

Article

Stability Analysis and Numerical Simulation Study of Surrounding Rock in a Large-Span Open-Off Cut of a Mine with Weakly Cemented Strata

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Abstract

To address the stability challenges of surrounding rock in large-span open-off cuts within weakly cemented strata of western China, this study investigated the 1219 open-off cut at the Shila Wusu Coal Mine. An analytical elastic model for rectangular roadway stress was developed using complex variable function theory to examine the influence of the lateral pressure coefficient on stress distribution. Furthermore, numerical simulations were employed to characterize plastic zone evolution and evaluate support effectiveness. The results demonstrate that the lateral pressure coefficient significantly dictates the stress field: circumferential stress at the ribs intensifies with the increasing lateral pressure coefficient, while stress in the roof and floor decreases accordingly. Notably, tensile stresses develop in the roof and floor when the lateral pressure coefficient is less than 1. Stress extremes are concentrated at the roadway shoulders, exhibiting a distribution pattern where the ribs experience higher concentration than the roof and floor. The circumferential stress concentration coefficient exhibits a marked positive correlation with the lateral pressure coefficient. Numerical results indicate that post-support compressive stress at the shoulders reaches 39.24 MPa, with plastic zone widths of 1.64~2.06 m at the ribs, 2.70 m at the roof, and a significant 5.33 m at the floor, highlighting a pronounced risk of floor heave. Field loosening zone measurements of 1.08 m in the roof and 2.49 m in the rib align closely with numerical findings, confirming that the implemented support effectively constrains plastic zone development. By integrating theoretical derivation, numerical modeling, and in situ observations, this study establishes a robust theoretical and technical framework for the support design of large-span roadways in similar geological settings.

Keywords: stress-elastic solution of surrounding rock; lateral pressure coefficient; circumferential and tangential stresses; concentration coefficient; loosening zone extent; width of the plastic zone



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1. Introduction

The western region of China contains over 70% of the country's coal resources. In response, the Chinese government has prioritized a strategic approach to coal resource development in the west, emphasizing safety and security of supply. This strategy aims to transform the region from a mere “energy warehouse” to a “national strategic hub”.

Consequently, ensuring the safe and sustainable exploitation of coal resources in western China is of paramount importance.

Weakly cemented sandstone is widely distributed throughout western China and is characterized by late-stage diagenesis, poor cementation, and inherently low mechanical strength [1]. This geological formation presents unique stability challenges. With the rapid advancement of mechanized and intelligent mining, the spans of open-off cuts have continuously expanded. This geometrical expansion induces profound roof subsidence, radically alters the surrounding stress environment, and exacerbates the development of stress concentration and expansive fractured zones within the roof and ribs [2–5]. Consequently, maintaining the structural integrity of large-span excavations in such fragile strata has become a paramount prerequisite for safe mining operations.

The deformation characteristics of surrounding rocks in weakly cemented sandstone formations are essentially consistent with those of soft rocks, implying that findings from soft rock mechanics are highly applicable to this geological context. Notably, the in-situ stress state dictates the deformation behavior of underground openings. Studies employing magnetic signal monitoring on pre-cracked sandstone under staged loading have elucidated the failure evolution, underscoring the controlling influence of the stress field on rock mass stability [6]. Drawing upon the “Butterfly Theory” of plastic zone evolution, Gao et al. [7] and Liu et al. [8] demonstrated that the efficacy of bolt-cable reinforcement in mitigating continuous deformation is heavily contingent upon the prevailing stress field. Furthermore, Ji et al. [9] revealed that asymmetric stress fields trigger non-uniform deformation, accelerating the structural deterioration of surrounding rock. This mechanism is further supported by research on the acoustic emission characteristics and infrared thermal effects of coal with varying burst tendencies [10,11]. To provide a rigorous quantitative framework for these stress-driven behaviors, Tan et al. [12] and Xu et al. [13] developed analytical models using complex variable function theory, successfully quantifying stress distribution under multifaceted boundary conditions. Additionally, the destructive stress ratio has emerged as a critical criterion for identifying the threshold of instability. In-depth investigations into the synergistic coupling of rock mechanical properties, regional stress fields, support stiffness, and cross-sectional geometry offer robust solutions for controlling large-scale deformation in soft rock environments [14,15].

Global research on the stability control of roadway surrounding rocks has advanced significantly, with extensive studies focusing on innovative support technologies, critical zone identification, and multi-method synergistic analysis. Zhu et al. [16] proposed the “Three Shells” synergistic support technology, leveraging stress shells to constrain plastic zone expansion and alleviate the challenges of stability maintenance. Similarly, Zhen et al. [17] demonstrated that optimizing the roof-cutting angle and implementing an asymmetric coupling support system can effectively bolster the stability of surrounding rocks in inclined coal seams. Crucially, the efficacy of reinforcement measures is fundamentally predicated on the precise identification and targeted mitigation of the loosening and plastic zones. For instance, Zhao et al. [18] utilized borehole peep imaging to delineate the loosening zone based on rock fragmentation levels, while Wang et al. [19] elucidated the deformation and failure mechanisms of surrounding rocks by integrating field monitoring with borehole imaging techniques. Furthermore, numerous researchers [20–22] have employed numerical simulations to investigate plastic zone evolution, subsequently proposing optimized support schemes to enhance roadway integrity. These collective findings establish a robust theoretical framework and practical reference for the advancement and application of stability control technologies in complex underground environments.

However, while these static support and identification frameworks are robust, they often overlook the operational reliability of mining systems under dynamic conditions,

such as blasting vibrations and impact loads. As underground environments grow more complex, modern excavation safety increasingly necessitates the integration of static design with real-time dynamic protection [23,24]. Recent advancements have introduced smart controllers for autonomous vibration auditing and specialized mitigation devices, such as shell-type dampers with helical-cut geometries, to alleviate shock loads [25,26]. These dynamic control technologies represent an essential evolution, complementing traditional support by ensuring long-term structural integrity under fluctuating load conditions.

Despite the proliferation of underground engineering studies, a critical analysis of the current literature reveals that existing research remains insufficient in three fundamental dimensions. First, most models are framed exclusively within static geomechanics, failing to integrate the dynamic effects and real-time vibration monitoring that are increasingly vital for modern excavation safety. Second, there is a prominent inconsistency between idealized rock mass theories and the actual behavior of weakly cemented strata, with a notable scarcity of research that unifies analytical derivations and numerical approaches specifically for these complex formations. Third, traditional paradigms often assume symmetric deformation, resulting in an inadequate characterization of the asymmetric instability patterns—particularly the severe floor heave—observed in large-span open-off cuts. To address these gaps, this study selects the Shila Wusu Coal Mine as a quintessential case, where the significant burial depth and characteristic weakly cemented properties trigger large-scale deformations representative of deep mining in western China. By constructing an elastic stress model based on complex variable function theory and coupling it with a triple-verification approach comprising analytical, numerical, and field data, this research elucidates the regulatory mechanisms of support structures on the plastic zone. Ultimately, these findings bridge the gap between idealized theory and engineering reality, providing a precise theoretical grounding for targeted support optimization in large-span excavations.

2. Engineering Background and Calculation Method

2.1. Engineering Background

Shila Wusu Coal Mine is situated deep within the Dongsheng Coalfield in Inner Mongolia, with mineable coal seams located at significant depths. The 1219 open-off cut is located in the No. 12 panel area, primarily targeting the 2-2 coal seam. The roof strata belong to the Yan'an Formation. The maximum burial depth of the 1219 working face reaches approximately 745 m, with an average depth of about 705.5 m. The average minable thickness of the coal seam is 5.17 m. The immediate roof consists of 8.91 m of sandy mudstone, while the immediate floor is composed of 3.47 m of sandy mudstone. The main roof comprises 11.92 m of fine sandstone, and the main floor contains 4.74 m of fine sandstone. The strata in the 1219 open-off cut are dominated by weakly cemented sandstone characterized by low compositional and textural maturity. This rock mass is rich in rigid particles and features large thickness and strong integrity, yet it exhibits low strength, easy weathering, and significant argillization upon water contact. To clarify the broader relevance of this case, XRD tests were conducted on prepared specimens, with the relative mineral contents listed in Tables 1 and 2. The results reveal a high proportion of clay minerals (37%), predominantly illite and kaolinite. Illite, with its weak hydrophilicity and low interlayer force, aggravates the brittle fracture of the rock mass, while kaolinite leads to poor inter-particle cementation and overall softening. These mineralogical characteristics and the resulting complex mechanical behaviors are not unique to the Shila Wusu mine but are quintessential attributes of weakly cemented strata in deep mining environments. Consequently, this site serves as a representative geological model for investigating the stability and reinforcement of large-span roadways under similar complex conditions.

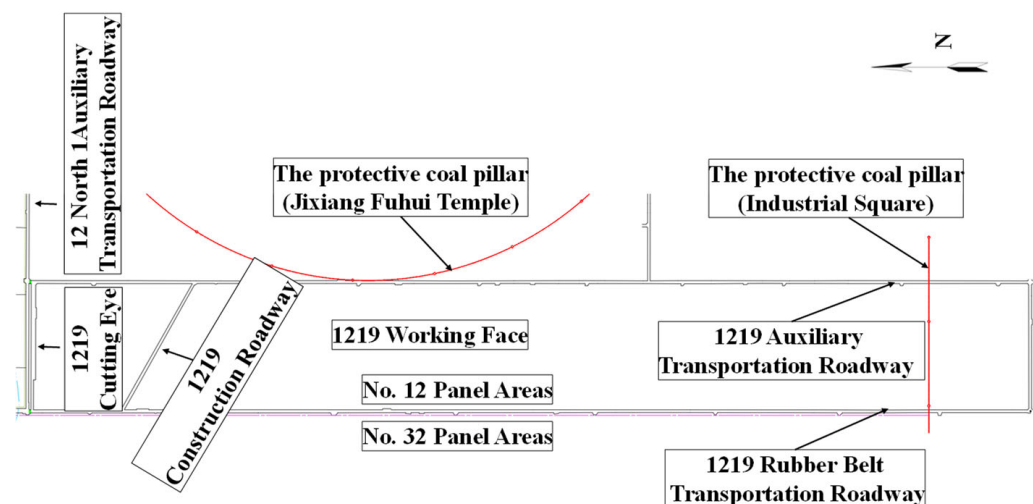
Table 1. Relative mineral content of weakly consolidated sandstone.

Mineral	Quartz	Plagioclase	Microcline	Clay
Content/%	43	12	8	37

Table 2. Clay mineral composition of weakly cemented sandstone.

Mineral Composition	S	I/S	It	Kao	C	C/S	I/S	C/S
Clay mineral content/%	-	19	19	48	14	-	5	-

The layout of the 1219 working face and open-off cut is illustrated in Figure 1. The northern boundary of the working face is marked by the 12 North 1 auxiliary transportation roadway, while the southern boundary is the protective coal pillar of the industrial square (designed stopping line). To the east lies the protective coal pillar of Jixiang Fuhui Temple, and to the west is the boundary between the No. 12 and No. 32 panel areas. The working face is oriented perpendicular to the west wing roadway, with the rubber belt transportation roadway on the west and the auxiliary transportation roadway on the east, both arranged in parallel. The specifications of the 1219 open-off cut support bolts and cables, along with the open-off cut dimensions, are shown in Figure 2. As illustrated in Figure 2, the 1219 open-off cut exhibits an aspect ratio of 2.87:1, characterizing it as a typical “large-span” roadway where width significantly exceeds height. In the weakly cemented strata of western regions, the surrounding rock’s structural self-stability is markedly inferior to that of hard roof structures. Consequently, spans exceeding 8 m present a substantial risk of roof suspension failure. With a net span reaching 11.2 m, the 1219 open-off cut far exceeds the conventional safety span threshold for such geological conditions, thus fitting the engineering definition of an ultra-large-span structure. Under the combined challenges of weakly cemented sandstone and high burial depth, maintaining the stability of the 1219 open-off cut is paramount, as it directly dictates mine safety and the roadway’s service life. Therefore, a systematic analysis of the stress state and stability of this large-span open-off cut is essential. Such work not only reveals the underlying deformation and failure mechanisms but also provides a rigorous scientific foundation for optimizing support parameters and selecting appropriate reinforcement technologies.

**Figure 1.** Layout plan of the 1219 open-off cut and working face roadway.

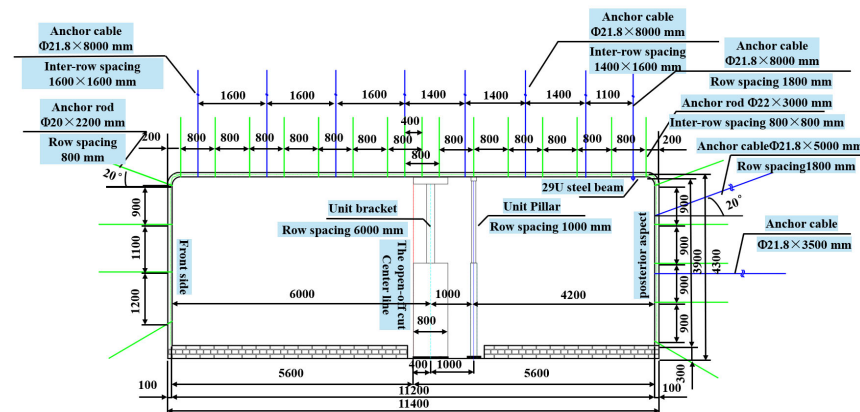


Figure 2. Cross-section diagram of 1219 open-off cut support.

2.2. Method of Calculating Elastic Solutions for Stresses in Surrounding Rock of Rectangular Roadways

To address the secondary stress redistribution in the surrounding rock following the excavation of the 1219 rectangular open-off cut, a two-dimensional plane strain model has been developed based on the theory of elastic mechanics. The model relies on the following foundational assumptions to simplify the analysis:

1. The roadway is considered to be located within an infinite planar region, where the extent of the surrounding rock is significantly greater than the dimensions of the roadway itself.
2. To isolate and reveal the fundamental influences of the in situ stress field and cross-sectional geometry on the stress redistribution of surrounding rock and to highlight the geomechanical response driven by tectonic stress, considering that the burial depth of the roadway is much larger than its cross-sectional dimension, the effect of body forces in the near-field surrounding rock is negligible compared with the far-field stress concentration. Therefore, the influences of body forces, support resistance, and other external factors are neglected in the theoretical analysis.

The model constructed under these basic assumptions is illustrated in Figure 3. In this model, P_1 represents the force acting in a vertical direction; P_2 denotes the force acting in the horizontal direction; and the lateral pressure coefficient λ is introduced such that $P_2 = \lambda P_1$.

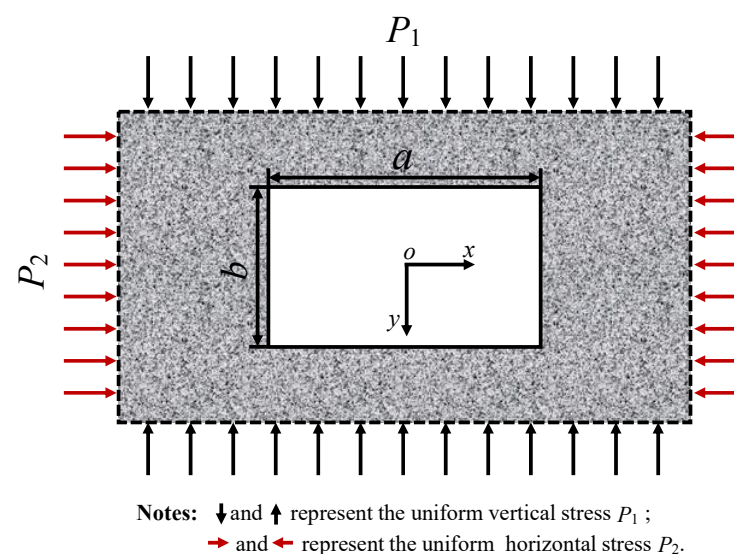


Figure 3. Simplified two-dimensional planar model of the surrounding rock of the roadway.

Based on the theory of elastic mechanics, the mechanical analysis of the rectangular roadway is simplified into a plane strain problem, with the physical configuration depicted in Figure 4. To further simplify the boundary conditions, the method of conformal mapping using complex variable functions is employed, whereby the region occupied by the rectangular roadway on the Z -plane is mapped onto the unit circle domain in the ζ -plane. Utilizing these methods, the stress-elastic solution for the surrounding rock of the rectangular roadway is derived. Due to the complexity and number of formulas involved in the derivation process, only the final result formulas are presented here, with the specific derivation process detailed as follows.

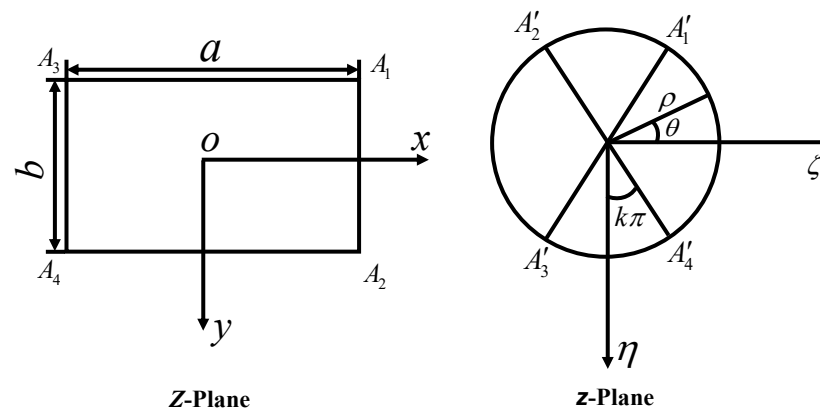


Figure 4. Conformal mapping of a rectangle boundary in the ζ -plane onto the interior of a unit circle in the Z -plane.

1. Determine the mapping function. As shown in Figure 4, the general mapping function can be obtained using the Schwarz–Christoffel conformal transformation [27], as expressed in Equation (1):

$$Z = \omega(z) = K \int_0^z \prod_{k=1}^n (z - x_k)^{(\alpha_k/\pi)-1} dz + f \quad (1)$$

In the formula, both K and f are complex variable constants that control the size, orientation and position of the polygon; K determines scaling and rotation; f determines translation; α_k is the interior angle of the polygon at the vertex (the sum of the interior angles of the polygon is $0 < \alpha_k < 2\pi$); and x_k is a known point on the real axis, e.g., $x_1 = -\infty$ or a fixed point at 0, 1, etc.

In a rectangular roadway, the z -plane rectangular orifice region is mapped to the complex plane by taking $n = 4$ and $f = 0$ [28], without considering the support resistance. Substituting the above parameters into Equation (1), the following can be obtained:

$$Z = \omega(t) = R \int_0^z (t - x_1)^{\frac{\alpha_1}{\pi}-1} (t - x_2)^{\frac{\alpha_2}{\pi}-1} (t - x_3)^{\frac{\alpha_3}{\pi}-1} (t - x_4)^{\frac{\alpha_4}{\pi}-1} dt \quad (2)$$

In the formula, R is a complex constant, and $\alpha_1, \alpha_2, \alpha_3$, and α_4 are the four top corners of the rectangle and have the value $3\pi/2$.

By the mapping condition, the complex forms of x_1, x_2, x_3 , and x_4 are $x_1 = e^{k\pi i}$, $x_2 = e^{(2-k)\pi i}$, $x_3 = e^{(1+k)\pi i}$, and $x_4 = e^{(1-k)\pi i}$, where k is the width-to-height ratio of the rectangular lane.

Substituting Equation (2) into Equation (1) and expanding the equation after substitution in the neighborhood of zeros and variable substitution for the solution of the

rectangular lane complex function, z takes the first three terms of the transformation and calculates them, resulting in the following [29]:

$$Z = \omega(\zeta) = R(\zeta^{-1} + \beta_1\zeta + \beta_3\zeta^3) \quad (3)$$

In the formula, $\beta_1 = \cos 2k\pi$, $\beta_3 = -\frac{1}{6} \sin^2 2k\pi$, and k is a constant.

The relationship between the physical coordinates (x, y) of the rectangular roadway boundary in the z -plane and the angular parameter θ in the complex mapping plane is expressed as

$$\begin{cases} x + iy = R(e^{-i\theta} + \beta_1 e^{i\theta} + \beta_3 e^{3i\theta}) \\ x = R(\cos \theta + \beta_1 \cos \theta + \beta_3 \cos 3\theta) \\ y = R(-\sin \theta + \beta_1 \sin \theta + \beta_3 \sin 3\theta) \end{cases} \quad (4)$$

In the unit-circle complex plane, the mapping function in Equation (1) is a single-valued function. Therefore, when $\theta = 0$ and $\theta = \pi/2$, the corresponding z -plane coordinates are $(x = a, y = 0)$ and $(x = 0, y = -b)$ respectively. Substituting these into Equation (4) yields the following:

$$\begin{cases} x = a = R(1 + \cos 2k\pi - 1/6 \sin^2 2k\pi) \\ y = -b = R(-1 + \cos 2k\pi + 1/6 \sin^2 2k\pi) \end{cases} \quad (5)$$

2. The complex potential function form is set according to the far-field stress and boundary condition constructors $\varphi(\zeta)$ and $\psi(\zeta)$. Based on the basic principles of elastic mechanics, the tunnel rock stress is jointly determined by two analytic functions $\varphi(\zeta)$ and $\psi(\zeta)$, and both are complex analytic functions with respect to the complex variable ζ , expressed as Equations (6) and (7):

$$\begin{cases} \varphi(\zeta) = \frac{1+u}{8\pi}(X + iY) \ln \zeta + B\omega(\zeta) + \varphi_0(\zeta) \\ \psi(\zeta) = -\frac{3-u}{8\pi}(X - iY) \ln \zeta + (B' + iC')\omega(\zeta) + \psi_0(\zeta) \end{cases} \quad (6)$$

$$\begin{cases} \Phi(\zeta) = \frac{\varphi'(\zeta)}{\omega'(\zeta)} \\ \Psi(\zeta) = \frac{\psi'(\zeta)}{\omega'(\zeta)} \end{cases} \quad (7)$$

In the formula, X and Y are the sums of the forces in the z -plane in the x and y directions, respectively, and B , B' and C' are constants related to the far-field stresses σ_x^∞ and σ_y^∞ , where $B = 1/4(\sigma_x^\infty + \sigma_y^\infty)$, $B' = 1/2(\sigma_y^\infty - \sigma_x^\infty)$ and $C' = \tau_{xy}^\infty$.

The value of the complex variable ζ bounded in the unit-circle complex plane is σ , and the symbol f_0 is introduced for computational simplicity; the expression of f_0 is shown in Equation (8). Then the conjugate equations of the Cauchy integral representation obtained by the complex functions $\varphi_0(\zeta)$ and $\psi_0(\zeta)$ using the boundary conditions are shown in Equations (9) and (10):

$$f_0 = i \int (X + iY) ds - \frac{X + iY}{2\pi} \ln \sigma - \frac{1+u}{8\pi} (\bar{X} - i\bar{Y}) \frac{\omega(\sigma)}{\omega'(\sigma)} \sigma - 2B\Gamma\omega(\sigma) - (B' - iC')\overline{\omega(\sigma)} \quad (8)$$

$$\begin{cases} \varphi_0(\zeta) + \frac{\omega(\sigma)}{\omega'(\sigma)} \overline{\varphi_0'(\zeta)} + \overline{\varphi_0(\sigma)} = f_0 \\ \psi_0(\zeta) + \frac{\omega(\sigma)}{\omega'(\sigma)} \overline{\psi_0'(\zeta)} + \overline{\psi_0(\sigma)} = f_0 \end{cases} \quad (9)$$

$$\begin{cases} \varphi_0(\zeta) = \frac{1}{2\pi i} \int_\sigma \frac{f_0 d\sigma}{\sigma - \zeta} - \frac{1}{2\pi i} \int_C \frac{\omega(\sigma)}{\omega'(\sigma)} \frac{\overline{\varphi_0'(\zeta)}}{\sigma - \zeta} d\sigma \\ \psi_0(\zeta) = \frac{1}{2\pi i} \int_\sigma \frac{\bar{f}_0 d\sigma}{\sigma - \zeta} - \frac{1}{2\pi i} \int_C \frac{\omega(\sigma)}{\omega'(\sigma)} \frac{\varphi_0'(\zeta)}{\sigma - \zeta} d\sigma \end{cases} \quad (10)$$

3. The stress components at any given point within the stress field are determined by taking the derivative of the complex potential functions. According to the fundamental principles of elastostatics, the stress distribution within the surrounding rock of a rectangular roadway aperture must satisfy Equation (11):

$$\begin{cases} \sigma_{\theta} + \sigma_{\rho} = 2[\Phi(\zeta) + \overline{\Phi(\zeta)}] = 4\text{Re}\Phi(\zeta) \\ \sigma_{\theta} - \sigma_{\rho} + 2i\tau_{\rho\theta} = \frac{2\zeta^2}{\rho^2\omega'(\zeta)} [\overline{\omega(\zeta)}\Phi'(\zeta) + \omega'(\zeta)\psi(\zeta)] \end{cases} \quad (11)$$

For the stress problem at the orifice of the surrounding rock of the rectangular roadway shown in Figure 2, considering that the burial depth of the 1219 open-off cut (705.5 m) is much larger than its cross-sectional dimensions, the gradient variation in self-weight stress near the excavation face is negligible, so the far-field stress state is regarded as a uniform constant. Furthermore, the inner boundary of the 1219 open-off cut is assumed to be a free surface, which represents the state of the surrounding rock immediately after excavation, before the support resistance takes effect. Therefore, the influence of body forces such as support resistance is neglected. Based on the above assumptions, the boundary conditions for the complex variable function are defined as follows: $\overline{X} = \overline{Y} = X = Y = 0$, $\sigma_x^{\infty} = p$ and $\sigma_y^{\infty} = \lambda p$. The boundary conditions are substituted into Formula (6), and the results obtained are substituted into Formula (8), Formula (9) and Formula (10) to derive $\varphi_0(\zeta)$ and $\psi_0(\zeta)$ for the boundary condition parameters of the analytical formula; then the results are expanded into the form of the Loren series, which is substituted into Formula (5) and Formula (11), and the relationship between the right-angled coordinates and polar coordinates ($\zeta = \rho e^{i\theta} = e^{i\theta} = \cos \theta + i \sin \theta$) is used to calculate the stress-elastic solution for the surrounding rock of the rectangular roadway.

3. Characteristics of Stress Distribution in Surrounding Rock of 1219 Open-Off Cut

3.1. Analytical Stress Analysis of Surrounding Rock in the 1219 Open-Off Cut

The average buried depth of the 1219 open-off cut is 705.5 m; the measured maximum horizontal principal stress, minimum horizontal principal stress and vertical stress are 26.16 MPa, 10.91 MPa and 17.74 MPa, respectively. The cutting hole at the 1219 working face of the Shila Wusu Coal Mine has a rectangular cross-section, with a net width and height of 11,200 mm and 3900 mm, respectively. The corresponding parameters for the above formula are $P = 17.74$ MPa, $a = 11.2$ and $b = 3.9$.

When the roadway direction is aligned parallel to the maximum horizontal principal stress, the excavation experiences minimal impact from horizontal stress, creating the most favorable conditions for roof and floor stability. Conversely, when the roadway direction is perpendicular to the maximum horizontal principal stress, it is most affected by horizontal stress, resulting in the least favorable conditions for roof and floor stability. When the roadway direction is oblique to the maximum horizontal principal stress, stress concentration occurs on one side of the roadway while stress is released on the other, leading to asymmetric deformation and damage to the roof and floor. Considering the influence of horizontal stress on the roadway, and combined with the measured in situ stress values in the 1219 open-off cut area mentioned above, the lateral pressure coefficients are defined as follow: the ratio of maximum horizontal principal stress to vertical stress ($\lambda_H = 1.47$), the ratio of average horizontal stress to vertical stress ($\lambda_a = 1.04$), and the ratio of minimum horizontal principal stress to vertical stress ($\lambda_h = 0.61$). Specifically, λ_H is chosen to simulate the “perpendicular layout,” representing the most unfavorable stress environment, where horizontal tectonic stress exerts maximum pressure on the roof and floor. Conversely, λ_h simulates the “parallel layout,” reflecting the most stable conditions

for the excavation. The inclusion of λ_a provides a baseline representing the average regional confinement. By evaluating this spectrum of coefficients, the study ensures that the stability analysis covers the full range of operational conditions—from the most favorable to the most critical—thereby providing a comprehensive reference for support design in the Shila Wusu mine and similar geological settings. The effects of different lateral pressure coefficients on the surrounding rock stress of the 1219 open-off cut are then calculated.

Previous studies have demonstrated that stress fields under varying lateral pressure coefficients exhibit axisymmetric and centrosymmetric distributions. Consequently, it is computationally efficient and theoretically sound to utilize a one-quarter model of the rectangular roadway to characterize the overall stress state, specifically within the domain of $0 \leq \theta \leq \pi/2$. The designated stress calculation path originates from the mid-rib ($\theta = 0$), passes through the roadway shoulder ($\theta = \pi/4$), and terminates at the center of the roof ($\theta = \pi/2$).

Utilizing MATLAB R2021b software, the specific coefficients for Equation (11) were determined through complex variable function theory and conformal mapping techniques. The calculated mapping function coefficients are summarized in Table 3.

Table 3. Mapping coefficients of elastic solution for surrounding rock stress of 1219 open-off cut.

Parameters	k	R	β_1	β_3
Calculated value	0.1816	4.3778	0.4169	−0.1377

Based on these parameters, the analytical solutions for the radial and tangential stress distributions of the 1219 rectangular open-off cut were derived. Figure 5 illustrates the resulting stress curves under three distinct lateral pressure coefficients, providing a comparative analysis of the roadway's geomechanical response.

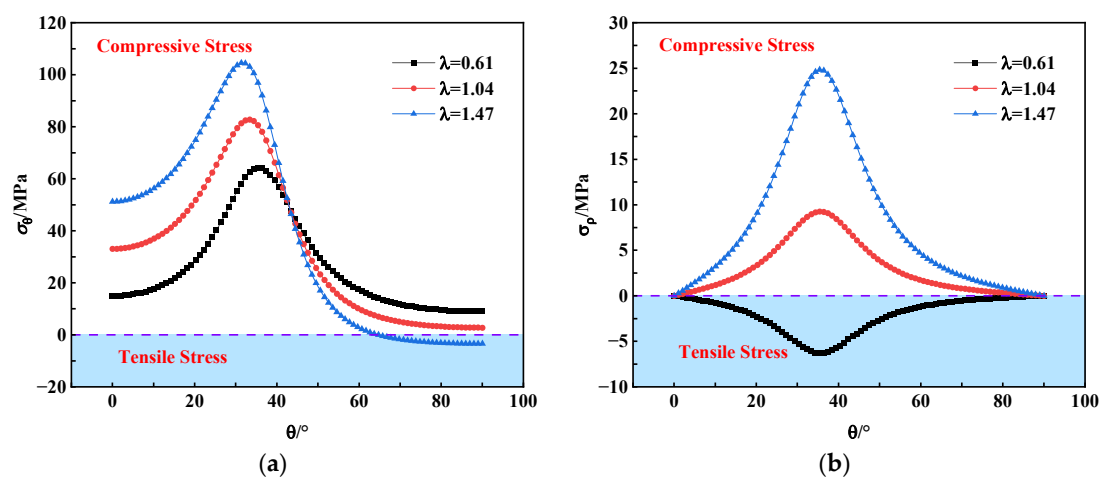


Figure 5. Curves of circumferential stress and tangential stress of 1219 open-off cut under different side pressure coefficients (note: (a) circumferential stress curve; (b) radial stress curve).

Figure 5 illustrates the distribution characteristics of circumferential stress (σ_θ) and radial stress (σ_r) around the perimeter of the 1219 rectangular open-off cut under three lateral pressure coefficients ($\lambda = 0.61, 1.04, 1.47$), where $\theta = 0^\circ$ corresponds to the rib midpoints and $\theta = 90^\circ$ to the roof or floor midpoints.

As shown in Figure 5a, the circumferential stress peaks near the shoulder corners under all three λ conditions. With the increase in λ , the peak positions shift toward the ribs, with corresponding angles of 35.39° , 33.37° , and 31.34° , respectively. The peak stress exhibits a linear positive correlation with λ , rising from 64.27 MPa to 104.52 MPa; notably,

the peak value at $\lambda = 1.47$ represents a 62.6% increase compared to that at $\lambda = 0.61$. Combined with the analysis of Figure 5b, $\lambda = 1$ is identified as the critical threshold for the surrounding rock's stress state. When $\lambda < 1$, the circumferential stress at the roof and floor mid-span remains compressive, while the radial stress across the entire roadway perimeter transitions into a tensile state, with a peak tensile stress of -6.3 MPa occurring near the shoulder corners. Conversely, when $\lambda > 1$, the circumferential stress at the roof and floor mid-span shifts from compression to tension, reaching a maximum tensile value of approximately -3.4 MPa at $\lambda = 1.47$. Simultaneously, the radial stress across the entire perimeter transforms into a compressive state, with the peak compressive stress increasing alongside λ .

Further analysis of Figure 5b indicates that although the surrounding rock is predominantly under compression, the sharp escalation in circumferential stress concentration at the shoulders exceeds the shear strength of the weakly cemented sandstone. Integrating this observation with the mineralogical data from the Shila Wusu mine regarding the 37% clay mineral content listed in Table 1, it is evident that under high lateral pressure coefficients the rock mass is highly susceptible to shear-induced butterfly expansion. Under low lateral pressure coefficients ($\lambda = 0.61$), the inherently low cementation strength and tensile capacity of the sandstone mean that the tensile radial stress exceeds the rock's cohesive limit, leading to tensile failure in the roof. Consequently, the tensile radial stress creates an extremely unstable geomechanical environment for the weakly cemented strata.

3.2. Stress Concentration Factor for 1219 Open-Off Cut

The preceding analysis reveals that the maximum stress is primarily concentrated around the roadway shoulders. This concentration suggests that the surrounding rock in these regions is highly susceptible to deformation or instability. To quantify this effect, the stress concentration factor is introduced, as defined in Equation (12).

$$K_t = \max(\sigma) / p \quad (12)$$

In the formula, σ is the circumferential stress or tangential stress (unit: MPa), p is the vertical stress (unit: MPa) and K_t is the stress concentration factor.

Based on the analytical elastic solution for the surrounding rock of the rectangular roadway, the absolute maximum values of the circumferential and tangential stress were determined. Using Equation (12), the maximum stress concentration coefficients under varying lateral pressure coefficients were calculated, as summarized in Table 4.

Table 4. Maximum stress concentration coefficient under different side pressure coefficients.

Lateral Pressure Coefficient	Circumferential Stress Factor			Tangential Stress Factor		
	$\theta_{\sigma\max}$	$(\sigma_{\theta})_{\max}$	K_t	$\theta_{\tau\max}$	$(\tau_{\theta})_{\max}$	K_t
0.61	35.39	64.27	3.62	35.39	6.31	0.36
1.04	33.37	82.78	4.67	35.39	9.27	0.52
1.47	31.35	104.52	5.89	35.39	24.84	1.40

Subsequently, a linear regression analysis was performed to establish the functional relationship between the lateral pressure coefficients and the stress concentration coefficients, which is graphically represented in Figure 6.

As illustrated in Table 2 and Figure 6, an increase in the lateral pressure coefficient causes the peak circumferential stress at the ribs to shift toward the roof and floor, with the tangential stress exhibiting a consistent positional migration. The stress concentration coefficients demonstrate a strong linear correlation with the lateral pressure coefficient.

Notably, the circumferential stress concentration coefficients are significantly higher than the tangential stress concentration coefficients. Specifically, at lateral pressure coefficient values of 0.61, 1.04, and 1.47, the circumferential stress concentration coefficients are 10.06, 8.98, and 4.21 times greater than the tangential stress concentration coefficients, respectively. This indicates that while both coefficients rise with the lateral pressure coefficient, the discrepancy between them progressively diminishes. The results suggest that as the lateral pressure intensifies, the stress state around the roadway becomes increasingly dominated by tangential components, although the circumferential concentration remains the primary factor for structural instability.

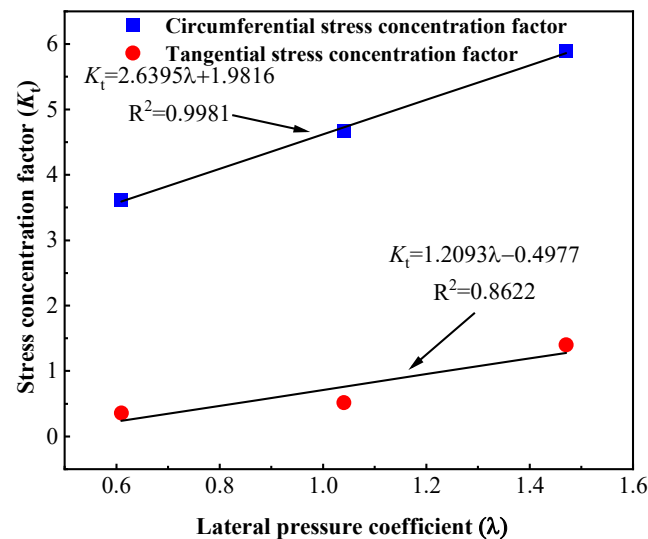


Figure 6. Relationship between the side pressure coefficient and the stress concentration coefficient.

The preceding analysis indicates that the lateral pressure coefficient significantly dictates the stress state of the surrounding rock in the 1219 open-off cut. While the stress environments remain consistent when defining the lateral pressure coefficient based on either the maximum horizontal principal stress or the average horizontal stress, a distinct stress distribution emerges when the minimum horizontal principal stress is utilized. This discrepancy underscores the necessity of a comprehensive consideration of the in situ horizontal stress field. Under all three defined coefficients, the roadway consistently exhibits stress concentration within the ribs and shoulder areas. Notably, the stress magnitude and its gradient within the ribs significantly exceed the values observed in the roof and floor. Consequently, the ribs and roadway shoulders are prioritized as critical reinforcement zones to ensure the stability of the 1219 large-span rectangular open-off cut.

3.3. Numerical Simulation Analysis After 1219 Open-Off Cut Support

The analytical results derived from the complex variable function theory provide a fundamental understanding of how the lateral pressure coefficient dictates the elastic stress distribution around the 1219 open-off cut. However, the theoretical model primarily characterizes the initial stress-driven failure trends under idealized elastic conditions, which cannot fully account for the nonlinear post-peak behavior of the rock mass or the complex interaction between the surrounding rock and the support system. To bridge the gap between theoretical stress patterns and practical engineering response, a comprehensive numerical simulation is necessary. This transition allows for the integration of site-specific parameters into a dynamic framework that evaluates staged excavation and support effectiveness. Consequently, the following section employs numerical modeling to quantify the

plastic zone evolution and deformation characteristics, thereby validating the theoretical critical zones and optimizing the reinforcement parameters for actual service conditions.

3.3.1. Establishment of the Numerical Calculation Model and Boundary Condition Settings

The model of the 1219 open-off cut is shown in Figure 7, with three-dimensional dimensions of 60 m × 60 m × 52 m; the cross-section of the 1219 open-off cut is rectangular, with a clear height of 3.9 m and a clear width of 11.2 m. The relevant rock mechanical parameters reduced for numerical simulation are listed in Table 5. The numerical calculation adopts the Mohr–Coulomb failure criterion, and the model’s boundary conditions are as follows: horizontal constraints are applied to the front, rear, left, and right boundaries; vertical constraints are applied to the bottom; and the top boundary is free. To simulate the self-weight stress of the 640 m overlying strata above the model, an equivalent self-weight stress of 16 MPa was applied at the top. Considering measured in situ stress data and the impact of maximum principal stress on the tunnel, the lateral pressure coefficient was set to 1.47. During the analysis process, comparisons were made with lateral pressure coefficients greater than 1. The 1219 open-off cut was excavated in 5 m cycles. After the first 5 m excavation, another 5 m was excavated. Once 10 m had been excavated, the first 5 m section was supported, and this process was repeated. A total of 12 excavation cycles were performed, resulting in 60 m of excavation. Subsequent analyses utilized the maximum deformation recorded during excavation.

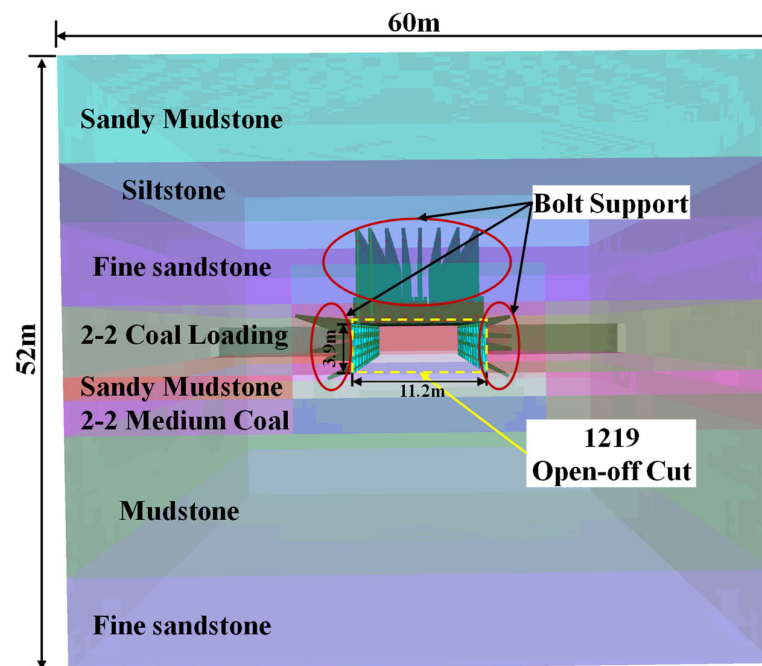


Figure 7. Numerical calculation model of the 1219 open-off cut.

Table 5. Physical and mechanical parameters of rock mass of 1219 open-off cut roadway.

Rock Designation	Thickness (m)	Density (kg/m ³)	Cohesive (MPa)	Friction (°)	Tensile Strength (MPa)	Modulus of Elasticity (GPa)	Poisson's Ratio
Sandy mudstone	9	2387	3.5	36.5	0.75	6.07	0.20
Siltstone	5	2219	3.0	38.0	0.60	19.50	0.22
Fine sandstone	7	2593	3.2	37.0	1.29	20.00	0.23
2-2 Coal loading	6	1430	1.0	28.0	0.50	1.02	0.36
Sandy mudstone	2	2387	3.5	36.5	0.75	6.07	0.20
2-2 Medium coal	3	1430	1.0	28.0	0.50	1.02	0.36
Mudstone	12	2500	4.0	34.0	0.70	7.80	0.24
Fine sandstone	8	2593	3.2	37.0	1.29	20	0.23

3.3.2. Numerical Analysis of Stress Distribution in the Surrounding Rock of the 1219 Open-Off Cut

The stress distribution within the surrounding rock of the 1219 open-off cut, as derived from the aforementioned support scheme, is illustrated in Figure 8. The results indicate a pronounced concentration of compressive stress within the four roadway shoulders, which closely aligns with the analytical elastic solution for a lateral pressure coefficient of 1.47. This high degree of correlation validates the consistency between the numerical simulations and theoretical calculations. Specifically, the maximum principal stress reaches 39.24 MPa, peaking at the shoulder corners of the roof and floor. Conversely, the minimum principal stress peaks at 12.99 MPa, predominantly distributed along the ribs of the open-off cut.

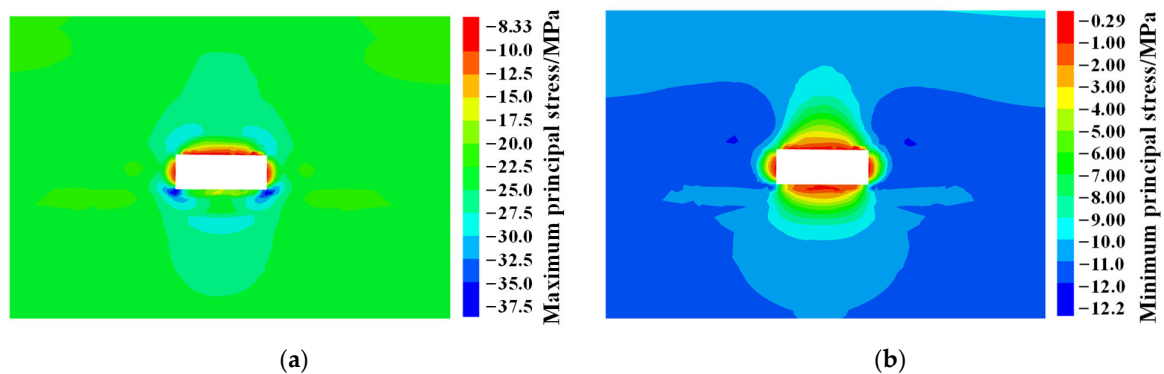


Figure 8. Stress distribution patterns within the overlying rock strata (note: (a) maximum principal stress; (b) minimum principal stress).

3.3.3. Numerical Investigation into the Deformation Evolution of Surrounding Rock in the 1219 Open-Off Cut

Utilizing the aforementioned support configuration, the deformation characteristics of the 1219 open-off cut are illustrated in Figure 9. The results reveal a slight asymmetry in the rib displacement, with the maximum deformation of the left rib (35.19 mm) marginally exceeding that of the right rib (34.92 mm); however, the right rib exhibited a more extensive displacement zone. Notably, the deformation of the floor was significantly more pronounced than that of the roof. The maximum floor heave reached 51.88 mm, substantially higher than the peak roof subsidence of 34.18 mm. Furthermore, the floor displayed a broader range of influence in terms of displacement distribution compared to the roof, indicating that the floor stability is a critical concern for this large-span roadway.

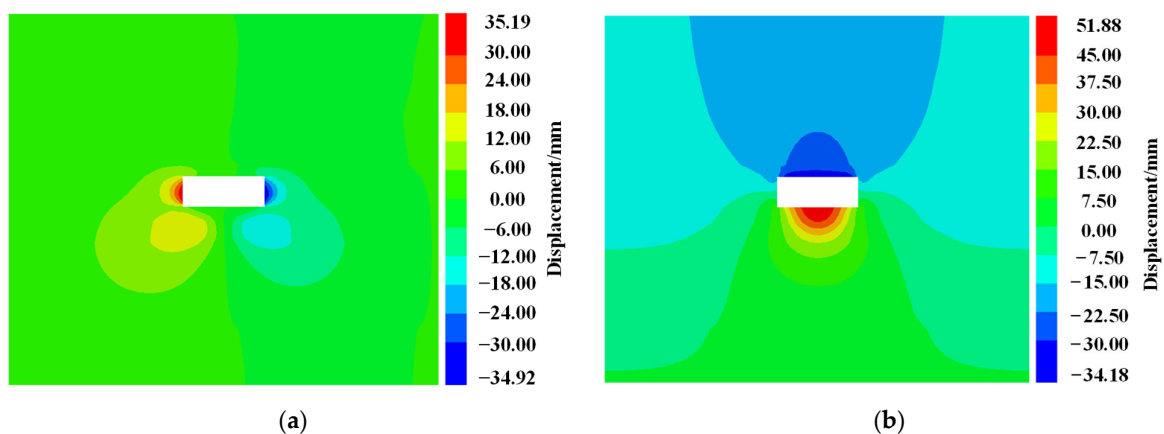


Figure 9. Deformation characteristics and evolution of the surrounding rock mass (note: (a) horizontal displacement; (b) vertical displacement).

In practical engineering, displacement monitoring instruments are typically not installed directly at the working face during the excavation of an open-off cut. Instead, the deformation of the surrounding rock was analyzed from the access tunnel adjacent to the open-off cut face. Two monitoring stations were established at distances of 50 m and 100 m from the starting point of the open-off cut to record the displacement of the left and right ribs, roof, and floor. The field-monitored data were compared with numerical simulation results to generate deformation–time curves or deformation-versus-excavation-distance curves, as illustrated in Figure 10.

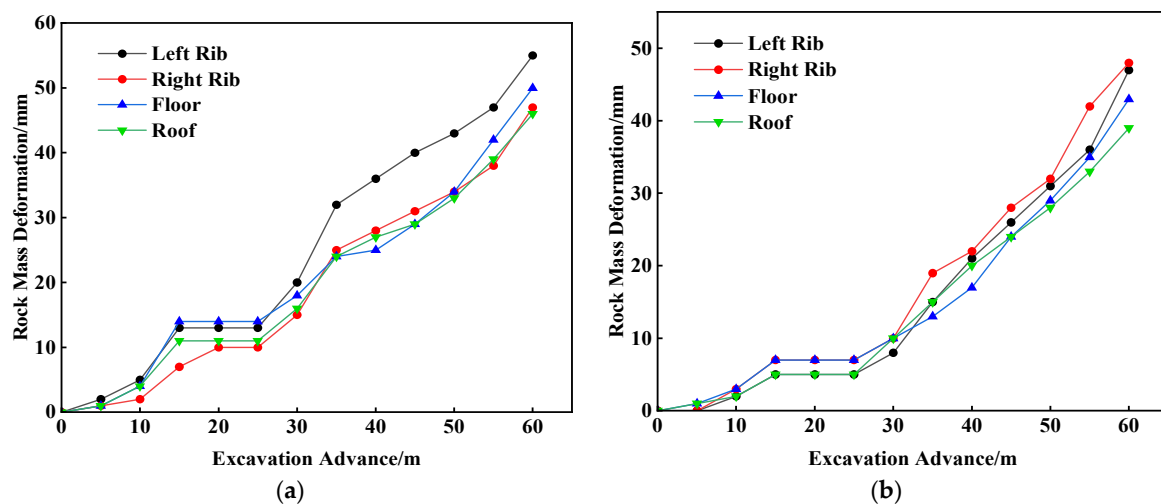


Figure 10. In situ deformation monitoring of the load-bearing rock mass (note: (a) 50 m from the 1219 open-off cut; (b) 100 m from the 1219 open-off cut).

3.3.4. Development and Distribution of the Plastic Zone in the 1219 Open-Off Cut

The plastic zone evolution of the surrounding rock in the 1219 open-off cut was analyzed, with numerical results presented in Figure 11. To accurately characterize the geomechanical response, the Mohr–Coulomb (M–C) strength criterion was employed. This selection is justified by its ability to effectively capture the pressure-dependent shear strength of the weakly cemented sandstone, which is a fundamental factor for large-span excavations. Notably, while the theoretical analytical model provides an elastic stress baseline, the numerical model builds upon this by quantifying the actual elasto-plastic extent. The elastic solutions serve as the “mechanical precursors,” identifying initial stress concentration zones that act as the primary drivers for subsequent failure. The results show that under this integrated framework, the open-off cut mainly undergoes compressive-shear failure, concentrated in the ribs, roof, and floor. Combined with theoretical calculations, when the lateral pressure coefficient $\lambda > 1$, although local tensile stress exists in the central roof and floor span, the shoulder corners and ribs remain under high tangential compressive stress. The overall stress state remains dominated by compression and shearing. Consequently, the local tensile stress identified in the theoretical analysis does not dictate the primary failure mode; instead, the compressive-shear characteristics revealed by numerical simulations remain in good agreement with the theoretical stress distribution. As shown in Figure 11, the plastic shear zones in the ribs (1.64 m and 2.06 m) exhibit approximately symmetric distribution. However, the plastic zone depth reaches 2.70 m in the roof and significantly escalates to 5.33 m in the floor. This consistency in the asymmetric expansion patterns across both the “theoretical baseline” and “realistic numerical environment” confirms that the far-field in situ stress field remains the dominant driver of stability, regardless of the complexity of the material constitutive model. These findings indicate that while

primary support mitigates rib and roof failure, the resulting stress redistribution intensifies the risk of floor heave.

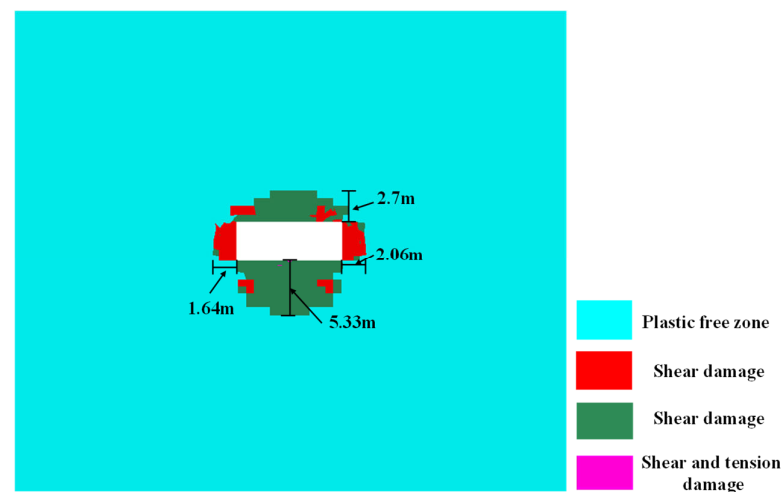


Figure 11. Evolution and distribution of the plastic zone in the surrounding rock.

To validate the theoretical framework, a systematic comparison was conducted between the analytical solutions and numerical simulation results. Despite the inherent differences in their physical completeness—specifically the simplified assumptions of neglecting body forces and support resistance in the analytical model—the two approaches demonstrate a high degree of consistency in characterizing the geomechanical response of the roadway.

The analytical model serves as a “mechanical baseline,” successfully isolating the fundamental impact of far-field stress and roadway geometry on stress redistribution. It identifies the critical stress concentration zones and the theoretical “worst-case” plastic zone morphology. In parallel, the numerical model, which incorporates staged excavation, specific rock mass parameters, and support installation, provides a refined quantitative verification. The comparative data show that the peak stress locations and the asymmetric evolution patterns of the plastic zone—specifically the expansion toward the “shoulders” and “ribs” under varying lateral pressure coefficients—remain consistent across both models.

This consistency justifies the simplified assumptions of the analytical model; it indicates that the far-field in situ stress field remains the dominant driver of stability in deep, weakly cemented strata, while secondary factors like body forces and support resistance, although influencing local magnitudes, do not alter the fundamental failure mechanisms. The transition from the “pure” theoretical model to the “realistic” numerical environment thus creates a robust, multi-level validation loop, ensuring the reliability of the stability control strategies proposed in this study.

3.4. Field Measurement Analysis of 1219 Open-Off Cut Surrounding Rock's Loosening Circle Range

In situ borehole TV inspection was conducted near the 1219 open-off cut to delineate the loosening zones within the surrounding rock. The investigation focused on the roof and the right rib, with the imaging results summarized in Figure 12. As illustrated in Figure 12a, horizontal cracks were identified in the roof at a depth of 0.23 m from the borehole collar, followed by vertical fractures at 0.43 m, and secondary horizontal cracks at 1.08 m. Beyond the 1.08 m mark, the rock mass appeared intact with no discernible fracturing. Consequently, the extent of the roof loosening zone is determined to be approximately 1.08 m. As illustrated in Figure 12b, the structural evolution of the surrounding rock in the ribs exhibits

distinct spatial differentiation. Within the 0~2.023 m range, the coal wall is disturbed yet remains largely stable. A significant intensification of fracturing is observed in the 2.023~2.491 m interval, characterized by pronounced fracture development and block detachment that result in a highly irregular borehole contour; this serves as a clear indicator of the core failure zone induced by stress concentration. Beyond the 2.49 m threshold, however, the borehole integrity recovers significantly. Although some circumferential bedding textures are visible at a depth of 6.29 m, the borehole remains circular and intact without macro-interconnected cracks, representing a nearly complete zone. Consequently, the radius of the fractured zone (loose circle) in the ribs is determined to be approximately 2.49 m.

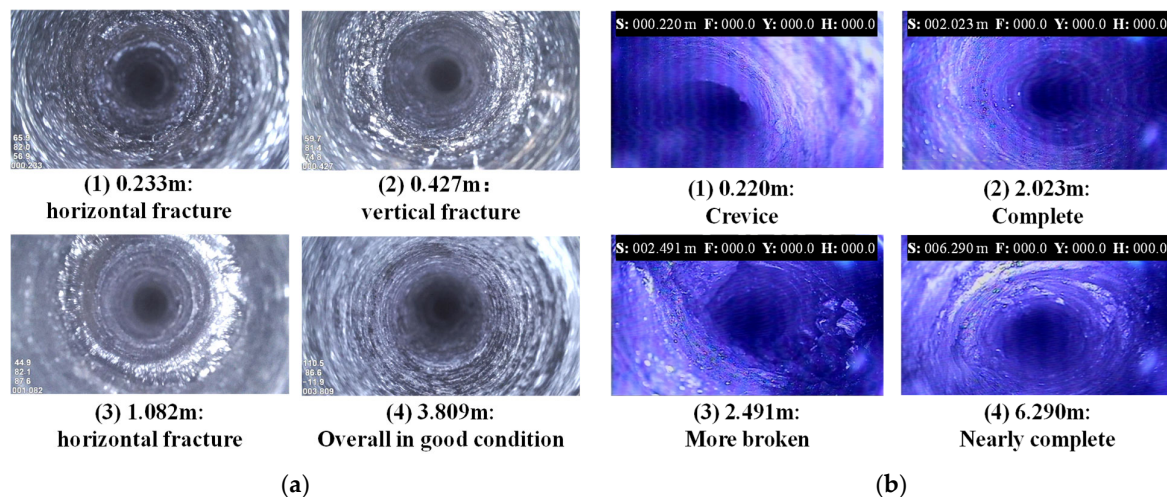


Figure 12. In situ investigation of the loosening zone within the surrounding rock mass of the 1219 open-off cut (note: (a) borehole inspection of the roof loosening zone; (b) borehole inspection of the right rib loosening zone).

According to the field measurements, the loosening zones of the roof and the right rib reach 1.08 m and 2.49 m, respectively, in their unsupported states. Based on the established relationship between the loosening and plastic zones [30,31], the latter encompasses both the fractured loosening zone and the plastic bearing zone; thus, the spatial extent of the plastic zone invariably exceeds that of the loosening zone.

Numerical simulations indicate that the post-support plastic zone in the roof is 2.70 m, implying that the corresponding loosening zone must remain below this threshold. The measured loosening zone of 1.08 m further corroborates the consistency between the numerical models and field observations. Additionally, the unsupported loosening zone of 2.49 m in the right rib suggests an even larger initial plastic zone. However, post-support simulations reveal that the rib's plastic zone is reduced to 2.06 m. Given the inherent hierarchical relationship between these zones, the post-support loosening zone in the rib is consequently restricted to less than 2.06 m. This comparative analysis demonstrates that the enhanced support configuration effectively constrains the expansion of plastic zones and mitigates fracturing within the surrounding rock.

4. Discussion

The stability control of large-span roadways in weakly cemented strata, such as the 1219 open-off cut at the Shila Wusu Coal Mine, remains a significant challenge in rock mechanics due to the low cementation strength and high sensitivity to stress redistribution. Our theoretical results, derived via conformal mapping, support the working hypothesis that the lateral pressure coefficient (λ) is the fundamental driver of boundary stress transi-

tions. Specifically, a distinct positive linear correlation is identified between parameter a and the stress concentration coefficient λ , with the latter reaching a peak value of 10.06; this correlation provides a clear mechanistic explanation for the prevalent “shoulder-cracking” failure phenomenon observed in rectangular roadway excavations. The proposed analytical solution shows excellent agreement with classical elastic theory and further establishes a more rigorous, quantitatively precise analytical framework for rectangular roadway excavations in weakly cemented coal-measure strata across Western China.

The numerical simulations further elucidate the “asymmetric instability” mechanism of the surrounding rock. While the roof subsidence was controlled at 34.18 mm, the floor heave reached a disproportionate 51.88 mm. This discrepancy suggests that while the current bolt-anchor support effectively “bridges” the roof and ribs, the floor remains an unconfined boundary susceptible to deep-seated plastic flow, reaching a depth of 5.33 m. These numerical simulation and field monitoring results are in excellent agreement with prior peer-reviewed studies on weakly cemented coal-measure strata, wherein floor instability has been widely demonstrated to act as the critical failure-controlling element that dominates the overall stability of large-span rectangular roadway systems. The high degree of correlation between our simulated plastic zones and the field borehole TV inspections (loosening zones of 1.08 m in the roof and 2.49 m in the ribs) validates the effectiveness of the support scheme in constraining rib expansion while simultaneously highlighting the urgent need for floor reinforcement.

Crucially, the Shila Wusu Coal Mine serves as a representative paradigm for the “deeply buried, weakly cemented” geological environments prevalent across western China, such as the Ordos and Ningdong basins. Mineralogical analysis (Tables 1 and 2) reveals that the rock mass possesses an exceptionally high clay mineral content (37%), dominated by kaolinite (48%) and illite/smectite mixed layers (19%). This high clay-to-quartz ratio is a hallmark of western China’s coal-measure strata, leading to the characteristic poor cementation and high water sensitivity observed in numerous regional mines, including the Bulianta and Yingpanhao mines. Therefore, the geomechanical responses and failure patterns identified at the 1219 site are not isolated phenomena but reflect the universal mechanical “fingerprints” of weakly cemented soft rock. Furthermore, the methodological framework proposed in this study—integrating complex variable function theory with hierarchical numerical validation—is inherently adaptable. By varying the lateral pressure coefficient and rock mass parameters within the analytical model, the results can be generalized to a broad spectrum of similar geological settings. The consistency between our theoretical “worst-case” predictions and the “realistic-case” numerical outcomes underscores that the far-field in situ stress remains the dominant driver of instability, regardless of minor local variations in body force or support resistance.

From a broader perspective of deep underground mining and rock engineering, the robust findings from our analytical, numerical and field validation collectively demonstrate that for large-span roadway excavations in weakly cemented soft rock masses, a dynamic, fully integrated, full-life-cycle support design framework must be systematically implemented. However, it should be noted that these conclusions are primarily contextualized within the specific geomechanical environment of the 1219 roadway. The inherent variability of coal-measure strata across different basins means that future research should focus on generalizing these analytical models to include a broader spectrum of roadway geometries and more complex geological settings, such as high-stress or fault-impacted zones.

5. Conclusions

This study investigated the geomechanical stability and support efficacy of the 1219 large-span open-off cut using analytical modeling, numerical simulation, and field monitoring. The main conclusions are as follows:

- (1) By transforming the rectangular roadway into a unit-circle domain, a stress-elastic solution was derived. The results indicate that the lateral pressure coefficient (λ) determines the stress state; $\lambda > 1$ leads to increased rib compression, while $\lambda < 1$ risks tensile failure in the roof and floor.
- (2) Numerical and field data confirm that the roadway shoulders and floor are the primary zones of instability. Under the current support configuration, the floor's plastic zone extends to 5.33 m, resulting in a significant floor heave of 51.88 mm that far exceeds the roof subsidence of 34.18 mm.
- (3) The implementation of the high-strength bolt-anchor support successfully constrained plastic development in the ribs to within 2.06 m. The strong correlation between analytical predictions, simulated plastic envelopes, and borehole TV inspections validates the reliability of the proposed stability assessment.
- (4) Based on the calculated maximum plastic zone depth of 2.06 m and the determined loose circle of 2.5 m in the ribs, it is recommended that rib bolt lengths be extended to 2.5 m to ensure they anchor into the stable deep rock mass. Furthermore, in response to the stress concentration and asymmetric expansion identified under high-lateral-pressure conditions, the layout density of the roof anchor cables and the thickness of the shotcrete layer have been appropriately increased. These adjustments are specifically guided by the theoretical stress redistribution patterns, providing enhanced surface constraint for the roadway. This approach ensures that the primary support components are grounded in quantitative evidence, while the auxiliary measures are practically optimized to suit complex mining conditions.

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