stage, characterised by plastic deformation of the surrounding rock mass and partial load transfer to the arch; a critical stage, in which the contact zone fails, and the support operates under limit conditions. These stages describe the temporal evolution of loads acting on the support system.

The structural characteristics of the steel arch support, including the profile of steel sections, connection methods, and the presence of additional reinforcing elements, significantly influence its stiffness and stability. Analyses indicate that closed-loop systems reinforced with stiffening ribs provide a more uniform load distribution. The selection of support bearing points and support conditions is also crucial and depends on excavation width and lateral pressure characteristics. Accounting for secondary deformations during long-term operation of deep underground excavations is essential, as these are associated with rock creep and delayed settlements. Such processes may lead to additional loading of the support even after primary deformations have stabilised. To capture these effects, viscoelastic and rheological models must be employed, enabling prediction of the long-term evolution of the stress-strain state of steel arch support throughout the service life of the excavation. A necessary condition for implementing long-term prediction is the determination, through in situ investigations, of parameters describing the deformation characteristics of the rock mass, namely the time-dependent relationships of changes in the elastic modulus and Poisson's ratio.

Results and Discussion

The result of the study was the development and analytical study of a two-phase model describing the stress-strain state of a steel arch support in permanent underground workings. The model took into account the interaction with the surrounding rock mass, the transition from a quasi-static to an excited (impulse) state of loading, and separate mechanical mechanisms controlling each phase. The steel arch was modelled as an elastic curved beam with length L and bending stiffness EI. Hinged supports were provided at the base of the arch, where steel anchor bolts could be installed. The surrounding rock mass was represented as a solid medium capable of elastic-plastic behaviour with dilatancy after critical stress or deformation thresholds were exceeded. The interaction between the arch and the surrounding rock mass in both loading phases was analyses under plane deformation conditions.

The computational scheme considered a steel arch optionally reinforced with anchorage elements at the support points. In the first loading phase, the arch was subjected to a distributed displacement field $w(x)$ induced by the deforming surrounding rock mass. In the second phase, the loading was represented by a concentrated or distributed load $q(x)$ simulating the effects of roof instability or sudden stress redistribution. As an illustrative example, a standard double-track underground excavation supported by a steel arch made of SVP-27 special profile was analyses. The adopted parameters were: $L = 8.58$ m; $EI = 6 \times 10^5$ N/m^2; critical displacement $w_{cr} = 0.30$ m; the load distribution $q(x)$ is assumed to be piecewise constant. The modelling domain was defined so that the object of investigation was the steel arch support interacting with the surrounding rock mass deforming toward the excavation under in situ stresses and anthropogenic influences. The subject of investigation was the stress-strain state of the steel arch support. The surrounding rock mass was represented as a heterogeneous system including: a boundary zone of inelastic deformation; a fracture development zone; a direct contact zone where loosened rock interacted with the steel arch. The boundaries between modelling phases were determined based on field observations, geophysical measurements, and analytical assessment of critical displacement conditions.

The proposed loading model can be described as one of the options, as a time-dependent function that accounts for two distinct stages:

$$R(x, t) = (1 - \chi(t)) \cdot k(x)u(x, t) + \chi(t) \cdot (-q(x)), \quad (6)$$

where $k(x)$ – the modulus of the rock's elastic reaction; $q(x)$ – the intensity of the uniform vertical pressure from the collapsed mass; $\chi(t)$ – the Heaviside function, which defines the transition between the stages.

The transition between the loading regimes occurs when the critical vertical displacement in the floor rock mass is reached:

$$\chi(t)=\{0, \delta(t) < \delta cr1, \delta(t) \geqslant \delta kp, \quad (7)$$

where $\delta cr = 0.3$ m. The value $\delta cr = 0.3$ m was determined empirically for the geological and mining conditions of the Western Donbas coal basin based on field observations of floor heave processes and stability loss in deep excavations. This parameter should be considered as a calibrated value applicable to similar geomechanical environments. The critical displacement can be expressed in a generalised form as $\delta cr = f(H, \sigma c, E, S)$, where H – mining depth; σc – compressive strength of floor rocks; E – deformation modulus; S – support stiffness. However, for the considered case study, this relationship was simplified by adopting the empirically established value $\delta cr = 0.3$ m.

First regime: the displacements of the arch ends are prescribed. An approximate form of the displacement is used:

$$u(x) = u_{max}/2 \times (1 - cos(\pi x/L)), x \in [0, L]. \quad (8)$$

From this, the reaction of the rock mass can be determined:

$$R_1(x) = k(x) \times u(x), \quad (9)$$

and the bending moment:

$$M_1(x) = -EI \times (u_{max} \times \pi^2/(2L^2)) \times cos(\pi x/L). \quad (10)$$

Second regime: the load from the roof collapse is prescribed. This occurs after floor heave and the formation of roof falls:

$$R_2(x) = -q(x), q(x) = const = q_0. \quad (11)$$

A new form of $u(x)$ – determined, taking into account the boundary conditions, followed by the updated distribution of bending moments:

$$M_2(x) = -EI \times d^2u(x)/dx^2. \quad (12)$$

At the lower boundary of the digital model, fixed support conditions (zero displacement) are imposed. At the lateral boundaries, either symmetry or prescribed displacements are applied, depending on the symmetry of the problem. The upper boundary may be free or subjected to loading according to the depth of the excavation. The initial stress state is assumed to be hydrostatic or determined using justified empirical formulas.

Depending on the type of support, the boundary conditions for the steel arch are as follows:

hinged support: $u(0) = u(L) = 0,\ M(0) = M(L) = 0; \quad (13)$

fixed support: $u(0)=u(L)=0, du/dx|x=0=du/dx|x=L=0. \quad (14)$

Anchors are represented as elements that transfer loads to the rock mass through a contact zone. Their behaviour is defined by elastic properties (stiffness) as well as the parameters of the ultimate force at which failure occurs. Nonlinear contact interaction is modelled between the anchor and the surrounding rock mass. The model accounts for adhesion, possible micro-movements, friction, and strength reduction due to loosening. Upon reaching the limit conditions, detachment or sliding is implemented. For long-term prediction of the support system's behaviour, rock creep is taken into account. Rheological models such as Norton-Bailey, Burger, or others, which describe the dependence of deformation on time and stress, are applied. This allows considering the relaxation of loads in the "rock mass-support" system. Analytical solutions are possible in simple cases (constant k, cosine-shaped displacement); numerical implementation is carried out using the finite element method (FEM). The mesh is constructed with adaptation in zones of expected stress concentration. Nonlinear solution procedures with incremental load application and convergence control are used. Load distributions $R(x)$, displacements $u(x)$, and bending moments $M(x)$ are constructed for both phases.

The stress-strain state of the support, as well as the distributions of loads and bending moments for both phases, are schematically presented in Figure 1a and Figure 1b, respectively. These figures are intended to illustrate the proposed mechanical concept rather than represent specific measured data. Figure 1a schematically presents the time-dependent response of the "support-rock mass" system $R(t)$ and illustrates the mechanism of phase transition. The first phase corresponds to the regime of prescribed deformations, in which the load increases proportionally to the excavation contour displacement up to the level level R_1 at time t_1. Point B denotes the attainment of a critical contour displacement associated with the onset of elastic-plastic stability loss in the floor rocks and the initiation of floor heave development. From this moment, the system transitions to the second phase, characterised by a change in the interaction mechanism and the formation of impulsive loading, reaching the level R_2. Figure 1b schematically illustrates the variation in bending moment distribution within the steel arch support during each phase. In the first phase (prescribed deformation regime), a classical bending moment diagram is formed, with maximum values concentrated at the arch crown. In the second phase (prescribed load regime), redistribution of internal forces occurs due to nonlinear contact interaction and changes in loading configuration. This effect is conditionally represented by the modified shape of the bending moment diagram. Analysis of these dependencies allows identifying key features of steel support behaviour in each phase.

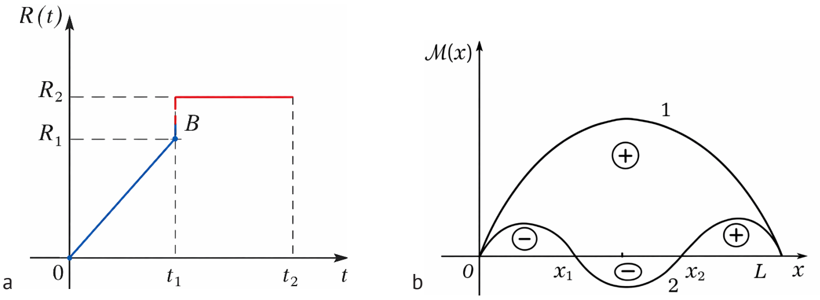

Figure 1. Conceptual schematic representation of the two-phase loading model of a steel arch support in deep underground workings

Notes: a) evolution of the load acting on the support during the first and second loading phases; b) distribution of bending moments in the support during the first and second loading phases

Source: developed by the authors based on the proposed theoretical model

In the first phase, under prescribed displacements from the rock mass, bending moments are a function of the deflection shape. The bending moment diagram has a sinusoidal form, with a maximum at the centre of the arch span. The maximum moment value is directly proportional to the displacement amplitude and the flexural stiffness of the support. The support state in this phase is considered quasi-static and controlled. In the second phase, the bending moment distribution assumes a classical form, transitioning from tensile forces along the contour to compressive forces. The combined moment diagram exhibits a complex shape depending on the parameters of the distributions in each phase. The presented calculations and diagrams illustrate changes in the deflection shape, moment distribution, and load nature in two fundamentally different phases. This allows clear differentiation of the support operation regimes, which is essential for designing reinforced structures adapted to real conditions at great depths.

Three main stages of the support's behaviour during rock deformation were identified: the first stage – the elastic stage-during which the support operated within the elastic range and load redistribution was moderate; the second is the transitional stage, characterised by plastic deformation of the surrounding rock and a gradual transfer of the load to the arch; and the third stage is the critical stage, during which contact deteriorated and the support approached its limit state under increased loading. These stages described the temporal evolution of loads acting on the support system and clarified the mechanism of transition from displacement-controlled behaviour to load-controlled response. The structural characteristics of the steel arch support-including profile geometry, connection methods, and the presence of reinforcing elements-were found to significantly influence stiffness and load distribution. Closed-loop systems reinforced with stiffening ribs provided a more uniform redistribution of internal forces along the perimeter. The selection of bearing points and boundary conditions was shown to depend on excavation width and lateral pressure characteristics. Secondary deformations associated with rock creep and delayed settlements were observed to produce additional loading of the support even after stabilisation of primary displacements. To capture these long-term effects, rheological and viscoelastic models were incorporated, enabling prediction of the long-term evolution of the stress-strain state throughout the service life of the excavation.

The results obtained were compared with studies of the last five years on the mechanisms of stability loss in deep underground excavations. The increase in plastic deformation with depth and the expansion of loosening zones identified in this study were consistent with the mechanisms of floor heave formation described by S. Mo *et al*. (2020). Their research confirmed that floor heave significantly influences support performance. The present results supported this conclusion and further specified the mechanical consequences of this influence through a displacement-based instability criterion. The change in stress distribution with increasing geostatic pressure was consistent with the deformation characteristics reported by G. Li *et al*. (2020), Y. Ling & X. Ying (2024). However, unlike their work, which primarily focused on empirical deformation observations, the present study introduced a formalised bifurcation-based transition criterion that defined the boundary between quasi-static and impulse phases.

The obtained results agree with the findings of I. Sakhno & S. Sakhno (2023) and R. Fan *et al*. (2024), who demonstrated that floor heave significantly influences excavation stability and support loading. The present study further develops this concept through a phase-dependent loading model. The numerical results regarding sharp increases in bending moments under instability conditions were consistent with the dynamic response patterns described by H. Shi *et al*. (2023). At the same time, the present model additionally demonstrated the migration of maximum bending moments from the arch crown toward the support zones during the impulse phase, which was not explicitly formalised in their analysis. The importance of nonlinear contact interaction confirmed in this research corresponded to the modelling results of K.-H. Ahn & A. Pouya (2023). Their conclusions indicated that neglecting contact nonlinearity led to an underestimation of internal forces. The present study supported this view and incorporated contact nonlinearity directly into the phase-dependent formulation.

The nonlinear mechanical behaviour of steel arches under high ground pressure described H. Shi *et al*. (2024) confirmed that geometric and material nonlinearities significantly influenced load-bearing capacity. The obtained results were consistent with these findings and demonstrated that such nonlinear effects became especially pronounced during the impulse loading stage. The time-dependent deformation processes identified in this work were comparable with the rheological behaviour reported by K. Arora & M. Gutierrez (2021), R.-C. Fu *et al*. (2024). Their results emphasised the significance of creep effects in deep roadways. The present study agreed with this interpretation and incorporated long-term deformation into the general stability framework. The effectiveness of reinforcement systems in controlling deformation of deep soft-rock roadways was confirmed by the numerical investigations of C. Zhang *et al*. (2026). Their results demonstrated that grouted rock bolting significantly improved the mechanical response of the surrounding rock mass and increased excavation stability under complex geomechanical conditions. The present study supports this interpretation and further extends it by demonstrating that support performance is strongly influenced by the phase-dependent evolution of loading conditions associated with floor-heave-induced instability.

The obtained results demonstrated that instability develops progressively and may be accompanied by substantial changes in the stress-strain state after reaching a critical deformation level. This interpretation is consistent with contemporary studies of deep roadway instability but extends them through the introduction of an explicit displacement-based transition criterion. The monitoring-based assessment of deformation development and support performance reported by S. Qin *et al*. (2026) confirmed that instability evolves progressively before reaching a critical state. The present findings are consistent with this interpretation and provide an analytical explanation of the observed deformation evolution within a unified mechanical framework. The probabilistic assessment of underground excavation stability proposed by F. de Silva *et al*. (2023) demonstrated the importance of accounting for uncertainty. Although the developed model remained deterministic, its structure allowed for further extension toward reliability-based analysis, which was considered a promising direction for future research.

The rock mass characterisation principles formulated by M. Kimour *et al*. (2023) confirmed that accurate mechanical parameterisation was essential for stability prediction. The present study supported this methodological position and applied compatible mechanical assumptions focused on phase evolution. In addition, the numerical investigations of nonlinear support-rock interaction conducted by T. Yang *et al*. (2023) demonstrated that coupled behaviour significantly influenced stress redistribution. The present results agreed with this conclusion and integrated the coupled interaction into the proposed framework. The mechanical representation of the support as a structural element interacting with an elastic foundation was conceptually consistent with the simplified finite element formulation. The beam-on-two-parameter-foundation model demonstrated that accounting for shear interaction within the foundation layer significantly affected bending moment distribution and displacement patterns. The present study agreed with this methodological approach and extended it to deep underground conditions by incorporating phase-dependent instability development and nonlinear contact interaction. Unlike the simplified elastic formulation adopted in their work, the current model additionally captured the transition to impulse loading and redistribution of internal forces after reaching the critical contour. Field observations of large deformation control presented by S. Zhang *et al*. (2026) confirmed that inadequate accounting for floor heave led to progressive loss of stability. The current findings supported this practical conclusion and provided a theoretical basis for interpreting such cases.

Overall, the reviewed studies confirmed the relevance of considering progressive deformation and failure evolution in deep underground workings. Most of the analysed works supported the key theses formulated in this article regarding the decisive role of stress redistribution, nonlinear interaction, and combined loading. However, unlike existing models, the present research formalised the impulse loading stage through a bifurcation-based displacement criterion and integrated nonlinear contact interaction, rheological effects, bending moment migration, and analytical load redistribution within a single consistent mechanical framework. This unified approach provided a comprehensive interpretation of support behaviour under deep geomechanical conditions and clarified the similarities and differences between the obtained results and contemporary scientific findings.

Conclusions

The necessity of considering the phased interaction between steel supports and the surrounding rock mass in deep underground excavations has been substantiated. It has been shown that the stress-strain behaviour of the "support-rock mass" system cannot be adequately described within a single loading regime due to fundamentally different mechanical responses before and after floor instability. A two-phase geomechanical model has been proposed and analytically justified. The model reflects the evolution of the stress-strain state during progressive rock mass deformation and formalises the transition between quasi-static and impulse loading stages. In the first phase, prescribed deformations of the surrounding rock mass are assumed. The support behaviour is governed by controlled bending, and its mechanical state is defined by the deflection geometry and stiffness of the arch elements. The bending moment distribution remains continuous and is primarily dependent on the imposed displacement contour. The transition to the second phase is triggered by the loss of elastic-plastic stability of the floor rock, leading to heave development, partial unloading of the surrounding mass, and local roof destabilisation. This transition results in an impulsive loading regime accompanied by rapid stress redistribution within the steel support.

The results demonstrate a fundamental difference in bending moment distributions between the two phases. After the onset of floor instability, the maximum bending moments shift from the arch crown towards the support zones, while stress concentration and internal forces increase significantly, raising the likelihood of reaching the ultimate load-bearing capacity. A bifurcation-based displacement criterion has been formulated to define the boundary between quasi-static deformation and impulse loading, providing a physically justified indicator of instability initiation. The analysis confirms that neglecting nonlinear contact interaction and phase transition effects leads to underestimation of internal forces and incomplete representation of support performance. The proposed model integrates nonlinear contact behaviour, stress redistribution, bending moment migration, and long-term deformation within a unified framework. The results are of both theoretical and practical significance, contributing to improved support design reliability and more accurate assessment of critical operational states. Future work should focus on experimental validation, three-dimensional modelling, probabilistic analysis, and the development of design-oriented software tools.

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Передумови дослідження двофазної моделі навантаження сталевого кріплення у виробках глибокого закладення

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Анотація. Глибокі підземні виїмки на значних глибинах характеризуються підвищеним тиском гірської породи, підйомом дна та явищами нестабільності, що істотно змінюють умови навантаження на сталеві арочні опори. Традиційні квазістатичні розрахункові моделі не враховують у повній мірі перехід до імпульсного навантаження, пов'язаного з втратою пружно-пластичної стійкості навколишньої гірської породи. Метою даного дослідження було розроблення та обґрунтування двофазної моделі навантаження для сталевих арочних опор у глибоких підземних виїмках з урахуванням нелінійної взаємодії з гірською породою. Методологія поєднувала аналітичне моделювання на основі теорії вигнутої балки на пружному фундаменті, моделювання методом скінченних елементів та порівняння з даними моніторингу на місці. Цей інтегрований підхід дозволив проаналізувати еволюцію напружено-деформаційного стану системи «опора-гірська порода» та визначити критерій фазового переходу. Результати демонструють значні відмінності у стані напруження-деформації сталевої опори між квазістатичною та імпульсною фазами. Запропоновано та обґрунтовано критерій фазового переходу, заснований на критичному зміщенні контуру виїмки. У першій фазі максимальні згинальні моменти зосереджуються на вершині арки та залежать від амплітуди зміщення та жорсткості на згин. У другій фазі навантаження стає імпульсним, а пікові згинальні моменти зміщуються у бік зон опори залежно від конфігурації обвалу покрівлі. Проаналізовано перерозподіл внутрішніх сил за умов нелінійної контактної взаємодії, а також вплив жорсткості анкера. Показано, що імпульсне навантаження може призвести до переходу опори в граничний стан, навіть якщо квазістатичні умови здаються прийнятними. Отримані результати можуть бути застосовані при проектуванні та оцінці сталевих опор у глибоких гірничих виробітках, а також при розробці систем моніторингу для раннього прогнозування втрати стійкості

Ключові слова: напружено-деформований стан; пучення; геомеханічна взаємодія; модель біфуркації; згинальні моменти