

Article

Study on Roof Support Technology of the Upward Two-Step Staggered-Drift Cut-and-Fill Mining Method Under Fractured Ore-Rock Conditions

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Abstract

To address the challenging stope roof support problem associated with the the Upward Two-Step Staggered-Drift Cut-and-Fill Mining Method (UTSCF method) under fractured surrounding rock conditions, this study establishes an elasto-plastic zoning model for drift roof surrounding rock to calculate the thicknesses of the loose zone and plastic hardening zone, proposes a combined support scheme comprising split-set (SS) bolts, metal mesh and cross-timber (CT) caulking for the working condition where backfill on both side walls fails to achieve full roof contact, and analyzes the support performance of this method via FLAC3D numerical simulation. The results indicate that the total thickness of the loose zone (LZ) and plastic zone (PZ) in the roofs of 6 m and 8 m drifts reaches 2.02 m and 2.15 m, respectively; after support application, the maximum principal stress (MPS) of the roof decreases by 49%, the plastic zone volume is reduced by 35.5–47.2%, and the average roof displacement drops by approximately 29%. Field industrial tests have preliminarily verified the engineering adaptability and stability control performance of the proposed support method. The mechanical mechanism of secondary roof contact by wedged cross timbers, as well as the quantitative relationship between theoretical zoning and support adequacy, still requires further verification.

Keywords: fractured surrounding rock; drift cut-and-fill mining method; roof support; elasto-plastic analysis; split-set bolts; secondary roof contact



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1. Introduction

The upward horizontal slicing drift cut-and-fill mining method (hereinafter referred to as the horizontal slicing method) has been increasingly applied under complex geological conditions such as fractured ore bodies, deep high in situ stress, and well-developed joint systems; however, fractured rock masses exhibit poor self-stability upon exposure in goafs, with prominent risks of roof separation and collapse during the unsupported roof interval [1], which severely threatens stope operational safety and constitutes a critical bottleneck restricting the broader adoption of this method.

Roof support technologies for the upward horizontal slicing cut-and-fill mining method have evolved into a mature system centered on combined bolt–anchor cable support. For large-span roofs in Class IV fractured rock masses, Long et al. [2] proposed

a combined support scheme comprising bolt-mesh-shotcrete, prestressed bolts, and prestressed anchor cables, which yielded marked improvements in plastic height and stress state compared with the original design. Pre-control roof technology using long anchor cables can effectively suppress caving in jointed roofs, and fully grouted anchor cables are capable of forming a self-balancing system within the roof [3]; Qin et al. [4] further verified via numerical simulation that this combined support scheme can effectively alleviate tensile stress concentration in stope roofs. The combined load-bearing action of backfill and bolts generates a compression zone inside the backfill, providing a mechanical basis for support design [5]. Both numerical simulation and rock mass quality classification approaches have confirmed that combined support can markedly reduce roof tensile stress and constrain plastic zone propagation [6,7]. Stope support in the upward horizontal slicing backfill mining method belongs to small-span stope support, and the stope span is generally controlled within 4–8 m. Existing numerical simulation studies indicate that roof stability decreases significantly when the span exceeds 8 m [8]; the stope span for upward horizontal slicing backfill mining can be extended to less than 12 m [9]. The dip length of the stope is generally no more than 50 m, and the strike length ranges from 10 m to 100 m. In some other domestic metal mines adopting upward drift backfill mining, the stope strike length is arranged to exceed 100 m [10].

In terms of anchoring mechanisms and support parameter optimization, scholars worldwide have carried out extensive studies on the mechanical behavior of the bolt-surrounding rock interface: the bond-slip constitutive model for the bolt-grout interface was established through pull-out tests [11–13], the interaction mechanism between bolts and surrounding rock and the economically reasonable threshold of bolt length were revealed based on the shear-lag model [14,15], and the load transfer mechanism was clarified from the perspectives of bolt geometric parameters and joint aperture [16,17]. For support parameter optimization, the mechanical mechanism whereby high pre-tightening force suppresses separation and slip by improving the shear strength of structural planes has been revealed [18–21], elasto-plastic analysis models for prestressed bolt support have been established [22,23], the optimal diameter difference between bolts and boreholes [24,25] and an equivalent material analysis method for bolt-reinforced surrounding rock [26,27] have been put forward, and an analytical model for combined prestressed bolt-anchor cable support has been developed [28]. Liu et al. [29] systematically reviewed the application conditions and accuracy constraints of the strength criterion method and mathematical model method in calculating LZ thickness; Li et al. [30] proposed a calculation method for the width and theoretical depth of zonal disintegration zones from the perspective of stress wave propagation. Theoretical calculation of the loose zone and zonal disintegration analysis provide theoretical explanations for predicting the failure range of surrounding rock [31,32].

In terms of novel support technologies, research achievements including the yielding support system [33], the butterfly-shaped bearing structure formed by ultra-high prestressed bolts [34], the yielding support mechanism of expansion pillars [35], prestressed expandable pillars [36–39], and graded reinforced combined support technology [40–42] have further expanded the technical approaches for support design under fractured surrounding rock conditions. The zonal support scheme proposed for mining roadways in extremely close coal seams [43] and the mechanized filling collaborative technology for inclined thin ore bodies [44] provide references for roof control in mines under similar conditions. Field measurement results using ground-penetrating radar have verified the theoretical prediction of the development law of the loose zone [45].

Nevertheless, three notable gaps persist in the existing literature. First, systematic research on roof support for the drift-based slicing cut-and-fill mining method remains lim-

ited. Most available studies focus on roadway or tunnel engineering, whereas drift stopes differ substantially from roadways in terms of excavation section geometry, mining-filling sequence, and surrounding rock disturbance characteristics, making existing findings difficult to extrapolate directly. Second, the handful of studies addressing drift stopes predominantly concentrate on optimizing individual support parameters or analyzing the applicability of specific support patterns, with a lack of systematic investigation into the instability mechanism of drift roof surrounding rock and support design methodologies. In particular, there is an absence of targeted theoretical guidance for roof support design in drift stopes under Class IV fractured rock mass conditions. Third, most studies operate under the assumption of complete backfill roof contact, overlooking the practical field condition of inadequate roof contact rates. The resulting unsupported roof gap can significantly impair the supporting capacity of backfill and alter the stress boundary conditions of the roof.

In view of the above, this paper takes a “three-under” copper mine (under buildings, traffic routes, and water bodies) with gently inclined medium-thick to thick ore bodies adopting the UTSCF method as the engineering background. Aiming at the difficult roof-support problem of grade-IV fractured rock mass, theoretical analysis is comprehensively used to investigate the elastoplastic characteristics of surrounding rock after roadway excavation and reveal the roof instability mechanism. A roof support design scheme and a secondary contact scheme for the condition where sidewall backfill cannot fully contact the roof are further proposed, and their rationality is verified by numerical simulation. A technical route of “theoretical analysis—support design—numerical verification” is formed. The specific contributions of this study are as follows. First, a three-zone elastoplastic model of “loosened zone—plastic hardening zone—elastic zone” is established. Different from the classical circular opening model, this model quantitatively relates the thickness of each zone to bolt anchorage length, providing quantitative criteria rather than empirical selection for support design. Second, a secondary contact mechanism is proposed for the condition that sidewall backfill fails to contact the roof. Specifically, timber cribs act as rigid load-transfer cushions to restore the continuous load transfer path between backfill and roof. The subsequently poured cemented backfill slurry infiltrates the fractures of the non-contacted roof and re-establishes lateral constraints after solidification, which explains why roof stability can still be improved under incomplete contact conditions. Third, a design principle for bolt length selection in staggered-drift stopes is established: the bolt length should cover the sum of the loosened zone and plastic hardening zone plus an anchorage segment embedded into the elastic zone. Verified by numerical simulation and field application, this principle can be extended to similar fractured-rock drifts and stopes under the UTSCF method.

2. Engineering Background

In a copper mine under “three-under” mining conditions (mining under buildings, railways and water bodies), villages, a concentrator and a tailings pond lie above the mining area. The No. 1 orebody in the main mining zone, located directly beneath a decommissioned tailings pond, occurs in layered and quasi-layered forms, dipping to the southwest at 10–30° (average 20°), with a burial depth of 58–626 m. The rock mass mechanical parameters are derived from ore-rock quality evaluation results in the existing rock mechanics report of the mining area. Sampling was carried out within the geological horizon hosting the orebody, covering the orebody as well as the hanging-wall and footwall surrounding rocks. The adopted parameters are listed below. The hanging-wall and footwall surrounding rocks have identical lithology: density $\rho = 2.77 \text{ g/cm}^3$, tensile strength $\sigma_t = 0.80 \text{ MPa}$, elastic modulus $E = 9.09 \text{ GPa}$, cohesion $c = 0.95 \text{ MPa}$, internal friction angle

$\varphi = 38.15^\circ$, Poisson's ratio $\mu = 0.22$; Orebody: density $\rho = 2.85 \text{ g/cm}^3$, tensile strength $\sigma_t = 0.65 \text{ MPa}$, elastic modulus $E = 5.09 \text{ GPa}$, cohesion $c = 0.86 \text{ MPa}$, internal friction angle $\varphi = 42.56^\circ$, Poisson's ratio $\mu = 0.26$; Cemented backfill: density $\rho = 1.95 \text{ g/cm}^3$, tensile strength $\sigma_t = 0.25 \text{ MPa}$, elastic modulus $E = 0.51 \text{ GPa}$, cohesion $c = 0.37 \text{ MPa}$, internal friction angle $\varphi = 35.00^\circ$, Poisson's ratio $\mu = 0.28$; Uncemented backfill: density $\rho = 1.90 \text{ g/cm}^3$, tensile strength $\sigma_t = 0.01 \text{ MPa}$, elastic modulus $E = 0.16 \text{ GPa}$, cohesion $c = 0.1 \text{ MPa}$, internal friction angle $\varphi = 34.00^\circ$, Poisson's ratio $\mu = 0.19$.

According to rock mass quality evaluation results, the orebody and its hanging wall and footwall surrounding rock are classified as Class III–IV rock masses, with moderate integrity (poor in local sections), medium strength but well-developed joint fissures, and moderately inferior integrity and stability, as shown in Figure 1. In situ backfill generally suffers from incomplete roof contact due to shrinkage and settlement, which further deteriorates roof stability. To tackle this problem, the mine adopts the UTSCF method (see Figures 1–3) with a staggered drift layout: within the same slice, the roof of the second-step drift is staggered by 1.5–2.5 m relative to that of the first-step drift, so as to optimize the roof stress state and constrain plastic zone propagation. Compared with the conventional scheme featuring a stagger of one full slice height, this layout retains the stress regulation advantages of the staggered structure while reducing redundant investment in mine development engineering.

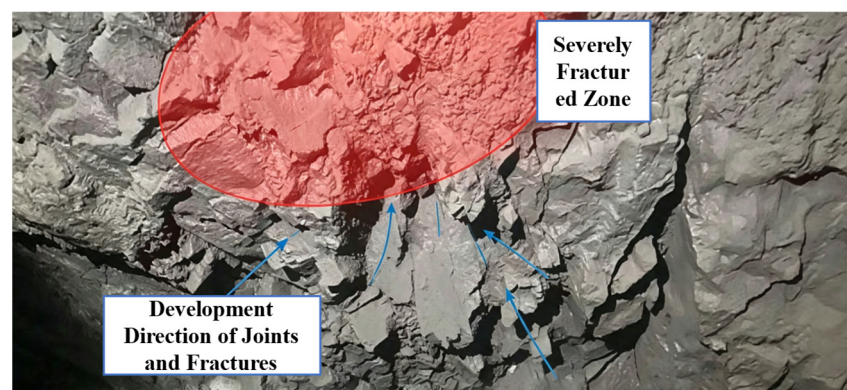


Figure 1. Fractured hanging wall rock mass with well-developed joint fissures.

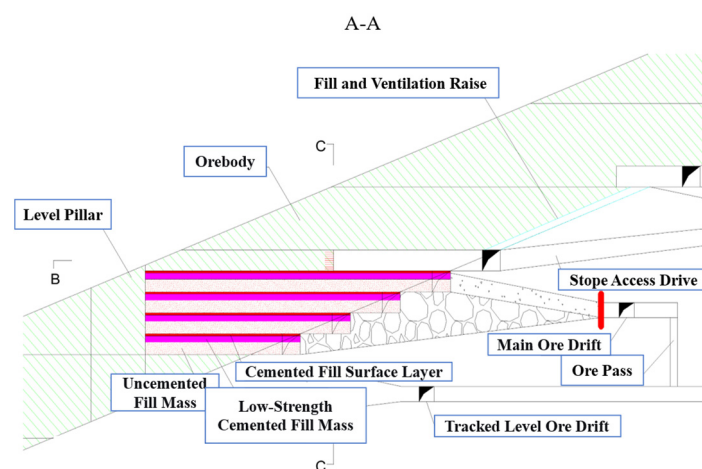


Figure 2. Cross section of the upward two-step staggered-drift cut-and-fill mining method.

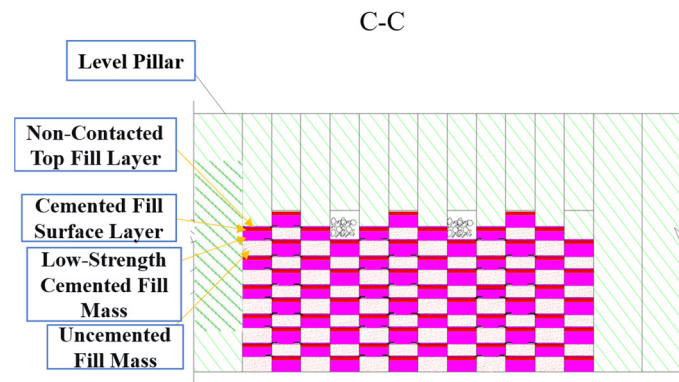


Figure 3. Longitudinal section of the upward two-step staggered-drift cut-and-fill mining method.

Ore blocks are arranged along the strike, with a panel length of 100 m, a width equal to the horizontal thickness of the orebody, and a slicing height of 5.5 m. A two-step strip mining scheme is adopted, with a span of 6 m for first-step strips and 8 m for second-step strips. Both the first and second steps are excavated sequentially in two sections following a “one-in-three” mining sequence; that is, the first section is excavated and backfilled before the second section is extracted. In the first slice, first-step strips are extracted first; after cemented backfilling to form artificial pillars, second-step strips in the same slice are mined. The roof of the second-step drift is advanced by 1.5–2.5 m relative to that of the first-step drift, forming a staggered roof structure. During the extraction of both steps, the previously excavated sections are filled with low-cement backfill, while the remaining sections are filled with uncemented tailings backfill. As an artificial pillar, the 28-day uniaxial compressive strength of cemented backfill is tested in accordance with the Technical Standard for Backfill Engineering in Metal and Nonmetal Mines. The strength indices are determined based on mine design documents: The 28-day compressive strength of primary bottom backfill shall not be less than 1.0–1.5 MPa; the 28-day compressive strength of secondary bottom backfill shall not be less than 0.5–1.0 MPa; and the 28-day compressive strength of surface backfill shall not be less than 2.0 MPa. The cement dosage of cemented backfill accounts for 3–25% of the dry mass of backfill materials. The cement-sand ratio is 1:6–8 for primary backfill, 1:12–15 for secondary backfill, and 1:4–5 for surface backfill.

3. Elasto-Plastic Theoretical Analysis of Drift Roof

3.1. Theoretical Model for Thickness Calculation of Elasto-Plastic Zoning in Surrounding Rock

3.1.1. Basic Assumptions and Calculation Model

To quantify the zonal failure characteristics of roof surrounding rock and the extent of the LZ after stope drift excavation, this paper introduces a surrounding rock zoning model for theoretical analysis. Built upon the classical elasto-plastic analysis framework for surrounding rock of circular chambers [46] and drawing on the surrounding rock zoning method proposed by Yuan et al. [47], this model divides the rock mass surrounding the chamber into three successive zones from the free face to the deep rock mass: the loose zone (LZ, fractured zone), plastic hardening zone (PH zone, yield zone), and elastic zone, as illustrated in Figure 4. To facilitate mathematical treatment, the following basic assumptions are made:

- The surrounding rock is regarded as a continuous, isotropic, and homogeneous elasto-plastic medium;
- The stope section is simplified as a circular chamber by the equivalent area method, with an equivalent radius $a = \sqrt{BH/\pi}$, where B denotes the drift span and H denotes the slicing height;

3.1.2. Equilibrium Equations and Yield Equations

As derived by Zhang et al. [50] in their axisymmetric elasto-plastic analysis based on the MC criterion, with body forces neglected, the radial stress σ_r and tangential stress σ_θ satisfy:

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \quad (1)$$

After the surrounding rock enters the plastic state, its stress state is governed by the MC yield criterion, which can be expressed in the principal stress space as:

$$\sigma_\theta = \frac{1 + \sin \varphi}{1 - \sin \varphi} \cdot \sigma_r + \frac{2c \cos \varphi}{1 - \sin \varphi} \quad (2)$$

where φ denotes the internal friction angle of the surrounding rock and c denotes the cohesion. By substituting the yield criterion into the equilibrium equations, the stress distribution within the plastic zone can be derived.

3.1.3. Non-Associated Flow Rule

Assume the plastic potential function is $Q = \sigma_\theta - \eta \sigma_r$, where:

$$\eta = \frac{1 + \sin \psi}{1 - \sin \psi} \quad (3)$$

ψ is the dilation angle, which reflects the degree of volume expansion of the rock mass after yielding. Based on the rock mass rating (RMR) and in situ development characteristics of joint fissures, the dilation angle is taken as 10° on the conservative side. The dilation angles of the LZ and plastic hardening zone are denoted as ψ_c and ψ_p respectively, with the corresponding parameters denoted as η_c and η_p .

3.1.4. Stress Expressions for Each Zone

In determining the stress distribution of each zone, stress expressions are derived separately for the LZ, plastic hardening zone, and elastic zone.

The stress distribution in the LZ ($a \leq r \leq R_c$) is given by:

$$\sigma_r^c = (p_0 + c \cot \varphi) \left(\frac{r}{a} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cot \varphi \quad (4)$$

$$\sigma_\theta^c = \frac{1 + \sin \varphi}{1 - \sin \varphi} (p_0 + c \cot \varphi) \left(\frac{r}{a} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cot \varphi \quad (5)$$

where p_0 denotes the support pressure, which is set to 0 under unsupported conditions.

By enforcing the continuity of radial stress at the interface between the LZ and the plastic zone, the stress distribution in the plastic hardening zone ($R_c \leq r \leq R_p$) is derived as:

$$\sigma_r^p = (p_0 + c \cot \varphi) \left(\frac{R_c}{a} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} \left(\frac{r}{R_c} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cot \varphi \quad (6)$$

$$\sigma_\theta^p = \frac{1 + \sin \varphi}{1 - \sin \varphi} (p_0 + c \cot \varphi) \left(\frac{R_c}{a} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} \left(\frac{r}{R_c} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - c \cot \varphi \quad (7)$$

Based on the classical solution to the thick-walled cylinder problem in elasticity theory, the stresses in the elastic zone ($R_p \leq r < \infty$) are expressed as:

$$\sigma_r^e = p - \left(p - \sigma_r^p \Big|_{r=R_p} \right) \frac{R_p^2}{r^2} \quad (8)$$

$$\sigma_{\theta}^e = p + \left(p - \sigma_r^p \Big|_{r=R_p} \right) \frac{R_p^2}{r^2} \quad (9)$$

where p represents the in situ virgin rock stress.

3.1.5. Governing Equations for Zonal Radii

The radii of the LZ R_c and the plastic hardening zone R_p are determined by solving the following two governing equations simultaneously.

1. Stress continuity must be satisfied at the elastic-plastic interface ($r = R_p$); that is, the stress state of the elastic zone exactly satisfies the MC yield criterion at the interface.

This condition can be written as:

$$\sigma_{\theta}^e \Big|_{r=R_p} = \frac{1 + \sin \varphi}{1 - \sin \varphi} \cdot \sigma_r^e \Big|_{r=R_p} + \frac{2c \cos \varphi}{1 - \sin \varphi} \quad (10)$$

$$R_p = \left[\frac{(p + c \cot \varphi)(1 - \sin \varphi)}{(p_0 + c \cot \varphi) \left(\frac{R_c}{a} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} + c \cot \varphi} \right]^{\frac{1 - \sin \varphi}{2 \sin \varphi}} \cdot a \quad (11)$$

2. Radial strain continuity is satisfied at the interface between the LZ and the plastic zone ($r = R_c$).

The tangential strain in the LZ is given by:

$$\varepsilon_{\theta}^c = \frac{1 + \mu}{E \zeta_c} \left[(p_0 + c \cot \varphi) \left(\frac{R_c}{a} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - (1 - 2\mu)(p_0 + c \cot \varphi) \right] + U \left(\frac{R_c}{R_c} \right)^{-(\eta_c + 1)} \quad (12)$$

The tangential strain in the plastic hardening zone is given by:

$$\varepsilon_{\theta}^p = \frac{1 + \mu}{E \zeta_p} \left[(p_0 + c \cot \varphi) \left(\frac{R_c}{a} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} \left(\frac{r}{R_c} \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} - (1 - 2\mu)(p_0 + c \cot \varphi) \right] + V \left(\frac{r}{R_p} \right)^{-(\eta_p + 1)} \quad (13)$$

The interface continuity condition is expressed as:

$$\varepsilon_{\theta}^c \Big|_{r=R_c} = \varepsilon_{\theta}^p \Big|_{r=R_c} \quad (14)$$

where ζ_c and ζ_p are material constants associated with Poisson's ratio, internal friction angle, and dilation angle, calculated as $\zeta = \left(\frac{1 + \mu}{1 - \mu} \right) \cdot \left(\frac{\eta - 1}{\eta + \mu} \right)$; U and V are integration constants determined from the displacement continuity conditions at zonal interfaces, which are eliminated via numerical iteration in the final solution for R_c and R_p . The radii R_c and R_p are obtained by simultaneously solving Equations (1) and (2).

3.2. Calculation Results for Zonal Thicknesses of Surrounding Rock

Once the radii of the LZ R_c and the plastic hardening zone R_p are determined by solving the governing equations for zonal radii, the thickness of each zone is calculated as follows:

$$L_c = R_c - a \quad (15)$$

$$L_p = R_p - R_c \quad (16)$$

where L_c denotes the thickness of the LZ and L_p that of the plastic zone. In accordance with the Technical Code for Engineering of Rock Bolting and Shotcrete Support (GB50086-2015) [51], the rock mass mechanical parameters from the mine rock mechanics research

report are reduced. Cohesion characterizes the cementation and interlocking effect between rock mass particles and is sensitive to joints, fissures, joint fillings, and weathering. It is reduced according to rock mass integrity, from 0.95 MPa to 0.72 MPa (reduction factor of 0.76). The internal friction angle is mainly controlled by discontinuity roughness, interlocking degree, and normal stress level. Its degradation cannot be characterized by a single reduction coefficient. Therefore, a value of 32.7° is adopted by combining the RMR classification results of grade-IV fractured rock mass in the mining area and engineering analogy. It should be noted that the rock mass mechanical parameters for theoretical calculation listed in Tables 1 and 2 are equivalent parameters reduced from the values in the above report. These conservative values are only used to determine the ranges of loosened zone and plastic zone requiring bolt anchorage. The zonal rock mass mechanical parameters used in the calculation are listed in Table 1, and the calculated zonal thicknesses of drift roofs with spans of 6 m and 8 m are presented in Table 2. In Table 1, c denotes the cohesion of the surrounding rock, and σ_p denotes the tensile strength of the surrounding rock.

Table 1. Structural dimensions of the chamber and mechanical parameters of the rock mass.

B/m	a/m	P/MPa	c/MPa	σ_p /MPa	$\varphi/(^\circ)$	E/GPa	μ
6	3.24	7.06	0.72	0.8	32.7	9.09	0.22
8	3.74	7.06	0.72	0.8	32.7	9.09	0.22

Note: B—drift span (m); a—radius of equivalent circular opening (m); P—in situ stress (MPa); c—cohesion (MPa); φ —internal friction angle ($^\circ$); σ —tensile strength (MPa); E—elastic modulus (GPa); μ —Poisson's ratio.

Table 2. Calculation results for zonal thicknesses.

B/m	R_c /m	R_p /m	L_c /m	L_p /m
6	3.87	4.78	0.84	1.18
8	4.36	5.38	0.82	1.33

Note: R_c —radius of loosened zone (m); R_p —radius of plastic strengthening zone (m); L_c —thickness of loosened zone (m); L_p —thickness of plastic strengthening zone (m).

4. Design of Secondary Roof-Contact Backfilling for Non-Contacted Backfill Layers

During staggered drift excavation (as illustrated in Figure 5), both the first and second mining steps are divided into two sections for sequential stoping: mining and backfilling of the first section are completed prior to stoping of the second section, following a skip-mining sequence of “one stope mined and three adjacent stopes retained” within each group. The roof stability of second-step stopes is significantly governed by the roof contact quality of the backfills on both sides; well-contacted first-step backfills provide effective roof support, whereas roof contact deficiencies are prevalent in first-step backfills in field practice, leaving the roof of excavated second-step drifts exposed with inadequate lateral confinement. Timber chock roof contacting is a widely adopted temporary support technique in mine roof management [52]; targeting the gaps formed by insufficient roof contact of the two side backfills, this study employs square CTs for gap filling as temporary reinforcement, which operates on the same principle as timber chock support: timber members fill the void between the roof and the supporting medium to establish a continuous load transfer path.

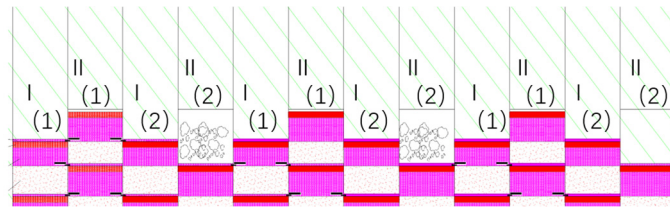


Figure 5. Stopping sequence diagram of the stope (I and II denote the first and second major mining steps, respectively; numerals 1 and 2 represent the excavation sub-steps within each major step).

4.1. Parametric Study on Cross-Timber Gap-Filling Roof Contacting

As illustrated in Figure 6, following excavation of the first section of the second-step stope (II-1), voids emerge between the backfills on both side walls and the roof surrounding rock. CTs are deployed to support the non-roof-contacted layers of the first-step backfills on both sides. As verified by field operating conditions, the CTs lack effective rock support at both ends and therefore cannot be analyzed using the simply supported beam bending model.

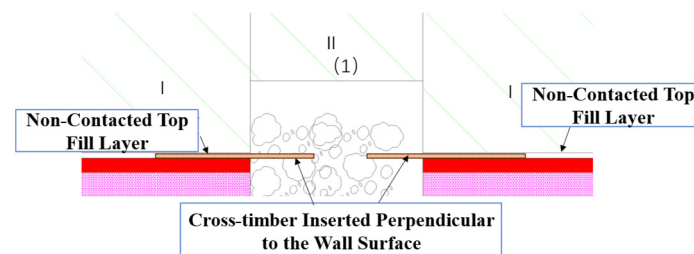


Figure 6. Schematic diagram of the first section of second-step excavation.

The CT serves only as a rigid load-transfer cushion between the backfill and the roof, and its mechanical behavior is characterized by axial compression along the grain direction; accordingly, the calculation model is revised from a flexural member to a compression member. Subject to axial compression along the grain, the bearing capacity of the CT is verified with reference to the calculation method specified in the Standard for Design of Timber Structures (GB50005-2017) [53].

1. The bearing capacity of a single CT is calculated based on the design value of compressive strength parallel to grain, and the bearing capacity of the axially compressed member shall satisfy:

$$N \leq f_c \cdot A \quad (17)$$

Bearing capacity of a single CT:

$$F_{\text{single}} = f_c \cdot A \quad (18)$$

where N is the design value of axial compressive force, in N or kN; F_{single} is the axial compressive bearing capacity of a single CT, in N or kN; f_c is the design value of compressive strength parallel to grain for timber, in MPa (N/mm²); A is the cross-sectional area of the CT, in mm² or m². The CT is configured as a rectangular bar with a square cross-section, whose side length is taken as 0.2 m, consistent with the thickness of the non-roof-contacted backfill layer.

2. The total required number of CTs is determined by dividing the total roof pressure by the bearing capacity of a single timber:

$$n * F_{\text{single}} = P_{\text{total}} \times S_{\text{stope}} \quad (19)$$

where S_{stope} is the total area of the stope, in m^2 ; P_{total} is the total roof pressure, in t or kN; and n is the total number of required CTs.

3. The layout spacing is determined according to the stope dimensions.

Equivalent bearing width of a single CT (i.e., layout spacing):

$$d = \frac{L_{\text{stope}}}{n_{\text{design}}} \quad (20)$$

where d : layout spacing of CTs, unit: m. As CTs serve as temporary supports, long-term creep may induce strength degradation of timber. Therefore, the safety factor is controlled at 3.5, and the value can be set as follows in practical application:

$$n_{\text{design}} = n \times 3.5 \quad (21)$$

The design strength of timber is taken as 40 MPa in accordance with Clause 4.3.1 of GB50005-2017. The calculated layout spacing of CTs is 1.37 m.

4.2. Secondary Roof Contact of Backfill Slurry

As shown in Figure 7, cemented backfill is adopted for the advance excavation space of the first part of the second step (II-1). Driven by the height difference, backfill slurry flows into the unfilled roof fissures of the first-step backfill masses on both sides to realize secondary roof contact, which greatly improves the roof stability of adjacent stopes. As illustrated in Figure 8, when excavating the second part (II-2), the backfill masses on both ribs have achieved complete roof contact, yielding a relatively safe excavation condition for the stope.

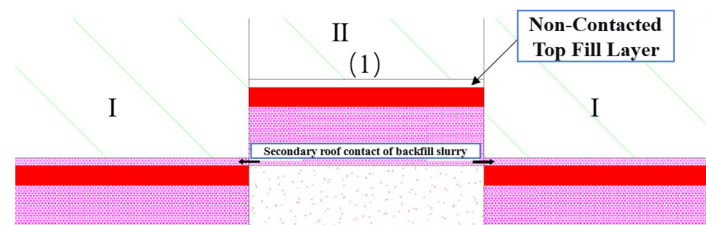


Figure 7. Secondary roof contact of backfill slurry at the stope roof.

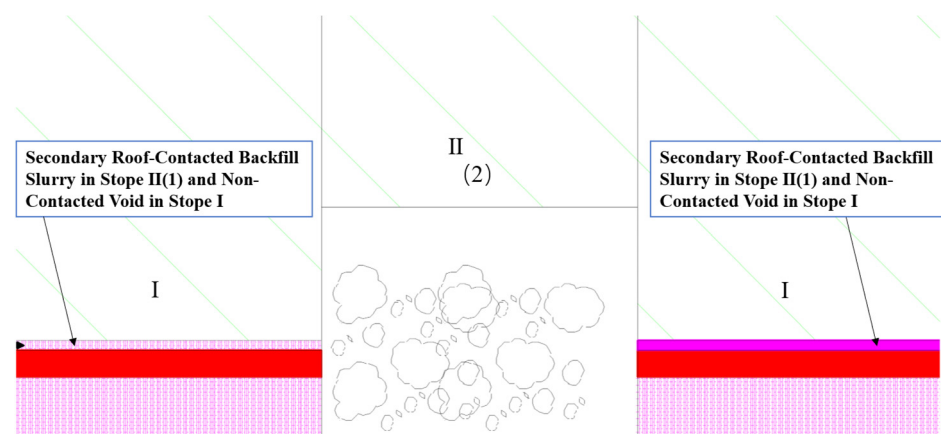


Figure 8. Schematic diagram for excavation of the second part in the second step.

5. Support Design of Drift Stopes Under Staggered Excavation

5.1. Selection of Support Scheme

In view of the engineering conditions of stope drifts with a span of 6–8 m, Grade IV fractured roof rock mass and poor self-stability, a combined support scheme of SS bolts and metal mesh is adopted in this study. This support scheme is selected based on the following considerations: layered drift mining is characterized by large preparatory excavation volume and frequent stope replacement, leading to relatively low mining efficiency. Therefore, the support method must adapt to the rapid mining advance rhythm to avoid excessive occupation of stoping time by support construction. SS bolts possess the advantages of fast installation and low cost, which is highly compatible with the operational principle of “rapid excavation and timely support” for layered drift mining. Its support mechanism is as follows (as shown in Figure 9): after the bolt is forcibly driven into the borehole, the elastic restoring force generated by the radial compression of the steel pipe applies a radial pressure P_1 to the borehole wall. The interfacial friction restricts the sliding and dislocation of bedding planes and fractures and cooperates with the surface constraint of the end pallet to jointly control the discontinuous deformation of the roof. When significant displacement occurs in the surrounding rock, the surrounding rock extrusion pressure P increases correspondingly, which further enhances the friction force. This realizes a favorable support performance of immediate load bearing after installation and progressive strengthening with rock deformation. Metal mesh is laid between adjacent bolts to prevent rock blocks from falling. Combined with SS bolts, it forms a composite support system featuring shallow rapid constraint and surface protection, which well meets the requirements of stope turnover efficiency for the layered drift filling mining method.

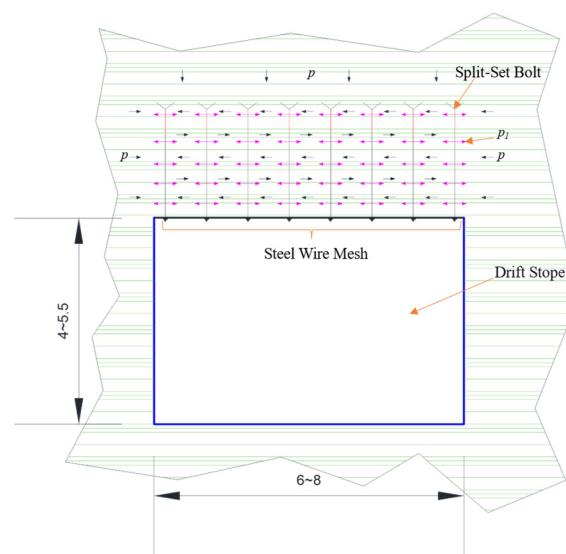


Figure 9. Schematic diagram of roof support scheme for drifts in fractured rock mass.

5.2. Investigation of Support Parameters

5.2.1. Row Spacing of Rock Bolts

The spacing and row spacing of rock bolts are determined based on the composite arch theory. Under the support of bolt groups, the compressive stress zones induced by individual bolts overlap to form a continuous compression arch. The spacing and row

spacing can be calculated according to the designed anchorage force of bolts and the load conditions of fractured rock mass:

$$a = b = \sqrt{\frac{N}{K \cdot \gamma \cdot h}} \quad (22)$$

where a and b are the bolt spacing and row spacing, respectively; N is the designed anchorage force of the bolt, which denotes the bearing capacity of the friction anchorage system between the bolt and the borehole wall. It depends on the friction coefficient, radial pressure, and contact area between the steel tube and borehole wall. According to MT/T285-2025 Slotted Tube Bolt [54], for slotted tube bolts with outer diameter ≥ 35 mm, the ultimate tensile capacity of the bolt shank ≥ 110 kN, pull-out resistance of the retaining ring weld ≥ 80 kN, and bearing capacity of face plate ≥ 80 kN. These three indices correspond to three failure modes: shank tensile rupture, retaining ring weld detachment, and face plate crushing. Considering the above three bearing capacities and incorporating an engineering safety margin, the designed anchorage force is taken as 150 kN. K is the safety factor (taken as 1.8); γ is the bulk density of rock mass; h represents the height of the caving arch, which is calculated by Protodyakonov's theory:

$$h = \frac{a}{f} \quad (23)$$

The span of stope drifts ranges from 6 to 8 m, corresponding to a half-span a of 3–4 m. The surrounding roof rock is classified as Grade IV fractured rock mass. The Protodyakonov coefficient f is set to 1.0, and the calculated height of the caving arch is 3–4 m (on the premise of ensuring smooth excavation, the roof support of secondary drifts with a large caving arch height is mainly considered; therefore, the value of 4 m is adopted). Substituting into the spacing and row spacing formula yields $a = b = 0.88$ m. For safety considerations, the actual bolt spacing and row spacing are selected as $0.8 \text{ m} \times 0.8 \text{ m}$. Prefabricated steel mesh is installed between adjacent bolts. The steel bars have a diameter of 8 mm with a mesh size of $100 \text{ mm} \times 100 \text{ mm}$. The overlapping length between mesh sheets shall not be less than 100 mm, and the mesh is compacted and fixed by bolt pallets.

5.2.2. Diameter of Rock Bolt

The anchorage force of SS bolts originates from the elastic restoring force induced by radial compression of the steel tube. Based on elastic mechanics and the frictional anchoring mechanism, Zheng Yutian et al. [55] established a theoretical formula describing the relationship between the anchorage force of SS bolts and geometric parameters of the bolt body, material properties, as well as mechanical parameters of surrounding rock:

$$N = \frac{\pi f_0 \sigma_s t^2}{6R\alpha} \times \frac{e^{\alpha l} - e^{-\alpha l}}{e^{\alpha l} + e^{-\alpha l}} \quad (24)$$

where the comprehensive deformation parameter α is calculated by the following formula:

$$\alpha = \sqrt{\frac{K}{Et}} \quad (25)$$

where: f_0 —friction coefficient at the bolt-borehole interface (0.30–0.45), adopted as 0.35; σ_s —yield strength of steel. The bolt shank is made of Q235 carbon structural steel. Although MT/T285-2025 Slotted Tube Bolt specifies the use of Q345 or higher-grade high-strength low-alloy steel, Q235 steel has been maturely applied in similar engineering projects within this mining area. According to GB/T 700-2006 [56], the tensile strength of Q235 steel ranges

from 370 to 500 MPa. Calculations verify that the ultimate tensile capacity of the bolt shank meets the requirement (≥ 110 kN) for slotted tube bolts with outer diameter greater than 35 mm specified in MT/T285-2025 Slotted Tube Bolt. Therefore, Q235 steel (yield strength of 235 MPa) is adopted; t —wall thickness of SS bolt, 2.0–3.0 mm; R —average radius of bolt, mm; l —effective length of bolt, m; K —shear stiffness coefficient of surrounding rock, MPa/m²; E —elastic modulus of steel, taken as 2.0×10^5 MPa;

Based on rock mechanics test results and RMR rock mass classification outcomes, combined with engineering analogy of shear stiffness coefficient for surrounding rock in the mining area, the iterative trial-and-error method is adopted to obtain the theoretical bolt radius under given wall thickness for different rock mass classes. The calculation results are presented in Table 3.

Table 3. Calculation results of SS bolt diameter.

Working Condition	Wall Thickness t (mm)	Comprehensive Deformation Parameter α (m ^{−1})	Calculated Diameter D (mm)	Recommended Diameter (mm)
Grade III–IV rock mass	2.5	10.0	34.4	36~38
fractured zones of Grade IV–V	3.0	6.45	37.0	40~42

5.2.3. Anchorage Length

According to the fundamental principle of suspension theory, after bolt installation, the suspension load borne by a single rock bolt equals the total weight of unstable rock strata within its control range. The calculation formula is expressed as:

$$N_{\text{actual}} = \gamma \cdot a \cdot b \cdot h \quad (26)$$

where N_{actual} —Suspension load borne by a single rock bolt, kN; h —Thickness of unstable rock strata (effective length), m.

The design basis for anchorage length is that the bond force between the anchoring agent and the borehole wall shall be sufficient to bear the actual suspension load transferred by a single rock bolt. In accordance with Technical Code for Rock Bolt and Shotcrete Support Engineering (GB50086-2015), the anchorage length shall be calculated by the following formula:

$$L_a \geq \frac{K_0 \cdot N_{\text{actual}}}{\pi \cdot D \cdot f_{mg}} \quad (27)$$

where L_a —anchorage length, m; K_0 —safety factor, taken as 1.8 considering the fractured characteristic of Grade IV surrounding rock in the roof; D —borehole diameter, m; f_{mg} —design bond strength between anchoring agent and rock mass of borehole wall, MPa.

According to SS Rock Bolts (MT/T285-2025), the initial anchoring force of SS bolts shall not be less than 30 kN/m. Calculated based on the effective bolt length h , the total initial anchoring force of a single bolt along its full length is expressed as:

$$N_i = 30 \times h \quad (28)$$

According to the above calculation results, the thickness of the loosened zone for the 8 m span drift is 0.82 m, the thickness of the plastic zone is 1.33 m, and the effective bolt length is 2.15 m. The initial anchoring force of a single bolt is 64.5 kN, which is larger than the actual suspension load (37.15 kN calculated from Equation (26)), satisfying the

suspension requirement. The initial anchoring force refers to the force required to pull the bolt shank out of the borehole after bolt installation and before significant surrounding rock deformation occurs. It represents the initial anchorage capacity of the anchoring system and is different from the designed anchorage force. As specified in MT/T285-2025, the initial anchoring force is defined as anchorage capacity per unit length. It is adopted to verify whether a single bolt can suspend the weight of unstable rock strata within its control range. In contrast, the designed anchorage force is the overall bearing capacity of a single bolt determined by comprehensively considering the ultimate tensile capacity of the bolt shank, pull-out resistance of the retaining ring weld, and bearing capacity of the face plate. It incorporates an engineering safety margin and is used for determining bolt spacing and row spacing. They differ in units, mechanical meanings, and application purposes and cannot be directly compared. As surrounding rock deformation increases, the compressive stress exerted on the bolt rises, and the frictional resistance increases accordingly. The anchorage force gradually rises from the initial anchoring force and approaches the designed anchorage force. Therefore, it is reasonable that the designed anchorage force is greater than the initial anchoring force.

When D is 40 mm (corresponding to the recommended bolt diameter $\Phi 36\text{--}42$ mm), the bond strength f_{mg} is taken as 1.0 MPa (conservative value for Grade IV surrounding rock, referring to GB50086-2015). Considering rock mass heterogeneity, construction errors, long-term creep, and other factors, the recommended embedment length of the bolt into the stable rock mass is 0.25 m. The calculated bolt length is:

$$L = l_1 + h + l_2 \quad (29)$$

where: l_1 —embedment length of bolt in stable rock mass; h —effective length; l_2 —exposed length of bolt, adopted as 0.1–0.15 m. According to the calculation, the 2.5 m-long SS bolt satisfies the length requirement.

The MQT series pneumatic leg-type bolter (Jining Dongya Industrial and Mining Materials Co., Ltd., Jining, China) is adopted for bolt construction, and the borehole diameter is 2–3 mm larger than the bolt diameter. The drilling and installation time for a single bolt is approximately 2–4 min. Bolt installation is closely linked to the drift excavation cycle. Two rows of bolts are installed after each excavation advance of 1.5–2.0 m, which brings no extra occupation of mining cycle time.

6. Numerical Simulation Analysis of Roof Support Effect

The theoretical analysis, support design, and numerical simulation in this paper form a technical route of “theoretical analysis—support design—numerical verification”. The elastoplastic zoning model is adopted in the theoretical analysis to calculate the thicknesses of the loosened zone and plastic strengthening zone of the roof surrounding rock after excavation, providing a basis for the anchorage range required by bolts. Numerical simulation is used to further evaluate the variations in roof displacement, plastic zone, and stress after support installation and verify the effectiveness of design parameters. It should be noted that the zoning thickness calculated by the theoretical model corresponds to the unsupported condition after excavation, which represents the lower limit of the required bolt anchorage range. The numerical simulation reflects the plastic zone distribution during actual mining, considering slice mining and bearing capacity of backfill. Although they are defined in different dimensions, both provide support for the support design.

6.1. Software and Computational Model

According to the stope structural dimensions, Rhino 8 combined with Griddle 2.0 was adopted to establish the geometric stope model for the UTSCF method, with hexahedral

meshes as basic elements. The spans of first-step and second-step stopes are 6 m and 8 m, respectively. Adjacent stopes are arranged in a staggered manner. The overall model dimension is $630\text{ m} \times 462\text{ m} \times 230\text{ m}$, and a 0.2 m unfilled gap layer is set at the top to simulate the non-contact filling condition. The model consists of surrounding rock, first-step and second-step stope strips of four levels, unfilled gap zones, and other components, covering a total of 96 stopes. Mesh refinement is implemented around the stopes, with a gradual transition in regions far away from the stopes; the size gradient ratio of adjacent elements is controlled within 2. The total number of zones is approximately 7.53 million. The drift slicing backfill mining method induces relatively small disturbance to surrounding rock. An element size of 0.2 m around the stope is sufficient to capture the roof failure characteristics, fully reflect the roof failure patterns and plastic zone distribution, and meet the accuracy requirements for stability analysis of stope surrounding rock. After modeling, the model is imported into FLAC3D for numerical calculation to analyze the stress distribution and deformation characteristics of the stope roof under supported and unsupported conditions. The detailed layout is shown in Figures 10 and 11.

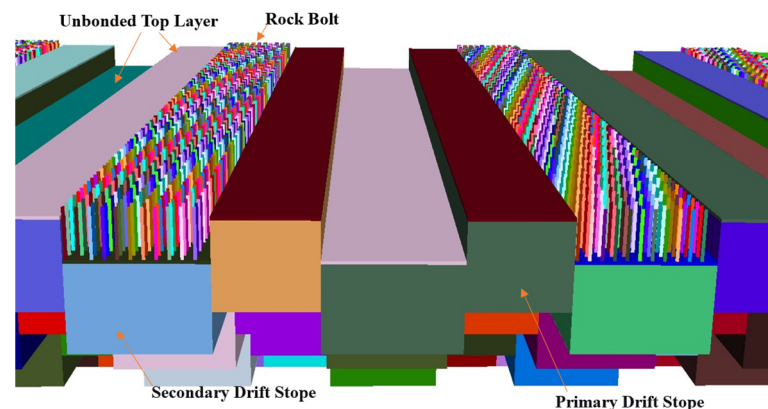


Figure 10. Numerical model of panel stope support engineering.

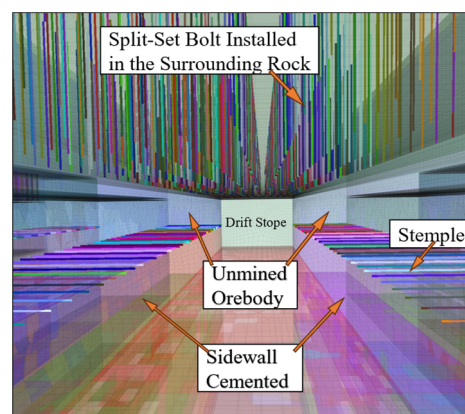


Figure 11. Schematic diagram of in-drift support engineering.

In the numerical simulation, the slotted tube bolts are simulated by cable structural elements, and the frictional contact model is adopted to describe the interaction between bolts and surrounding rock. The elastic modulus of the bolt is set to 200 GPa. The tensile strength of the bolt shank is 370 MPa according to Q235 steel specified in GB/T700-2006, and the ultimate tensile capacity of the shank is 114.7 kN. The frictional contact parameters between the bolt and surrounding rock are determined following the recommended grouting parameters for cable elements in the FLAC3D manual [57]. The friction angle is set to 25° , and the bond stiffness is 500 MN/m (to govern frictional sliding behaviour). Cross

timbers are simulated by beam structural elements. Their mechanical behaviour is axial compression parallel to the grain, with a loading mode of compression at both ends and force transfer along the member. Therefore, the beam elements are arranged vertically with each segment length of 0.2 m and placed in rows along the strike direction (the spacing between timber segments within one row is 0). Each row is equivalent to one cross timber. The beam element serves as a pressure transfer medium, contacting the surrounding rock at the top and the backfill at the bottom. The elastic modulus of cross timber is 12 GPa, density is 600 kg/m³, and Poisson's ratio is 0.3, adopted from the mechanical parameters of common timber in GB/T50005-2017. The metal mesh is laid between bolts. It functions to prevent the fall of surface loose rocks and local rock blocks, which is a superficial passive protection measure. Since it does not participate in the mechanical transfer of stress and deformation of surrounding rock, it is not modelled separately in the numerical simulation.

Normal displacement constraints are applied to the lateral boundaries and the bottom surface of the model, and a stress boundary is imposed on the top surface. The calculation proceeds following the sequence of “excavation—support—backfilling—next cycle”. Within one slice, the primary large step is mined first, followed by the secondary large step. Each large step is excavated sequentially in two small sub-steps; After excavation of the first sub-step, cross timber gap filling and slotted tube bolt support are implemented immediately, followed by backfilling. Uncemented backfill is placed below the elevation of the staggered roof on both sides, while cemented backfill is used above this elevation. During backfilling, slurry simultaneously infiltrates the non-contact roof zones on both sides to realize secondary roof contact. When excavating the second sub-step, the backfill on both sides has already contacted the roof, and only roof support and backfilling are required. The default unbalanced force ratio of FLAC3D is adopted as the convergence criterion, set to 1×10^{-5} . In the numerical simulation, stope excavation together with the installation of bolts and cross timbers is completed within a single calculation stage. That is, support is activated immediately after excavation with no time interval; The field construction adopts the organization principle of “short advance and timely roof support”. Bolts and metal mesh are installed after each excavation advance of 1.5–2.0 m. The simulation does not reproduce every excavation cycle step by step. Instead, excavation and support of each sub-step are combined into one stage to equivalently reproduce the construction effect of “immediate support after excavation”.

The Mohr–Coulomb constitutive model is adopted in the numerical simulation to describe the mechanical behaviour of ore and rock. The in situ stress field is dominated by gravitational stress. The vertical stress is calculated according to the orebody roof burial depth of 260 m and the average unit weight of overlying strata, and the horizontal stress is determined by the lateral pressure coefficient. The calculation formula for lateral pressure coefficient K_0 is as follows [58]:

$$K_0 = 1 - \sin\varphi \quad (30)$$

The densities of surrounding rock and orebody are measured from field samples. The orebody contains metal sulfides, so its density is slightly higher than that of the surrounding rock. The density of cemented backfill is 1.95 g/cm³, and that of uncemented backfill is 1.90 g/cm³, both obtained from mix proportion design and field sampling tests. Cemented backfill has a higher cement content and thus a slightly larger density than uncemented backfill. The mechanical parameters for various rock masses are listed in Table 4.

Table 4. Mechanical parameters of ore-rock mass and backfill.

Lithology	Density ρ (g/cm ³)	Tensile Strength σ /MPa	Elastic Modulus E/GPa	Cohesion C/MPa	Internal Friction Angle φ (°)	Poisson's Ratio μ
Surrounding rock	2.77	0.80	9.09	0.95	38.15	0.22
Orebody	2.85	0.65	5.09	0.86	42.56	0.26
Cemented backfill	1.95	0.25	0.51	0.37	35.00	0.28
Uncemented backfill	1.90	0.01	0.16	0.1	34.00	0.19

The roof support construction of the drift consists of drift excavation, cross-timber support for the unfilled gap layer of side-wall backfill, and SS bolt support for the roof. Two schemes, i.e., excavation without support and SS bolt support, are designed for comparison. The mechanical parameters of SS bolts and cross-timbers are listed in Tables 5 and 6.

Table 5. Mechanical parameters of SS bolt.

Elastic Modulus /GPa	Tensile Strength/MPa	Cross-Sectional Area of Bar	Ultimate Tensile Strength of Bolt Shank/KN	Borehole Diameter/mm
200	370	3.10×10^{-4}	114.7	47

Table 6. Mechanical parameters of cross-timber.

Elastic Modulus /GPa	Poisson's Ratio	Density/kg·m ⁻³	Cross-Sectional Area/m ²	Moment of Inertia About Y-Axis/m ⁴	Moment of Inertia About Z-Axis/m ⁴	Polar Moment of Inertia/m ⁴
12	0.3	600	0.04	1.33×10^{-4}	1.33×10^{-4}	2.67×10^{-4}

6.2. Analysis of Support Performance

To verify the rationality of the above-mentioned theoretical support analysis results, normal ore-body mining under unsupported conditions is taken as the comparison benchmark. Drifts at different spatial positions are selected for comparative analysis from three aspects: displacement, plastic zone, and stress distribution, so as to systematically evaluate the regulation effect of the support scheme on roof surrounding rock stability. To separately evaluate the effects of bolt support and cross-timber gap filling, the excavated drift and the non-roof-contacted drifts on both sides are intercepted for comparison: the displacement variation of the excavated drift before and after support is compared to reflect the effect of bolt support; the displacement variation of the non-roof-contacted drift before and after cross-timber gap filling is compared to reflect the effect of secondary roof contact achieved by cross timbers.

6.2.1. Displacement Field and Deformation Characteristics

As shown in Figure 12 (positive values in the legend represent upward displacement, while negative values denote downward displacement), under unsupported conditions, the sidewall backfill fails to achieve effective roof contact. After simultaneous excavation of four drifts in the first group of second-step stopes, the roof undergoes symmetric overall subsidence due to stress redistribution. The vertical roof displacement of drifts at the panel edge ranges from 0.91 cm to 4.04 cm, and that of adjacent drifts with unfilled gaps is 1.22–2.68 cm. For drifts in the central panel area, the vertical roof displacement is 1.09–5.52 cm, and the adjacent drifts with unfilled gaps show roof displacement of 2.10–5.54 cm.

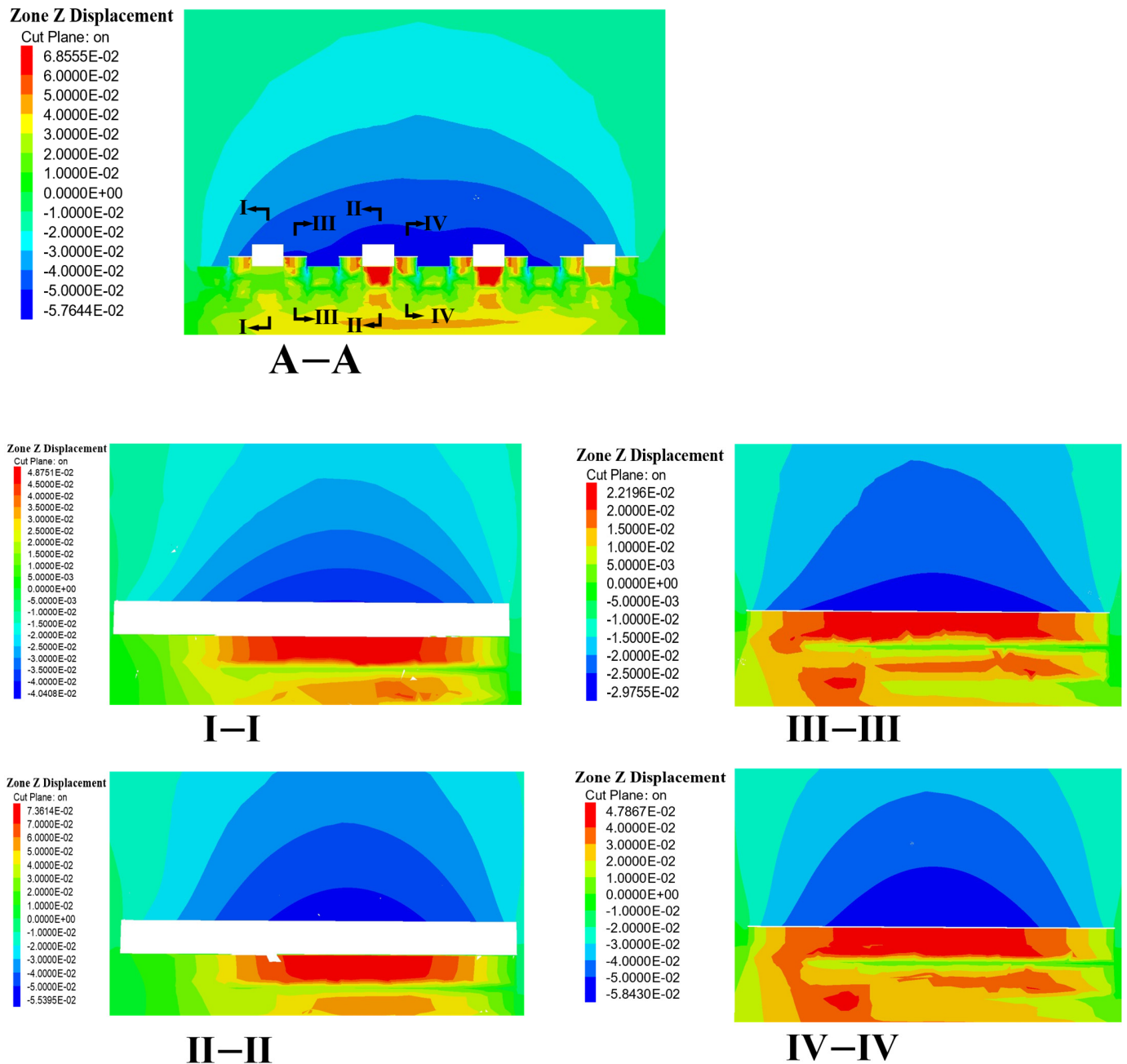


Figure 12. Displacement contour map of slope excavation under unsupported conditions. (A—A) Longitudinal section of the panel; (I—I) cross-section of a drive at the panel edge; (II—II) cross-section of a drive in the panel center; (III—III) cross-section of a drive at the panel edge not connected to the top layer; (IV—IV) cross-section of a drive in the panel center not connected to the top layer. Note: The numerical values are presented in the original FLAC3D output format, in which “E” denotes “ $\times 10^n$ ” (e.g., 8E3 = 8×10^3).

According to the displacement distribution results of supported stopes after excavation (Figure 13), after the installation of SS bolt support, the vertical roof displacement of drifts at the panel edge is reduced to 0.5–3.49 cm; the roof displacement of adjacent drifts with unfilled gaps decreases to 0.59–2.48 cm; the vertical roof displacement of drifts in the central panel area is limited to 0.51–5.01 cm; and the roof displacement of adjacent drifts with unfilled gaps is reduced to 1.14–5.16 cm.

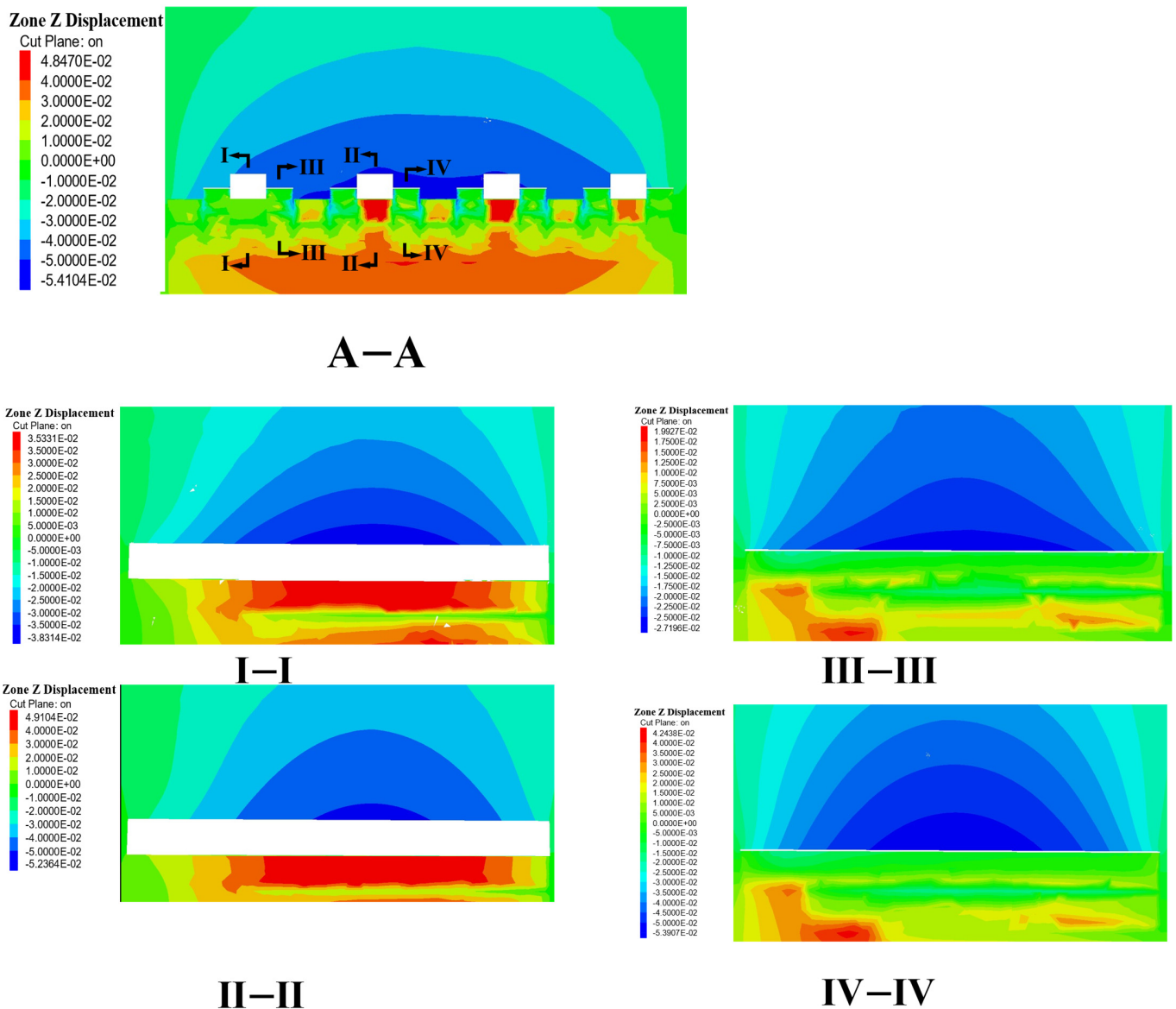


Figure 13. Displacement contour map of supported stope after excavation. (A–A) Longitudinal section of the panel; (I–I) cross-section of a drive at the panel edge; (II–II) cross-section of a drive in the panel center; (III–III) cross-section of a drive at the panel edge not connected to the top layer; (IV–IV) cross-section of a drive in the panel center not connected to the top layer. Note: The numerical values are presented in the original FLAC3D output format, in which “E” denotes “ $\times 10^n$ ” (e.g., 8E3 = 8×10^3).

As indicated by the displacement comparison (Figure 14), support implemented after drift excavation effectively mitigates roof displacement. The roof displacement of edge drifts decreases by 29.4%, and that of central drifts is reduced by 31.2%. After cross-timber support is adopted for unfilled gap layers, the roof displacement of edge drifts with unfilled gaps drops by 29.6%, while that of central drifts with unfilled gaps decreases by 26.3%. According to the vertical-section displacement contours, the area of large roof displacement zones after support is reduced by approximately 69% compared with the unsupported case. The surrounding rock deformation is effectively controlled, and roof stability is remarkably improved.

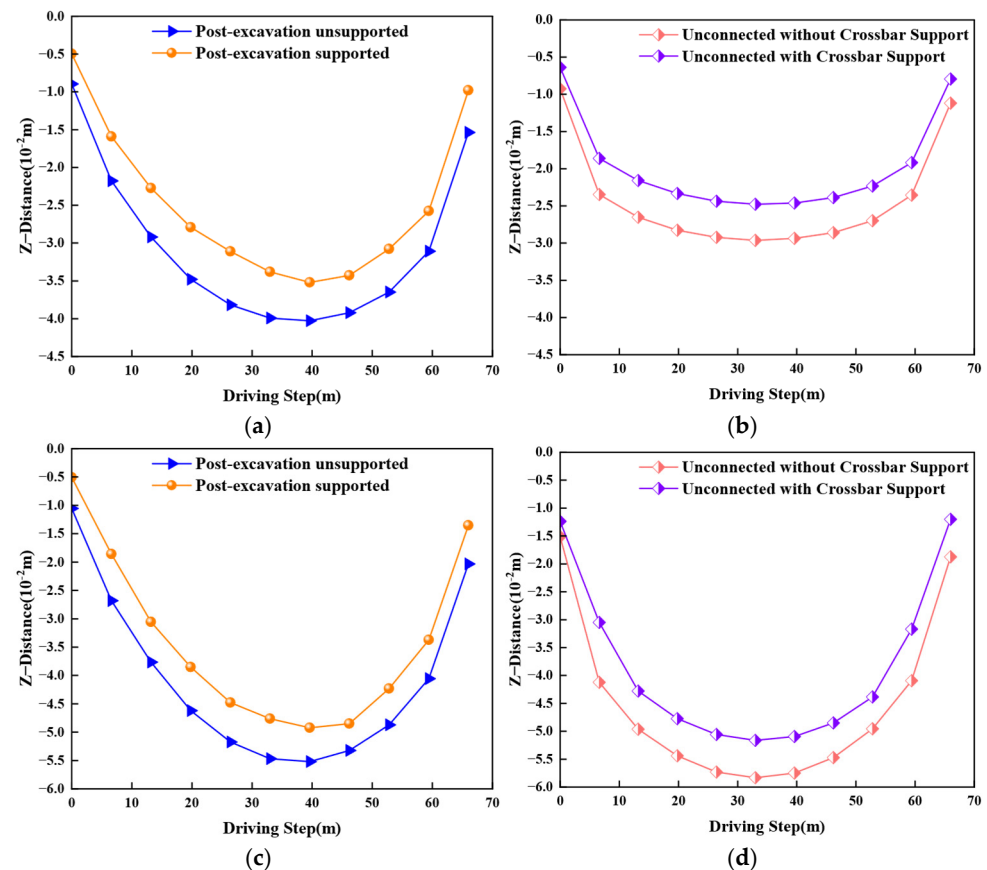


Figure 14. Comparison chart of displacement data for drifts before and after support. (a) Cross-section of a drive at the panel edge; (b) cross-section of a drive in the panel center; (c) cross-section of a drive at the panel edge not connected to the top layer; (d) cross-section of a drive in the panel center not connected to the top layer.

6.2.2. Analysis of Plastic-Zone Results

After slicing-drift mining, the plastic failure zone of the roof presents an arch-shaped distribution. The roof of the exposed void is dominated by tensile failure and coupled tensile-shear failure. As shown in Figure 15, after the first-stage excavation of second-step stopes under unsupported conditions, the side-wall backfill fails to contact the roof. The plastic failure zones of the exposed drift roof and side-wall roofs are connected to varying degrees, which leads to a large development height and extensive scope of the overall plastic zone. Affected by strong end restraint and stress concentration at drift ends, the surrounding rock in these regions fails under multi-axial stress, and the plastic-zone height is evidently larger than that at the drift middle section. The maximum height of the plastic failure zone is 1.41 m at the middle and 3.09 m at the end of drifts on the panel edge. For drifts in the panel center, the maximum plastic-zone height reaches 4.20 m in the middle and 5.62 m at the end. For unfilled-gap drifts at the panel end, the maximum plastic failure height of the roof is 4.07 m, while that for unfilled-gap drifts in the panel center is 4.59 m.

After the roof of the excavated void stope is supported by SS bolts and the unfilled gap layers are filled and supported with cross-timbers, the plastic-zone distribution is presented in Figure 16. After support installation, the plastic zones of side-wall drift roofs are decoupled from those of mining drifts, and the plastic-zone height decreases significantly. The maximum height of the plastic failure zone is 0.93 m in the middle and 1.98 m at the end of drifts on the panel edge. For drifts in the panel center, the maximum plastic-zone height is 2.65 m in the middle and 4.01 m at the end. For unfilled-gap drifts on

the panel edge, the maximum roof plastic failure height is 3.50 m, and that for unfilled-gap drifts in the panel center is 3.48 m.

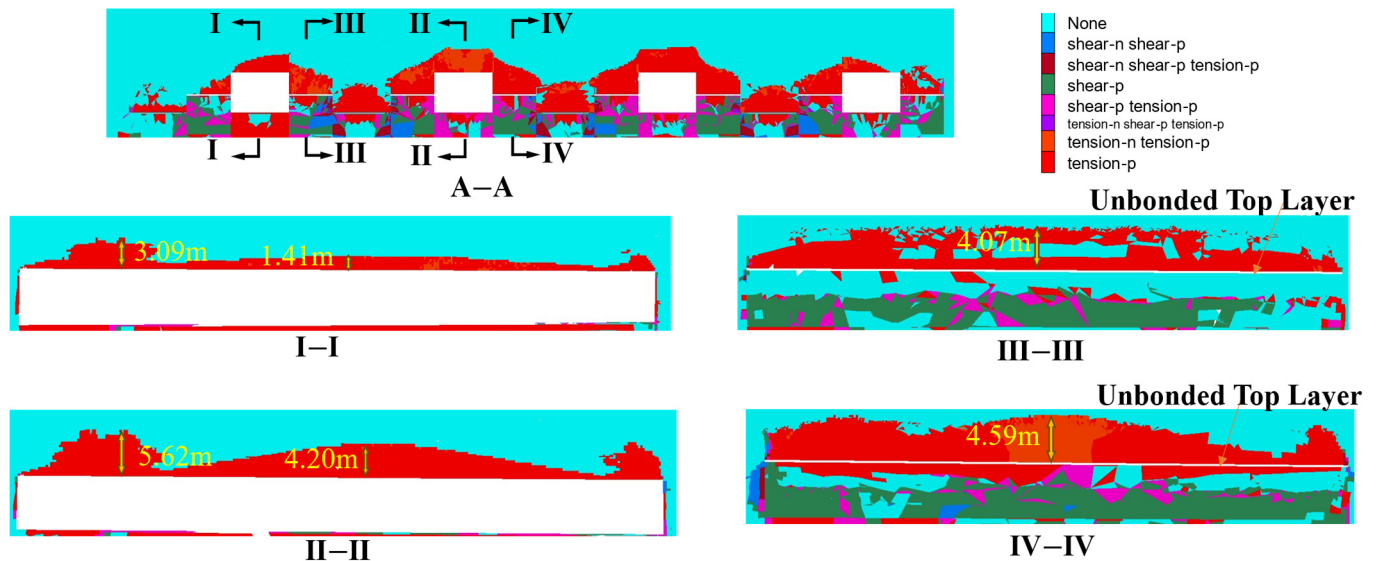


Figure 15. Plastic-zone contour map of slope excavation under unsupported conditions. (A–A) Longitudinal section of the panel; (I–I) cross-section of a drive at the panel edge; (II–II) cross-section of a drive in the panel center; (III–III) cross-section of a drive at the panel edge not connected to the top layer; (IV–IV) cross-section of a drive in the panel center not connected to the top layer. Note: The white area represents the mined-out space after stope excavation.

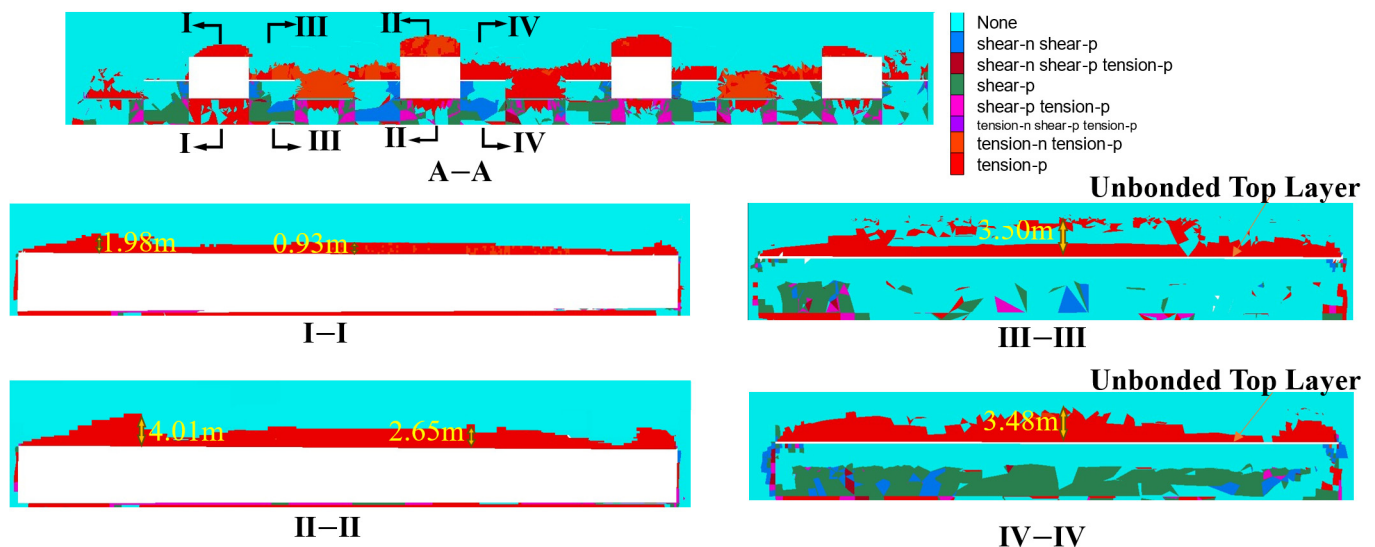


Figure 16. Plastic-zone contour map of supported stope after excavation. (A–A) Longitudinal section of the panel; (I–I) cross-section of a drive at the panel edge; (II–II) cross-section of a drive in the panel center; (III–III) cross-section of a drive at the panel edge not connected to the top layer; (IV–IV) cross-section of a drive in the panel center not connected to the top layer. Note: The white area represents the mined-out space after stope excavation.

As shown in Figure 17, after the installation of SS bolts, the maximum plastic-zone height of the roof for excavated drifts at the panel edge is reduced by 35.9%, and the plastic-zone volume of the roof decreases by 35.5%. For excavated drifts in the panel center, the maximum roof plastic-zone height drops by 28.6%, and the plastic-zone volume is reduced by 36.7%. For unfilled-gap drifts at the panel edge, the maximum roof plastic-zone height

decreases by 14%, and the plastic-zone volume is reduced by 46.5%. For unfilled-gap drifts in the panel center, the maximum roof plastic-zone height is lowered by 24.2%, and the plastic-zone volume drops by 47.2%. This indicates that the combined support scheme of SS bolts, bolt mesh, and cross-timber gap-filling can effectively restrain the deep propagation of plastic zones and guarantee the overall roof stability of grade-IV fractured rock mass.

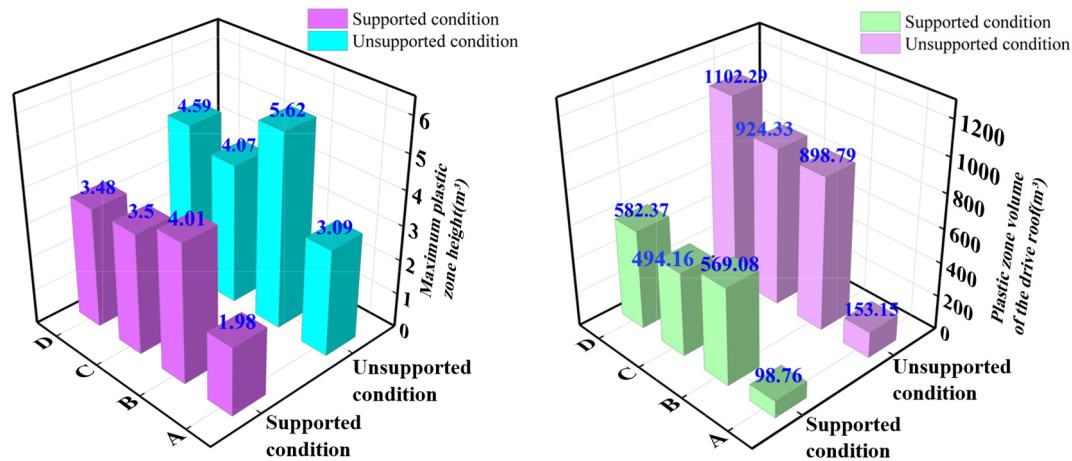


Figure 17. Summary of plastic zone data. (A) Excavated edge drive; (B) excavated central drive; (C) edge drive not connected to the top; (D) central drive not connected to the top.

6.2.3. Stress Distribution

Slicing-drift mining induces fundamental adjustment of the stress field in stope surrounding rock. The mined-out void acts as a dual zone for both stress release and stress concentration. The bearing capacity of backfill is much lower than that of the intact rock mass, so it cannot effectively support the overlying strata. Consequently, overburden loads transfer to the surrounding rock around the stope, and distinct high-pressure stress-concentration zones are formed at both sides and ends of the stope.

As shown in Figure 18, the MPS of the drift roof in the panel is tensile stress under unsupported conditions, with a peak value of 9.7 MPa. After adopting SS bolt support and cross-timber gap-filling, the MPS is remarkably reduced to approximately 5.0 MPa, a decrease of 49%. This demonstrates that the support system effectively mitigates tensile stress concentration in the roof. In terms of the minimum principal-stress distribution, the roof of unsupported drifts is also subjected to tensile stress ranging from 0 to 1.68 MPa, which places the roof in an unfavorable tensile state. After support installation, local compressive-stress zones (−1–1.53 MPa) appear on the roof of drifts at the panel edge, showing a coexistence of tensile and compressive stress. The area of tensile regions in the roof decreases substantially, and the stress state becomes more reasonable. Overall, the combined support scheme consisting of cross-timber gap-filling for roof contact and SS bolts for roof reinforcement greatly improves the stress environment of the roof. It not only reduces stress peak values, but also effectively shrinks the extent of high-stress zones, so that the overall safety conditions of the stope are obviously enhanced. Under unsupported conditions, the maximum principal stress in most areas of the goaf roof is close to 0 MPa, reaching the tensile failure threshold. The tensile stress exceeds the tensile strength of the rock mass and stress redistribution occurs; tensile stress concentration only appears in a few local positions of the roof, with a peak maximum principal stress of approximately 9.7 MPa. This peak mainly occurs near the junctions between the roof and sidewalls and end sections: after the tensile stress in the central roof exceeds the tensile strength, tensile failure takes place and stress transfers to the surrounding intact elements; In contrast, the roof at sidewalls and ends is constrained by backfill and unmined ore, which restricts

deformation and prevents free release of tensile stress. Combined with stress concentration induced by abrupt geometric changes, the tensile stress of local elements rises to 9.7 MPa. This peak value reflects local element stress concentration rather than the overall tensile stress level of the roof. After adopting split-set bolt support and cross-timber gap filling, the roof stress state is significantly improved. The peak maximum principal stress decreases to approximately 5.0 MPa, representing a reduction of 49%.

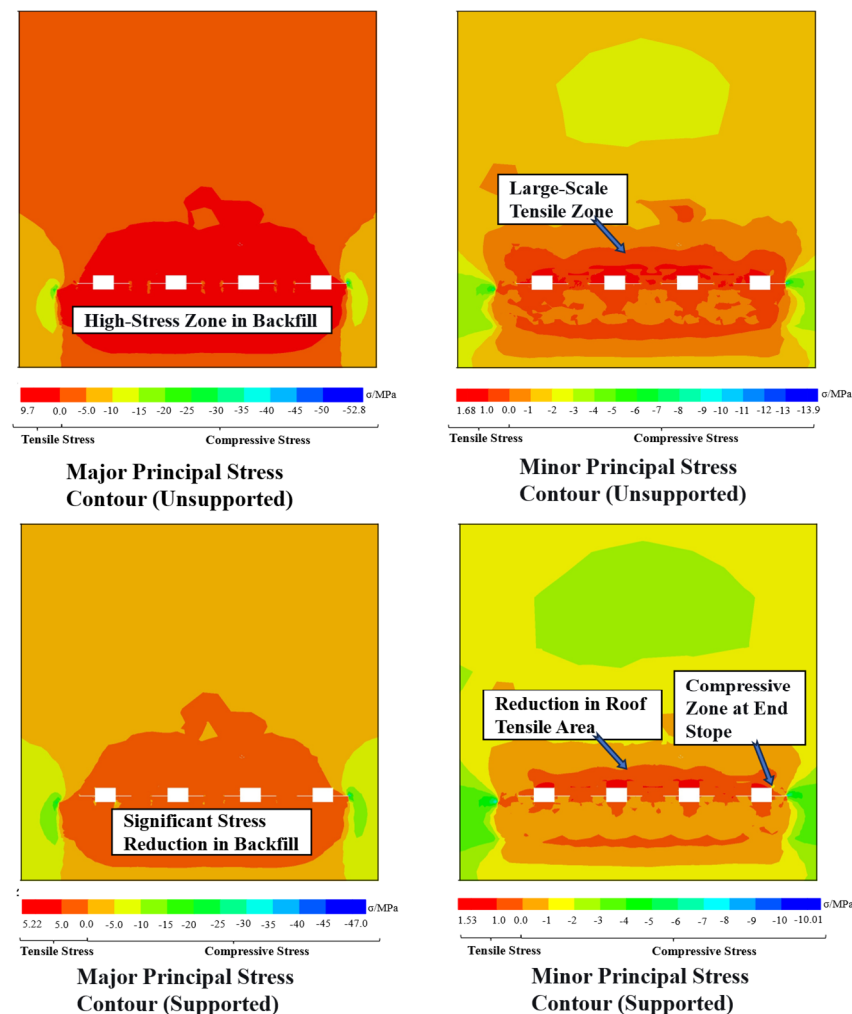


Figure 18. Stress contour map. Note: The white area represents the mined-out space after stope excavation.

7. Industrial Experiment

At present, the UTSCF method has been applied for ore extraction in stope K-1 of the 1520 m middle level in the eastern part of the first mining area of this mine. The roof support design of the stope is determined according to the aforementioned parameters, and the detailed layout is shown in the corresponding Figure 19. Bonded bolts are not adopted in the field mainly for the following considerations. Split-set bolts feature fast installation and low cost, which well match the operation rhythm of “rapid excavation and timely support” in drift mining; Bonded bolts require grouting and curing time, which will occupy the mining cycle and are not suitable for the short advance and timely support workflow of this mine. Considering the poor stability of the ore-rock mass, the mining principle of “short-advance excavation and timely roof support” is adopted during drift mining. Bolt-and-metal-mesh support is implemented immediately after each excavation cycle of 1.5–2.0 m. The SS bolts have a diameter of 42 mm, wall thickness of 2.5 mm, and

length of 2.5 m (Figure 20). Metal meshes are fixed simultaneously during bolt installation. The mesh parameters are 100 mm $\times$ 100 mm aperture and 8 mm wire diameter. Boreholes are drilled by an MQT series pneumatic leg-mounted bolter (Jining Dongya Industrial and Mining Materials Co., Ltd., Jining, China). The borehole diameter is 2–3 mm larger than the bolt diameter. After drilling, rock cuttings and accumulated water inside holes are cleared by high-pressure air. Subsequently, bolts are driven into boreholes by the impact force of the drilling machine. Anchorage capacity is provided by radial friction along the full length between bolt shank and borehole wall. The support installation is finished when the face plate closely fits the rock surface.



Figure 19. Support construction drawing.

Since the completion of mining in three slices of Stope k-1 at the 1520 level in the eastern part of the first mining area in May 2026, the roof has remained intact during drift excavation and goaf exposure without caving or bed separation. Split-set bolts feature convenient construction and immediate load-bearing upon installation, which match the high-intensity turnover rhythm of slice drift mining and bring no extra occupation of mining cycle time. For the non-contact roof gaps formed by local backfill shrinkage, cross-timber packing is adopted. After treatment, the backfill on both sides is in tight contact with the roof, and the lateral support condition of the stope is effectively maintained. Roof stress of the stope cannot be measured directly on site. Displacement monitoring requires long-term deployment of professional measuring equipment. The site adopts the workflow of “short advance with timely support and backfilling immediately after mining”. The exposure time of the goaf is short, and the site environment is not suitable for installing and maintaining monitoring equipment. Therefore, field verification is mainly based on qualitative observation, and real-time measurement techniques suitable for such operating conditions are currently being explored. Engineering practice shows that the adopted combined support scheme has favourable construction adaptability and preliminary stability control performance.

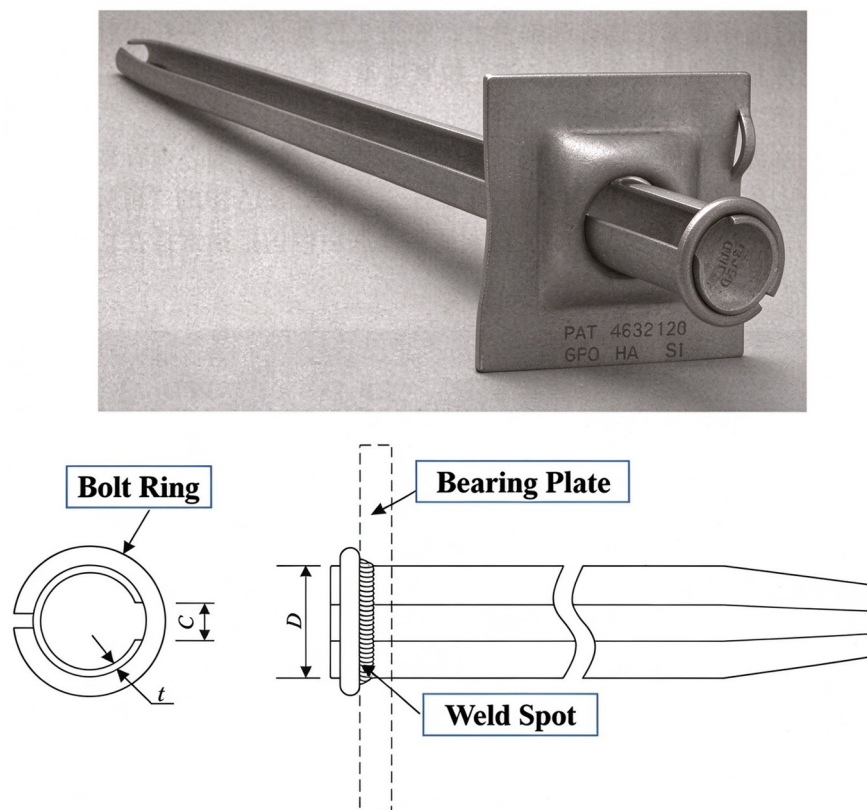


Figure 20. Split-set bolt.

8. Conclusions

1. Based on the elastoplastic analysis framework for surrounding rock of circular openings, a three-zone model consisting of “LZ-PH zone-elastic zone” for drift roof surrounding rock is established. The thicknesses of the loosened zone are determined to be 0.84 m and 0.82 m for drifts with spans of 6 m and 8 m, respectively, and the thicknesses of the plastic strengthening zone are 1.18 m and 1.33 m correspondingly. On this basis, a secondary roof-contacting scheme with cross-timber gap-filling is proposed. According to the axial compression model along the grain direction, the layout spacing of cross-timbers is determined as 1.37 m.
2. The combined support scheme of SS bolts and metal meshes is highly compatible with the “rapid excavation and immediate support” operation rhythm of drift mining. According to the composite arch theory, the bolt spacing and row spacing are determined as 0.8 m × 0.8 m. Based on the friction anchorage mechanism, the calculated bolt diameter ranges from 36 mm to 42 mm with an effective length of 2.15 m. Therefore, SS bolts of 2.5 m length can satisfy the support requirements.
3. FLAC3D numerical simulation results show that after SS bolt support and cross-timber gap-filling, the MPS of the roof decreases from 9.7 MPa to 5.0 MPa, with a reduction of 49%. The maximum plastic-zone height of each drift roof is reduced by 14.0–35.9%, and the plastic-zone volume decreases by 35.5–47.2%. The average reduction of roof vertical displacement is approximately 29%, and the area of large-displacement zones is reduced by about 69% compared with the unsupported condition. Accordingly, the safety state of the roof is effectively improved.
4. Field industrial tests show that the combined support scheme adopting $\Phi 42$ mm × 2.5 m SS bolts combined with metal meshes and cross-timber gap-filling features convenient construction and immediate load-bearing capacity, which is compatible with the high-frequency cyclic mining rhythm of sliced drift mining. Since the com-

pletion of mining of three slices in the K-1 stope of the 1520 m middle level in the eastern first mining area, the stope roof has remained intact, which preliminarily verifies the engineering adaptability and stability control performance of the proposed support scheme.

This study has the following limitations. First, the numerical simulation fails to separate the individual contribution of supporting components such as SS bolts and CTs. The combined mechanism of SS bolt support and secondary roof contact by CTs requires further investigation. Second, the theoretical zoning model calculates the zone thickness under unsupported conditions after excavation, which differs in definition and dimension from the PZ distribution at the actual mining stage in numerical simulation. The quantitative relationship between the theoretical zone thickness and support adequacy has not been independently verified. Third, restricted by the field operation condition of “excavate shortly, support promptly, fill immediately after mining”, roof stress and displacement cannot be directly measured on site. Field verification mainly relies on qualitative observation, and real-time measurement techniques suitable for such operating conditions are currently being explored.

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Abbreviations

The following abbreviations are used in this manuscript:

UTSCF method	Upward Two-Step Staggered-Drift Cut-and-Fill Mining Method
SS bolt	Split-set bolt
CT	Cross-timber

LZ	Loose zone
PZ	Plastic zone
PH zone	Plastic hardening zone
MPS	Maximum principal stress
MC	Mohr–Coulomb
FLAC3D	Fast Lagrangian Analysis of Continua in 3 Dimensions
RMR	Rock Mass Rating

References

1. Sun, G.J.; Wang, P.; Feng, T.; Yu, W.J.; Yin, Z.K.; Jiang, Y.L.; Liu, H.; Li, L.X. Deformation mechanism and control technology of surrounding rock in soft and broken roof roadway. *Coal Sci. Technol.* **2020**, *48*, 209–215. [\[CrossRef\]](#)
2. Long, G.; Wang, J.; Qiao, D.P.; Li, G.T.; Shi, R.Z. Research and application of large-span roof support technology for IV grade broken rock mass. *J. Undergr. Space Eng.* **2025**, *21*, 1328–1343. [\[CrossRef\]](#)
3. Williams, T.J.; Denton, D.K.; Larson, M.K.; Rains, R.L.; Seymour, J.B.; Tesarik, D.R. *Geomechanics of Reinforced Cemented Backfill in an Underhand Stope at the LUCKY Friday Mine, Mullan, Idaho*; National Institute for Occupational Safety and Health: Pittsburgh, PA, USA, 2001.
4. Qin, X.S.; Wang, Z.X. Simulation analysis on reinforcement effect of stope roof with combined support of anchor cable and bolt. *Nonferrous Met.* **2022**, *74*, 22–29. [\[CrossRef\]](#)
5. Zhao, X.D.; Li, H.B.; Zhang, S.J.; Yang, X. Stability analyses and cable bolt support design for a deep large-span stope at the Hongtoushan Mine, China. *Sustainability* **2019**, *11*, 6134. [\[CrossRef\]](#)
6. Blanco Martín, L.; Tijani, M.; Hadj-Hassen, F. A new analytical solution to the mechanical behaviour of fully grouted rockbolts subjected to pull-out tests. *Constr. Build. Mater.* **2011**, *25*, 749–755. [\[CrossRef\]](#)
7. Blanco Martín, L.; Tijani, M.; Hadj-Hassen, F.; Noiret, A. Assessment of the bolt-grout interface behaviour of fully grouted rockbolts from laboratory experiments under axial loads. *Int. J. Rock Mech. Min. Sci.* **2013**, *63*, 50–61. [\[CrossRef\]](#)
8. Su, F.B.; Liu, X.W.; Zhang, X.Z.; Zhang, Y.X.; Lan, X.; Yu, A.J.; Wang, J.D. Study on structural parameter optimization of stopes in upward horizontal slicing backfill mining method. *Min. Technol.* **2026**, *26*, 182–186. [\[CrossRef\]](#)
9. Wang, B.W.; Li, L.; Zhang, S.J.; Wu, S.C. Optimization of structural parameters of upward drift stope based on FLAC3D model. *China Min. Mag.* **2020**, *29*, 93–96. [\[CrossRef\]](#)
10. Ren, Y.D. Application of upward horizontal slicing step-and-fill mining method in Honghuagou Gold Mine. *Gold* **2021**, *42*, 49–53. [\[CrossRef\]](#)
11. Kilic, A.; Yasar, E.; Celik, A.G. Effect of grout properties on the pull-out load capacity of fully grouted rock bolt. *Tunn. Undergr. Space Technol.* **2002**, *17*, 355–362. [\[CrossRef\]](#)
12. Cai, Y.; Esaki, T.; Jiang, Y.J. An analytical model to predict axial load in grouted rock bolt for soft rock tunnelling. *Tunn. Undergr. Space Technol.* **2004**, *19*, 607–618. [\[CrossRef\]](#)
13. Cai, Y.; Esaki, T.; Jiang, Y.J. A rock bolt and rock mass interaction model. *Int. J. Rock Mech. Min. Sci.* **2004**, *41*, 1055–1067. [\[CrossRef\]](#)
14. Ghadimi, M.; Shahriar, K.; Jalalifar, H. A new analytical solution for the displacement of fully grouted rock bolt in rock joints and experimental and numerical verifications. *Tunn. Undergr. Space Technol.* **2015**, *50*, 143–151. [\[CrossRef\]](#)
15. Chen, J.; Liu, P.; Liu, L.; Zeng, B.; Zhao, H.; Zhang, C.; Zhang, J.; Li, D. Anchorage performance of a modified cable anchor subjected to different joint opening conditions. *Constr. Build. Mater.* **2022**, *336*, 127558. [\[CrossRef\]](#)
16. Zhao, T.B.; Xing, M.L.; Guo, W.Y.; Wang, C.W.; Wang, B. Anchoring effect and energy-absorbing support mechanism of large deformation bolt. *J. Cent. South Univ.* **2021**, *28*, 572–581. [\[CrossRef\]](#)
17. Feng, Q.; Wu, Z.; Xu, L.W.; Wang, Y.X.; Liang, Z.Q.; Wang, Z.X.; Liu, W.W.; Han, W.W.; Zhu, T.T.; Sun, S.Q. Tensile failure model and experimental study of anchorage body based on peridynamics. *Sci. Technol. Eng.* **2026**, *26*, 7437–7447. [\[CrossRef\]](#)
18. Liu, S.; He, D.; Jia, H.; Fu, M.; Hou, B. Anchoring eccentricity features and rectifying devices for resin grouted bolt/cable bolt. *Int. J. Min. Sci. Technol.* **2022**, *32*, 1059–1073. [\[CrossRef\]](#)
19. Wang, Z.Y.; Jiang, P.F.; Meng, X.Z.; Zhang, Z. Numerical study of support effectiveness and mechanism of pre-stressed bolts. *J. Min. Strat. Control Eng.* **2022**, *4*, 043015. [\[CrossRef\]](#)
20. Zou, J.F.; Wu, Q.F. Strain-softened elastoplastic analysis of surrounding rocks in the deep tunnel under pre-stressed anchor support conditions. *Chin. J. Undergr. Space Eng.* **2025**, *21*, 759–773. [\[CrossRef\]](#)
21. Chang, J.C.; He, K.; Pang, D.D.; Li, D.; Li, C.; Sun, B. Influence of anchorage length and pretension on the working resistance of rock bolt based on its tensile characteristics. *Int. J. Coal Sci. Technol.* **2021**, *8*, 1384–1399. [\[CrossRef\]](#)
22. Zuo, J.P.; Wen, J.H.; Li, Y.D.; Sun, Y.J.; Wang, J.T.; Jiang, Y.Q.; Liu, L. Investigation on the interaction mechanism and failure behavior between bolt and rock-like mass. *Tunn. Undergr. Space Technol.* **2019**, *93*, 103070. [\[CrossRef\]](#)

23. Wu, G.J.; Jia, S.P.; Chen, W.Z.; Yuan, J.; Yu, H.; Zhao, W. An anchorage experimental study on supporting a roadway in steeply inclined geological formations. *Tunn. Undergr. Space Technol.* **2018**, *82*, 125–134. [\[CrossRef\]](#)
24. Tan, L.H.; Ren, T.; Dou, L.M.; Yang, X.; Qiao, M.; Peng, H. Analytical stress solution and mechanical properties for rock mass containing a hole with complex shape. *Theor. Appl. Fract. Mech.* **2021**, *114*, 103002. [\[CrossRef\]](#)
25. Osgoui, R.R.; Oreste, P. Convergence-control approach for rock tunnels reinforced by grouted bolts, using the homogenization concept. *Geotech. Geol. Eng.* **2007**, *25*, 431–440. [\[CrossRef\]](#)
26. Luo, J.W.; Zhang, D.L.; Fang, Q.; Li, A.; Sun, Z.; Cao, L. Analytical study on pretensioned bolt-cable combined support of large cross-section tunnel. *Sci. China Technol. Sci.* **2020**, *63*, 1808–1823. [\[CrossRef\]](#)
27. Peng, W.Q.; Zhu, H.; Wang, Q. Stress distribution law of surrounding rock bearing structure of broken surrounding rock roadway. *J. Cent. South Univ. (Sci. Technol.)* **2023**, *54*, 2447–2458. [\[CrossRef\]](#)
28. Lai, Y.; Pu, Y.D.; He, R.K. Elastoplastic analysis on cave surrounding rock considering deterioration of elastic modulus. *Chin. J. Undergr. Space Eng.* **2023**, *19*, 1097–1105. [\[CrossRef\]](#)
29. Liu, G.; Xiao, Y.Z.; Zhu, J.F.; Jing, H.W. Overview on theoretical calculation method of broken rock zone. *J. China Coal Soc.* **2021**, *46*, 46–56. [\[CrossRef\]](#)
30. Li, L.M.; Tu, Y.L.; Zeng, X.R.; Xie, H.Y. Comparison research on experiments and theory of zonal disintegration belt in deep tunnel surrounding rock. *Chin. J. Undergr. Space Eng.* **2024**, *20*, 1180–1190.
31. Wu, K.; Shao, Z.S.; Qin, S.; Wei, W.; Chu, Z. A critical review on the performance of yielding supports in squeezing tunnels. *Tunn. Undergr. Space Technol.* **2021**, *115*, 103815. [\[CrossRef\]](#)
32. Cao, J.C.; Zhang, N.; Wang, S.Y.; Qian, D.; Xie, Z. Physical model test study on support of super pre-stressed anchor in the mining engineering. *Eng. Fail. Anal.* **2020**, *118*, 104833. [\[CrossRef\]](#)
33. Li, K.M.; Wang, Y.J.; Liu, K.; Li, Y.H.; Fu, Z.C.; Pang, B.X. Support mechanism of pre-stressed expandable props for stope roof in room-and-pillar mining. *J. Cent. South Univ.* **2025**, *32*, 3535–3555. [\[CrossRef\]](#)
34. Jiang, B.; Wang, M.Z.; Wang, Q.; Xin, Z.X.; Xing, X.Y.; Deng, Y.S.; Yao, L.D. Bearing mechanism of roof and rib support structure in automatically formed roadway and its support design method. *J. Cent. South Univ.* **2024**, *31*, 2467–2487. [\[CrossRef\]](#)
35. Liu, H.Y.; Zuo, J.P.; Zhang, C.W.; Wu, K.J.; Lei, B. Asymmetric deformation mechanism and control technology of roadway under room-pillar group in Huasheng coal mine. *J. Cent. South Univ.* **2023**, *30*, 2284–2301. [\[CrossRef\]](#)
36. Jin, A.B.; Liu, B.; Gao, Y.T.; Sun, H. Evaluation of safety and deformation characteristics of the secondary stope sandwiched between backfills in underground iron mines. *Bull. Eng. Geol. Environ.* **2022**, *81*, 83. [\[CrossRef\]](#) [\[PubMed\]](#)
37. Li, K.M.; Jiang, K.Y.; Li, Y.H.; Wang, X.; Liu, K.; Xu, S. Determination of the load bearing capacity of pre-stressed expandable props for ground support in underground mines. *Int. J. Min. Sci. Technol.* **2023**, *33*, 977–990. [\[CrossRef\]](#)
38. Shi, K.Y.; Wu, X.P.; Tian, Y.R.; Xie, X. Analysis of re-tensioning time of anchor cable based on new prestress loss model. *Mathematics* **2021**, *9*, 1094. [\[CrossRef\]](#)
39. Huang, G. Research on Supporting Mechanism and Application of Arch-Shell Combined Arch in Deep Roadway. Ph.D. Thesis, Henan Polytechnic University, Jiaozuo, China, 2019.
40. Meng, Q.B.; Han, X.; Huang, B.X.; Pu, H.; Wang, R.; An, G.J. Load-bearing characteristics of tiered bolt-grouting collaborative support in development and extension roadways within coal-bauxite coordinated mining zones. *J. Cent. South Univ. (Sci. Technol.)* **2026**, *57*, 1693–1707. [\[CrossRef\]](#)
41. Luo, W.; Huang, Q.X.; Fan, D.L.; Chen, S.S.; Du, J.W.; Wang, S.B.; Wang, Q.X.; Wang, W.; Guo, Q. Active forepoling of transportation roadway in shallow buried fully-mechanized mining face. *J. Min. Strat. Control Eng.* **2024**, *6*, 013037. [\[CrossRef\]](#)
42. Wang, X.; Tang, J.Z.; Li, Y.; Fu, Q. The failure law and combined support technology of roadways with weak surrounding rock in deep wells. *Appl. Sci.* **2023**, *13*, 9738. [\[CrossRef\]](#)
43. Xu, Q.; Zhang, B.S.; Guo, J.; Li, Z.; Liu, T.; Li, F.; Duan, D. Zoned support technology for advancing working-face driving roadways in ultra-close coal seams based on vertical stress evolution. *Energies* **2025**, *18*, 2657. [\[CrossRef\]](#)
44. Liu, Z.; Wen, J.L.; Zhou, X.H.; Li, S. Optimization of mechanized coordinated cut-and-fill mining method for inclined thin ore bodies. *China Saf. Sci. J.* **2025**, *35*, 118–124. [\[CrossRef\]](#)
45. Zhao, Q.C.; Fu, B.J. Study on loose zone testing and support technology of roadway surrounding rock affected by dynamic pressure. *J. Min. Strat. Control Eng.* **2020**, *2*, 023031. [\[CrossRef\]](#)
46. Kastner, H. Ein Beitrag zur Frage der Gebirgsbeanspruchung im Tunnelbau. *Der Bauing.* **1949**, *24*, 107–114.
47. Yuan, C.; Wang, W.J.; Zhao, Y.L.; Yu, W.J.; Peng, G.; Wu, H.; Tang, H. Theoretical analysis on roadway surrounding rock deformation based on the properties of rock plastic hardening and softening. *J. China Coal Soc.* **2015**, *40*, 311–319. [\[CrossRef\]](#)
48. Xia, R.M. The Theory and Numerical Analysis on Plastic Zone and Ground Arch Due to Tunnel Excavating in Bearing Soil. Master's Thesis, Beijing Jiaotong University, Beijing, China, 2008.
49. Guo, X.F.; Jin, Y.W.; Zhang, H.K. Expansion patterns of plastic zone in surrounding rock of rectangular roadway in non-uniform stress field and equivalent circumcircle analytical method. *J. Min. Sci. Technol.* **2024**, *9*, 870–883. [\[CrossRef\]](#)

50. Zhang, L.M.; Wang, Z.Q. Stress solution of pressure on tunnel liner considering nonlinear softening of rock mass post-peak. *Rock Soil Mech.* **2008**, *29*, 283–287. [[CrossRef](#)]
51. GB 50086-2015; Technical Code for Engineering of Ground Anchorages and Shotcrete Support. China Planning Publishing House: Beijing, China, 2015.
52. Jin, M.Q.; Yin, S.H. Study on top bag filling roof supporting technique in thick coal seam roadways developing along roof. *Coal Sci. Technol.* **2021**, *49*, 51–56. [[CrossRef](#)]
53. GB 50005-2017; Standard for Design of Timber Structures. China Architecture & Building Press: Beijing, China, 2017.
54. National Energy Administration. *Split set (MT/T 285-2025)*; Standards Press of China: Beijing, China, 2025.
55. Zheng, Y.T.; Zhu, F.S. Simplified mechanical model and reasonable parameters of slit wedge tubing bolt. In *Ground Pressure and Strata Control*; Energy, Mines, and Resources Canada: Ottawa, Canada, 1993.
56. GB/T 700-2006; Carbon Structural Steels. Standards Press of China: Beijing, China, 2006.
57. Itasca Consulting Group Inc. *FLAC3D Structural Elements Manual (Version 7.0) [M/OL]*; Itasca Consulting Group, Inc.: Minneapolis, MN, USA, 2019.
58. Jáky, J. Pressure in Silos. In Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, The Netherland, 21–30 June 1948; Volume 1, pp. 103–107.

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