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Quande WEI

University of Emergency Management

Quanjie ZHU

zhqj2016@ncist.edu.cn

University of Emergency Management

Haoyuan LIU

University of Emergency Management

Guangyu YANG

CCTEG Coal Mining Research Institute Co., Ltd

Xiaoguang SHANG

CCTEG Coal Mining Research Institute Co., Ltd

Yi LIU

Information Institute of the Ministry of Emergency Management

Qiling HAO

Yima Coal Industry Group Co., Ltd

Article

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Posted Date: October 16th, 2026

DOI: <https://doi.org/10.21203/rs.3.rs-10941959/v1>

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Additional Declarations: No competing interests reported.

Deformation Mechanism and Dual-Arch Synergetic Support in Large-Span Roadways under Multi-Seam Dynamic Loading

WEI Quande¹, ZHU Quanjie^{1,*}, LIU Haoyuan¹, YANG Guangyu^{2,3,4}, SHANG Xiaoguang^{2,3,4}, LIU Yi⁵, HAO Qiling⁶

(1. School of Emergency Technology and Management, University of Emergency Management, Langfang Hebei 065201, China; 2. CCTEG Coal Mining Research Institute Co., Ltd., Beijing, 100013, China; 3. CCTEG Coal (Xi'an) Mining Engineering Technology Co., Ltd., Xi'an, Shaanxi, 710000, China; 4. Coal Mining & Designing Department, Tiandi Science & Technology Co., Ltd., Beijing, 100013, China; 5. Information Institute of Ministry of Emergency Management, Beijing 100029, China; 6. Yima Coal Industry Group Co., Ltd., Yima 472300, China)

Abstract: To address severe surrounding-rock deformation in large-span dynamic-pressure roadways under shallow-buried closely-spaced coal seam mining, the No. 9 seam 914 panel entries of a typical mine are taken as the prototype. A combined analytical–numerical–similarity-test and in-situ monitoring framework is employed to investigate the evolution of mining-induced stress, the surrounding-rock deformation mechanism, and dual-arch synergetic support. (1) The lateral abutment pressure exhibits a single-peak distribution with $\sigma_{\max} \approx 12\text{MPa}$ at 16 m from the goaf boundary; advance abutment pressure peaks 14–20 m ahead of the coal wall. (2) Within the Rudnicki–Rice strain-localization framework, a rigorous displacement-vector-direction mutation criterion is derived, yielding $\theta_c \approx 27^\circ$ and a stress threshold of 15 MPa. (3) A "failure-depth / arch-stiffness matching" principle $K_i / K_o \in [0.1, 0.3]$ is introduced to construct a "bolt-mesh inner arch + anchor-cable outer arch" synergetic bearing structure. (4) Six-month monitoring at 15 stations shows that the optimized scheme reduces maximum roof subsidence from 236 mm to 89 mm (-62.3%, $p < 0.001$), two-rib convergence from 178 mm to 75 mm (-57.9%) and deep roof bed-separation from 32 mm to 6 mm (-81.3%).

Keywords: dynamic-pressure roadway; displacement-vector mutation criterion; strain localization; dual-arch synergetic support; multi-seam mining; in-situ monitoring.

Introduction

The stability control of surrounding rock in roadways under mining-induced disturbance is a core issue for safe and efficient coal mining. With the widespread application of large-mining-height fully mechanized caving and close-distance multi-seam mining technologies, the surrounding rock of large-span roadways is persistently exposed to a complex environment characterized by superimposed multi-source dynamic loads and frequent stress-path reconstruction^[1–3]. This easily induces roof separation, shear slip, and structural instability. Traditional support theories based on static load assumptions are therefore inadequate for effectively guiding engineering practice^[4–6].

In terms of abutment pressure behavior, Wilson^[7] established the theory of the yield core in coal pillars based on the limit equilibrium principle. Liu Kai et al.^[8] revealed the spatiotemporal evolution of abutment pressure in an isolated island working face under down-

dip mining through theoretical analysis and FLAC^{3D} numerical simulations. Peng^[9] and Mark^[10] systematically summarized the coupling laws between surrounding rock movement and abutment pressure under longwall mining conditions in the United States. Zhang et al.^[11] showed through numerical simulation that the abutment pressure ahead of a fully mechanized top-coal caving face follows an approximately parabolic distribution. Tang Long et al.^[12] investigated the evolution and distribution characteristics of mining-induced stresses in surrounding rock near folded structures using numerical simulation and field monitoring, and proposed a roof pre-split blasting pressure relief technique. Gao Zhaoning et al.^[13] revealed the asymmetric deformation and failure mechanism of the surrounding rock in roadways within inclined strata. Jiang et al.^[14] simulated the abutment pressure distribution ahead of a coal mining face with UDEC, clearly identifying the formation mechanism of the stress concentration zone ahead of the face. Li

¹ Corresponding Author: ZHU Quanjie, E-mail: zhqj2016@ncist.edu.cn

Author Biography: WEI Quande (born 1985), male, professor-level senior engineer, Ph.D., primarily engaged in research on the prevention and control of mining dynamic disasters. E-mail: wfwqd@126.com.

Funds: This work was supported by the Hebei Natural Science Foundation (E2023508021, E2024508004), the Key Science and Technology Program of the Ministry of Emergency Management (2024EMST070702), the Fundamental Research Funds for the Central University (3142021002; 3142024005), the Langfang City Science and Technology Plan Project (No. 2024011079).

Zhenhua et al.^[15] simulated the full-cycle stress-displacement evolution of the roof under different support intensities and confirmed significant differences in roof stress distribution characteristics under varying support intensities. Du et al.^[16] focused on the advanced abutment pressure distribution of a working face with a narrow coal pillar, revealing through numerical simulation and field monitoring its stress evolution characteristics along the mining direction, and pinpointing the peak stress location and influence range. The key stratum theory proposed by Qian Minggao^[17] laid the foundation for analytically modeling the lateral abutment pressure in shallow-buried fully mechanized top-coal caving faces. Regarding stability criteria for surrounding rock, Han Zijun et al.^[18] analyzed the failure characteristics of roadways under the combined action of principal stress magnitude, direction, and layered structure, and established a roadway stability analysis model that comprehensively considers the principal stress vector and layered structure. The bifurcation theory of strain localization, developed by Rudnicki and Rice^[19] uses the acoustic tensor singularity as the onset criterion for shear bands, providing a rigorous mathematical framework for quantitative analysis of instability in elastoplastic materials. Li et al.^[20] focused on the influence of surrounding rock heterogeneity on tunnel stability and proposed a method to evaluate stability based on the distribution characteristics of the plastic zone radius and weak interlayers using an elastoplastic analytical solution. Kaiser et al.^[21] extended this approach to rockburst-prone roadways. Griffiths and Lane^[22] introduced the displacement-mutation criterion for assessing the stability of engineering slopes and underground excavations using the strength-reduction method. Song Bo et al.^[23] used the area TEDI (enclosed by the time-energy density curve and the time axis) as a discriminant index for evaluating blasting vibration effects, and established a quantitative relationship between TEDI and the damage degree of the roadway surrounding rock. Wang Weijun et al.^[24] investigated the plastic zone expansion patterns of roadway surrounding rock under different stress dominance types, revealing the secondary failure mechanism of roadway surrounding rock under the action of advanced abutment

pressure. However, the systematic application of the above theories to large-span roadways under dynamic pressure and their direct correlation with field-observable displacement vector fields remain insufficiently studied.

In terms of support theory, the surrounding rock strength reinforcement theory for rock bolting established by Hou Chaojiong^[25] laid the foundation for rock bolting in China. Kang Hongpu^[26] proposed the concept of high pre-tensioned powerful support, establishing a support system that coordinates the pre-tensioned stress field with the composite arch. He Manchao^[27] developed the constant-resistance large-deformation compensation support principle using NPR cables. Zuo Jianping^[28] proposed the mechanical theory of equal-strength support for deep roadways. Meng Qingbin et al.^[29] applied rock rheology theory to analyze the time-dependent behavior of bolt-grouting support structures in deep roadways, constructing a rheological element that reflects the deformation characteristics of the bolt-grouting support structure. Jia^[30,31] focused on support technologies for soft-rock roadways under the natural caving mining method and proposed an active-passive collaborative support system based on the confining pressure recovery and reinforcement theory. Yin^[32] systematically investigated the deformation characteristics and support principles of soft rock roadways under high in-situ stress conditions, concluding that a rigid-flexible combined support mode can effectively adapt to high-stress soft rock conditions. Zhao Zhiqiang et al.^[33] proposed a butterfly-shaped plastic zone control method. A comprehensive analysis shows that existing work remains insufficient in the following three aspects: (1) a lack of mechanical criteria for identifying instability precursors from the displacement field perspective; (2) the bolt-mesh-cable combined support mechanism is mostly described qualitatively, lacking a quantitative framework based on failure depth and stiffness matching; and (3) under multi-seam dynamic pressure conditions, there are too few four-scale coupled validations (theoretical, numerical, physical, and field measurement) of support design.

To address the above issues, this paper uses the large-span headgate of the No. 9 coal seam in a certain

mine as the engineering background and employs theoretical analysis, numerical simulation, physical modeling, and field measurements. The main innovations are as follows: ① Strictly within the Rudnicki – Rice framework, a criterion for surrounding rock instability based on the abrupt change in displacement vector direction is derived, and its analytical relationship with the direction of the principal plastic strain and the acoustic tensor singularity is established; ② Based on the Hou–Kang support system, the principle of "failure depth – arch stiffness matching" is proposed, and a double-arch cooperative load-bearing structure along with a quantitative design method is developed; ③ Statistical validation of the support effectiveness is provided using field monitoring data from 15 monitoring stations over a period of 6 months.

1 Engineering Background and Geological Conditions

Table 1: Main Physical and Mechanical Parameters of Coal-Rock Mass

Rock layer	Unit weight/(kN·m ⁻³)	Elastic modulus/GPa	Poisson's ratio	Internal friction angle/(°)	Cohesion/MPa	Compressive strength/MPa
Medium-grained sandstone (key stratum)	26.5	18.2	0.22	38	4.8	72
Mudstone	24.8	6.4	0.27	29	1.6	23
Fine-grained sandstone	25.6	12.8	0.24	34	3.5	48
Sandy mudstone (immediate roof)	24.2	5.2	0.26	28	1.2	18
Coal seam No. 9	14.0	2.8	0.30	24	0.8	20
Coal seam floor (siltstone)	25.8	10.5	0.25	32	2.4	42

Hydraulic fracturing measurements show that at a burial depth of 250 m, $\sigma_v \approx 6.2\text{MPa}$, $\sigma_H \approx 7.8\text{MPa}$, and $\sigma_h \approx 3.1\text{MPa}$. The lateral pressure coefficient $\lambda = (\sigma_H + \sigma_h) / (2\sigma_v) \approx 0.88$, exhibiting typical characteristics of shallow depth and moderate lateral pressure.

2 Distribution Characteristics of Abutment Pressure in the Working Face

2.1 Analytical Model of Lateral Abutment Pressure

Based on Qian Minggao's key stratum theory^[17], after mining a single face in a shallow, nearly horizontal coal seam, overburden movement occurs in units of strata groups, where the key stratum controls the movement of its corresponding strata group. The lateral

The study is situated within the engineering context of the 913 and 914 working faces in the 91 mining area of a specific mine. Face 913 is the first mining face, with a strike length of 1530m, a dip length of 250m, a coal thickness of approximately 15m, and a burial depth of 250m. It adopts fully mechanized top-coal caving with a mining height of 3.5m and top-coal drawing of 11.5m. Face 914 is located to the east of Face 913; its upper and lower entries are arranged along the floor, with cross-sections of 5.0~5.5m×3.3~3.5m, representing typical large-span roadways that retain top coal. The coal seam dip angle is 3°~8°, and the geological structure is gentle. Above the No. 9 coal seam, the strata are, in ascending order: 2.5m of sandy mudstone, 6.8m of fine-grained sandstone, 4.2m of mudstone, 3.5m of the No. 4 coal seam (already mined), 13m of medium-grained sandstone, and a 30m interbedding of sandstone and mudstone. The physical and mechanical parameters of the main rock layers are listed in Table 1.

abutment pressure σ on the coal body on the gob side consists of the self-weight stress σ_q and the stress increment $\Delta\sigma$:

$$\sigma = \sigma_q + \Delta\sigma \quad (1)$$

Let the thickness of the i -th key stratum be M_i , the distance from the center of the key stratum to the floor be H_i , the half-width of the gob be I , and the breaking angle be α . Then the overhanging length is $L_i = 2I + 2H_i \cot \alpha$. The maximum abutment pressure transmitted from the overhanging part of each key stratum to the coal body on the gob side exhibits an isosceles trapezoidal distribution:

$$\sigma_{\max,i} = \frac{\gamma L_i M_i}{2H_i \cot \alpha} \quad (2)$$

By superimposing the stress increments from the

overhanging parts of n key strata, we obtain:

$$\Delta\sigma(x) = \sum_{i=1}^n \sigma_{\max,i} f(x, L_i, H_i) \quad (3)$$

Where $f(x, L_i, H_i)$ is the shape function of stress distribution at a distance x from the gob boundary (isosceles trapezoidal decay, as shown in Fig.1). Taking $I=120\text{m}$, $\gamma = 25 \text{ kN/m}^3$, $\alpha = 85^\circ$, $H=250\text{m}$, and $M_I=13.0\text{m}$, substituting into Eqs. (1) - (3) yields: a lateral abutment pressure peak of 12 MPa, a peak distance of 16m from the gob, and an influence range of 34m.

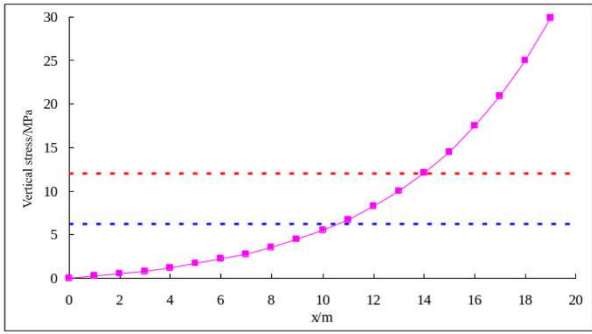


Fig.1 Isosceles Trapezoidal Distribution of Lateral Abutment Pressure

2.2 Advance abutment pressure in the strike direction

Based on Wilson's limit equilibrium theory^[7], the distribution of abutment pressure along the direction of the coal rib in the limit equilibrium zone is given by:

$$\sigma_y(x) = \left[(c_0 \cot \varphi_0 + q) e^{2fx/M} - c_0 \cot \varphi_0 \right] \frac{1 + \sin \varphi}{1 - \sin \varphi} \quad (4)$$

The peak position x_0 is obtained by solving $d\sigma_y/dx = 0$ and $\sigma_{y1} = k\gamma H$:

$$x_0 = \frac{M}{2f} \ln \frac{k\gamma H(1 - \sin \varphi) / (1 + \sin \varphi) + c_0 \cot \varphi_0}{q + c_0 \cot \varphi_0} \quad (5)$$

Taking $M=15\text{m}$, $H=250\text{m}$, $\varphi_0 = 18^\circ$, $c_0=0.4\text{MPa}$, $k=2.5$, $q=0$, and substituting into Eqs. (4) - (5), we obtain: $x_0 \approx 14.2 \text{ m}$, a peak value of approximately 12 MPa, and a pressure relief zone width of about 10 m. Combined with the estimation based on the periodic weighting interval (approximately twice the weighting step), the advance abutment pressure peak for Working Face 914 is located 14~20m ahead of the coal wall, with an influence range of 28~40m (as shown in Fig.2).

Compared with the peak distance of 22~26 m observed by Peng^[9] in the Powder River Basin, USA, the peak distance in this project (16m) is significantly

smaller. The reasons are as follows: the height of the fully mechanized top-coal caving gob (approximately 15m) is greater than that in the Powder River Basin (7~9m), resulting in a lower position of the key stratum and a shorter stress transmission path; and the stiffness of the medium-grained sandstone key stratum (18.2GPa) is significantly higher than that of the sandy mudstone (approximately 8GPa), leading to a more compact stress concentration.

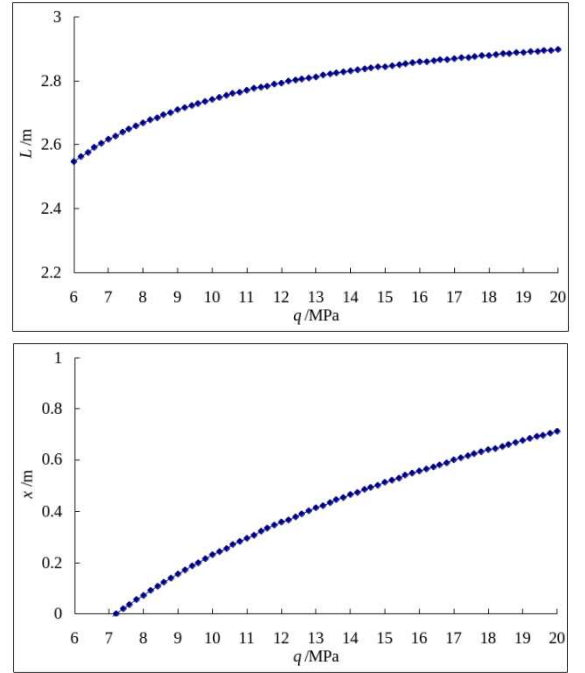


Fig.2 The evolution of disturbance zone length L and plastic zone width x_p with vertical stress q

3 Deformation Mechanism of Surrounding Rock in Large-Span Roadways Subjected to Dynamic Pressure

3.1 Failure Zone of Surrounding Rock and Numerical Model

Based on the Drucker-Prager yield criterion, assuming a horizontal coal seam of thickness $2h$, vertical stress q , horizontal stress λq , roadway half-width b , cohesion c_0 and friction angle φ_0 at the interface between the coal seam and the roof/floor, and support resistance P_0 , a closed-form solution for the disturbed zone length L can be obtained as:

$$L = \frac{h}{2 \tan \varphi_0} \ln \frac{(\lambda q + c \cot \varphi) \frac{1 - \sin \varphi}{1 + \sin \varphi} - q + \frac{2c \cos \varphi}{1 + \sin \varphi}}{P_0 + c \cot \varphi} \quad (6)$$

Plastic zone width:

$$x_p = L - \frac{h}{2 \tan \varphi_0} \ln \frac{q + \frac{2c \cos \varphi}{1 + \sin \varphi}}{P_0 + c \cot \varphi} \quad (7)$$

Taking $\mu = 0.26$, $\lambda = 0.5$, $\varphi = 35^\circ$, $c = 1.5 \text{ MPa}$, $\tan \varphi_0 = 0.1$, $c_0 = 0.1 \text{ MPa}$, $h = 1.75 \text{ m}$, $b = 2.75 \text{ m}$, $P_0 = 0.2 \text{ MPa}$, $q = 6 \sim 20 \text{ MPa}$, and substituting into Eqs. (6) and (7): when q increases from 6 MPa to 20 MPa, L increases from 2.34 m to 2.90 m, and x_p increases from 0.08 m to 0.70 m, exhibiting a nonlinear growth.

A two-dimensional plane strain model was established using FLAC^{3D}, with dimensions of 60 m × 60 m and 9,840 elements. An equivalent vertical load (6~21 MPa, in increments of 3 MPa per step) was applied to the top boundary to simulate different levels of mining-induced stress. The roadway cross-section is 5.5 m × 3.5 m. The mechanical criteria follow a layer-adapted approach: the Drucker–Prager criterion is used in theoretical derivations, while the Mohr–Coulomb criterion is used in numerical calculations. The two criteria are strictly equivalent when circumscribed to the triaxial compression meridian. After parameter reduction, the deviation between the analytical and numerical results is less than 8%.

3.2 Evolution Characteristics of the Displacement Field of the Surrounding Rock

The simulation results (Fig.3) show that the crown region exhibits dominant settlement, the two sidewalls display inward extrusion toward the roadway, the arch shoulder exhibits dilatancy, and the floor heave is relatively small. As the load increases, the displacements in each deformation zone grow nonlinearly, and the influence range extends deeper into the surrounding rock. The growth rate of roof subsidence is approximately 1.8~2.0 times that of the sidewalls, while the floor displacement accounts for only 8%~15% of the roof displacement. The dominant deformation ratio can be quantified as:

$$R_{rs} = \frac{u_{\text{roof}}}{u_{\text{rib}}} \approx 1.9 \quad (8)$$

This value is significantly higher than that of general small- and medium-section roadways (1.0~1.2), confirming, from the perspective of the displacement field, the geometric-mechanical amplification effect of roof settlement in large-span headgates. When the

vertical stress reaches 15 MPa, distinct zones of abrupt change in displacement direction appear in the deep crown (2.5~3.2 m from the free surface) and along the 45° direction of the arch shoulder (2.0~2.8 m from the free surface), indicating that the surrounding rock has entered the precursor stage of localized instability.

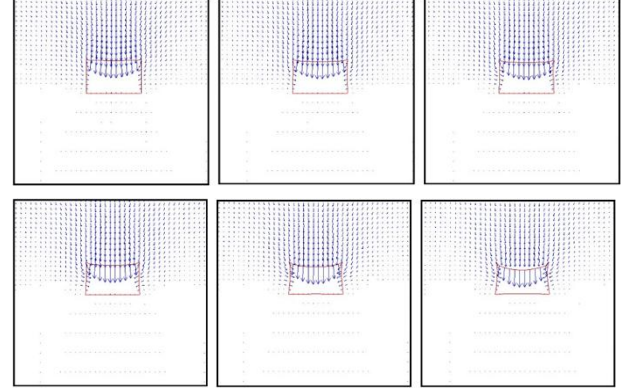


Fig.3 Displacement Vectors of Roadway Surrounding Rock Under Different Vertical Loads

3.3 Mechanical Derivation of the Displacement Vector Mutation Criterion

Most existing evaluations of surrounding rock stability rely on single indicators, such as displacement magnitude, stress level, or plastic zone extent^[19,21,22], making it difficult to directly identify instability precursors from the displacement field. In this section, within the framework of the Rudnicki–Rice strain localization bifurcation theory^[19] and associative flow plasticity, we derive an instability criterion based on an abrupt change in the displacement vector direction.

3.3.1 Kinematic Description

Let the displacement field of the surrounding rock be $\mathbf{u}(\mathbf{x}, t) = [u_1, u_2, u_3]^T$, and the unit displacement direction vector be $\mathbf{n}_u(\mathbf{x}) = \mathbf{u}(\mathbf{x}) / \|\mathbf{u}(\mathbf{x})\|$. The displacement gradient tensor $(\nabla \mathbf{u})_{ij} = \partial u_i / \partial x_j$, through its symmetric and antisymmetric decomposition, yields the strain tensor and the rotation tensor:

$$\boldsymbol{\varepsilon} = \frac{1}{2}(\nabla \mathbf{u} + \nabla \mathbf{u}^T), \quad \boldsymbol{\omega} = \frac{1}{2}(\nabla \mathbf{u} - \nabla \mathbf{u}^T) \quad (9)$$

Define the displacement vector direction mutation angle θ as the angle between the displacement direction vectors at two adjacent points P_1 and P_2 :

$$\theta = \arccos(\mathbf{n}_u(P_1) \cdot \mathbf{n}_u(P_2)), \quad \|\mathbf{P}_1 \mathbf{P}_2\| \leq \delta \quad (10)$$

Where δ is the spatial resolution scale. In the uniformly elastic deformation zone $\theta \rightarrow 0$; when a localized shear band appears, θ abruptly changes on both sides of the band.

3.3.2 Relationship with the direction of principal plastic strain

According to associative flow plasticity theory, the plastic strain rate obeys the normality flow rule:

$$\dot{\boldsymbol{\varepsilon}}^p = \dot{\lambda} \cdot \frac{\partial F}{\partial \boldsymbol{\sigma}} \quad (11)$$

Where F is the yield function and $\dot{\lambda} \geq 0$ is the plastic multiplier. For the Mohr–Coulomb criterion, the unit vector in the direction of the maximum principal plastic strain is:

$$\mathbf{n}_p = \left[\cos\left(\frac{\pi}{4} + \frac{\varphi}{2}\right), 0, \sin\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) \right]^T \quad (12)$$

After entering the plasticity-dominated stage, the direction of the displacement increment $d\mathbf{u}$ gradually converges to the principal direction $\mathbf{n}_p$, i.e. $\mathbf{n}_u \rightarrow \mathbf{n}_p$. Therefore, a quantitative relationship exists between the displacement vector mutation and the direction of the principal plastic strain:

$$\theta \approx \arccos(\mathbf{n}_p(P_1) \cdot \mathbf{n}_p(P_2)) = |\Delta\varphi| \cdot \cos(2\varphi) \cdot \text{sign}(\Delta\psi) \quad (13)$$

Where $\Delta\varphi$ is the effective change in friction angle before and after localization, and $\Delta\psi$ is the change in dilation angle. For a typical coal-rock mass ($\varphi=28^\circ$, $dy/dx=28^\circ \sim 38^\circ$, $\Psi=5^\circ \sim 10^\circ$), if the residual weakening is 15° , substituting into Eq.(12) gives $\theta \in [22^\circ, 31^\circ]$. Taking an intermediate value, this paper proposes $\theta_c \approx 27^\circ$ as the critical mutation angle for structural instability of the surrounding rock.

3.3.3 Localization criterion based on bifurcation theory

Let the unit normal vector of the shear band be n , and let the velocity jump across the band be $[[v]] = g(t)m$ ($g(t) \geq 0$, where m is the unit vector in the direction of the relative velocity inside the band). From the equilibrium equations and the constitutive relations, the necessary condition for localization is the singularity of the acoustic tensor $A(\mathbf{n})$:

$$\det[\mathbf{A}(\mathbf{n})] = \det[\mathbf{n} \cdot \mathbf{C}^{ep} \cdot \mathbf{n}] = 0 \quad (14)$$

Where $\mathbf{C}^{ep}$ is the elasto-plastic tangent modulus tensor. For Mohr–Coulomb materials, the bifurcation direction $\mathbf{n}^*$ is the normal to the shear band, and Arthur's formula gives the corresponding localization angle:

$$\beta_l = \frac{\pi}{4} + \frac{\varphi + \psi}{4} \approx 52^\circ \sim 55^\circ \quad (15)$$

After the formation of the shear band, the displacement direction inside the band changes from being nearly perpendicular to the band before its formation to sliding along the direction $\mathbf{m}$. The angle between the displacement directions on the two sides of the band is:

$$\theta_{\text{bif}} = 2\beta_l - \pi/2 \approx 24^\circ \sim 30^\circ \quad (16)$$

Equations (15) and (12) are in perfect agreement; they cross-validate the rationality of $\theta_c \approx 27^\circ$.

3.3.4 Engineering quantitative criterion and numerical validation

From an energy perspective, the total deformation energy is $W = W_e + W_p$, and the energy dissipation ratio is defined as $\eta_p = W_p / W$. Numerical simulation shows that when $\theta \geq \theta_c$ is satisfied, η_p exceeds 0.65, indicating that the surrounding rock has entered a dissipation-dominated critical instability state. Integrating Eqs. (10), (12)–(15), this paper proposes the displacement vector mutation criterion for structural instability of large-span roadways under dynamic pressure:

$$\theta(\mathbf{x}) \geq \theta_c, \wedge \frac{\partial \theta}{\partial r} > \zeta, \quad \theta_c = 27^\circ, \zeta \approx 15^\circ / \text{m} \quad (17)$$

Applying the criterion to the numerical results under different load conditions in Section 4.2: at $q \leq 12$ MPa, $\theta < 20^\circ$, there is no mutation zone in the entire field; at $q = 15$ MPa, mutation zones first appear in the deep crown and along the 45° direction of the arch shoulder ($\theta = 27^\circ \sim 32^\circ$); at $q = 18$ MPa, the mutation zones connect to form an embryonic shear slip band; at $q = 21$ MPa, the mutation zones form a closed arc-shaped belt, making structural instability of the surrounding rock unavoidable. Therefore, $q = 15$ MPa can be taken as the stress threshold, which agrees with the field monitoring critical indicator (the stress condition under which the periodic weighting interval decreases from 40m to 25~30m).

3.4 Core mechanism of surrounding rock deformation control

The stability control of surrounding rock in large-span roadways under dynamic pressure must prioritize the roof. The mechanism can be summarized as follows: the synergistic action of horizontal tectonic stress and self-weight stress induces incompatible

bending and delamination failure of the composite roof (top coal and sandy mudstone), resulting in $R_{rs} \approx 1.9$ much higher values than those in conventional roadways; the local abrupt change in displacement vector direction corresponds to the rapid reorientation of the principal plastic strain direction. The control strategy should specifically inhibit roof bending deformation and delamination development: ① In the shallow part, a high-density array of rock bolts actively reinforces the fractured roof zone, forming an "inner arch" with self-supporting capacity; ② In the deep part, high-pretension cables penetrate through the failure zone, anchoring the shallow structure into the deep stable rock strata, thereby constructing an "outer arch" that bears loads across the failure zone; ③ The mechanical system formed by the synergy of the two arches blocks the chain instability path of roof "bending – delamination – penetration".

4 Strata Movement Laws in Multi-Seam Mining

4.1 Influence of Extraction of Working Face 914 on the Surrounding Rock of the Roadway

A FLAC^{3D} simulation was conducted to model the extraction process of Working Face 914 (advancing 30m per step, for a total of 240m). The simulation results (Fig.4 and Fig.5) show that when the working face is more than 40m away from the roadway, the deformation of the surrounding rock increases slowly; when the distance is less than 40m, the roadway enters a stage of intense deformation, indicating that the mining influence exhibits a distinct critical-response characteristic. The displacement-distance curves (Fig.6) for solid roadways and gob-side roadways (with 20m/10m coal pillars) indicate that the advance influence distance of the gob-side roadway (approximately 130m) is greater than that of the solid roadway (approximately 40m). However, the safe distance for both roadway types is essentially the same, at about 40m, suggesting that under the mining conditions of the No. 9 coal seam, the coal pillar width can be appropriately reduced to improve resource recovery.

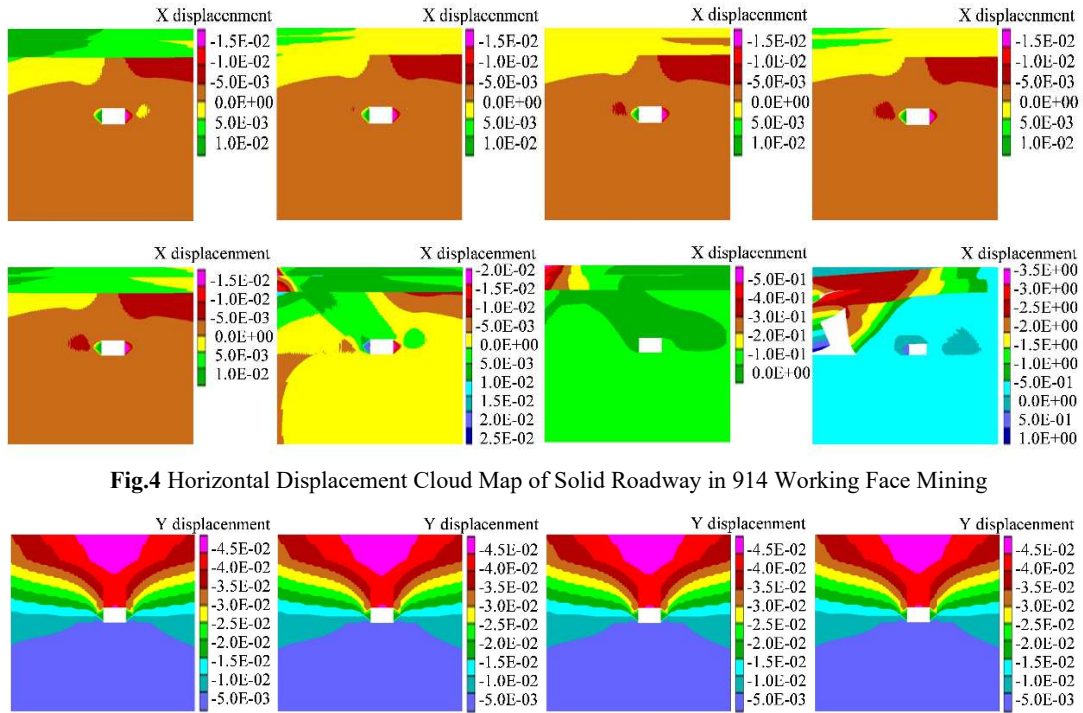


Fig.4 Horizontal Displacement Cloud Map of Solid Roadway in 914 Working Face Mining

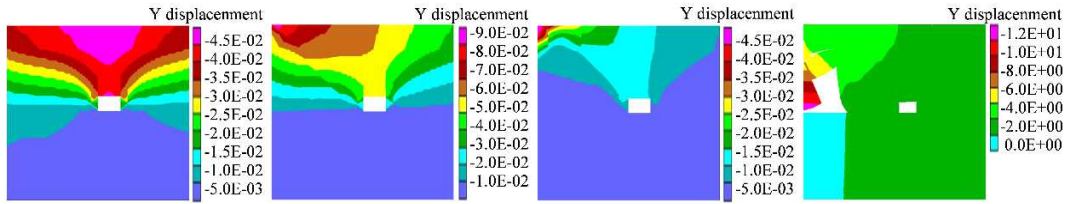


Fig.5 Vertical displacement contour of solid roadway in Working Face 914 mining

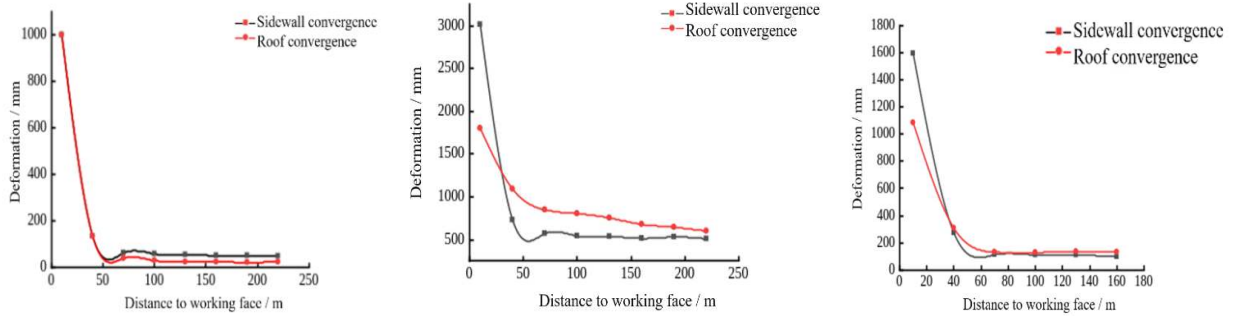


Fig.6 Comparison of Displacement-Distance Curves for Solid Roadway and Gob-Side Roadway (20m/10m Coal Pillar)

4.2 Stress distribution in the No.9 coal roadway under the condition of No.4 coal mining

After mining of the No. 4 coal seam, the vertical stress distribution in the No. 9 coal seam can be divided into four zones (Fig.7): the abutment pressure zone, the stress reduction zone, the stress recovery zone, and the stress concentration zone. For Working Face 412, which has a large panel size, the overlying strata contact the gob after extraction, leading to stress recovery in the

No. 9 coal seam. Stress concentrations occur on both sides of the gob, and the area near the gob edge in the coal body is a stress-reduction zone. For Working Face 413, which has a small panel size and the overlying strata do not touch the gob, the stress in the No. 9 coal seam is generally reduced. For Working Face 914, the material roadway is located in a pressure relief zone, while the belt roadway is situated in the in-situ stress zone.

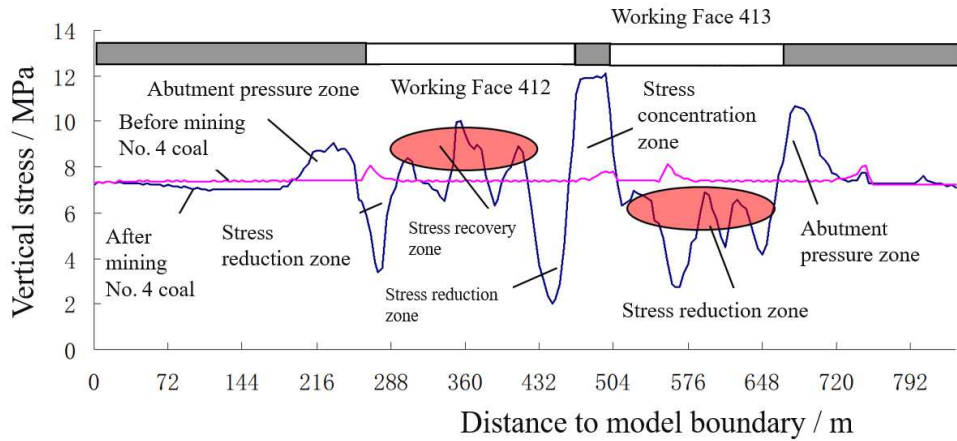
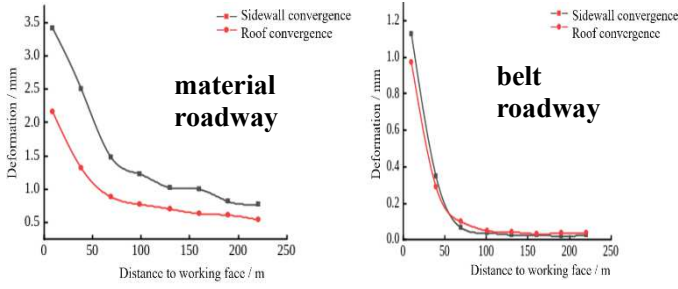


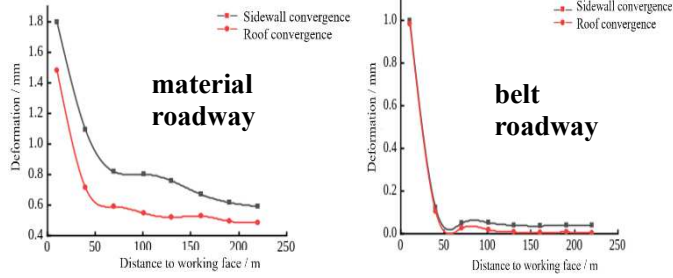
Fig.7 Vertical Stress Distribution of No.9 Coal Seam Before and After Mining of No.4 Coal Seam

Comparison of roadway surrounding rock deformation during extraction of Working Face 914 under two conditions: without mining No. 4 coal seam versus with mining No. 4 coal seam (Fig.8): When the No. 4 coal seam is not mined, the intense influence range is 50~70m; at a distance of 10m from the working face, the convergence of the two sidewalls in the material roadway is 3.34m, and the convergence between the roof and floor is 2.15m. Under the condition of mining

the No. 4 coal seam, the intense influence range reduces to 40~50m, the sidewall convergence decreases to 1.8m, and the roof-floor convergence decreases to 1.5m. After mining the No.4 coal seam, the initial caving step distance of the main roof for Working Face 914 increases from 40m to 60m, indicating a slowing of roof movement, which is beneficial for the surrounding rock control of the No. 9 coal seam extraction roadway.



(a) Deformation Curve of Roadway Surrounding Rock without Mining No. 4 Coal Seam



(b) Deformation Curve of Roadway Surrounding Rock with Mining No. 4 Coal Seam

Fig.8 Deformation Curve of Roadway Surrounding Rock
4.3 Similarity simulation and three-stage evolution model

Based on the three similarity theorems, a similar material model was designed with a geometric ratio of 1:200, a unit weight ratio of 1:1.6, and a strength ratio

of 1:320 (self-consistent $C_\sigma = C_L \times C_\gamma$). A total of 27 rock strata were simulated; the cross-section of the 914 headgate corresponds to 27.5mm×17.5mm. Five survey lines and 80 monitoring points were arranged, and digital image correlation (DIC) technology was employed to obtain non-contact, full-field displacement measurements with a spatial resolution of 0.3mm. The experiment was conducted in two stages: ① sequential extraction of Working Faces 411, 412, and 413; ② sequential extraction of Working Faces 913 and 914.

No.4 coal mining stage: In the initial stage (advancing 60 m), the overlying key stratum experienced its first breakage, forming an O-X type structure. After the continuous extraction of Working Faces 411–413, the gobs became interconnected, and the breakage step distance of the key stratum increased from 20 m to 35 m, resulting in the formation of a typical three-zone structure consisting of the caved zone, the fractured zone, and the bending subsidence zone. The No. 9 coal mining stage exhibits a three-stage evolution of "disturbance triggering – fracture connection – secondary reactivation", as shown in Fig.9 and Fig.10:

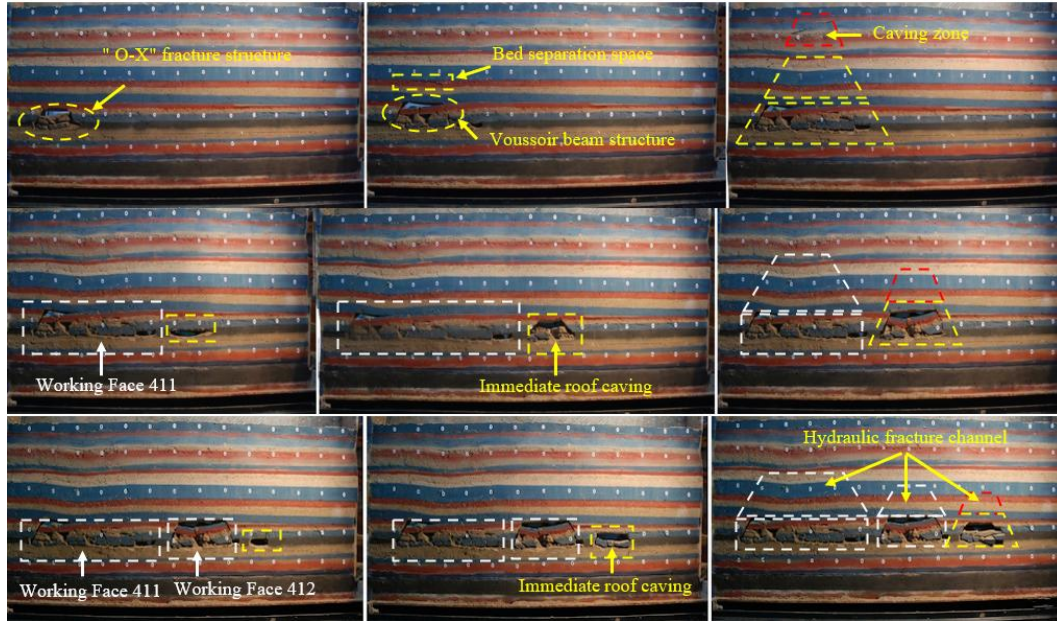


Fig.9 Failure and Instability Characteristics of Overlying Strata Induced by No.4 Coal Seam Mining (Working Face 411, 412, 413)

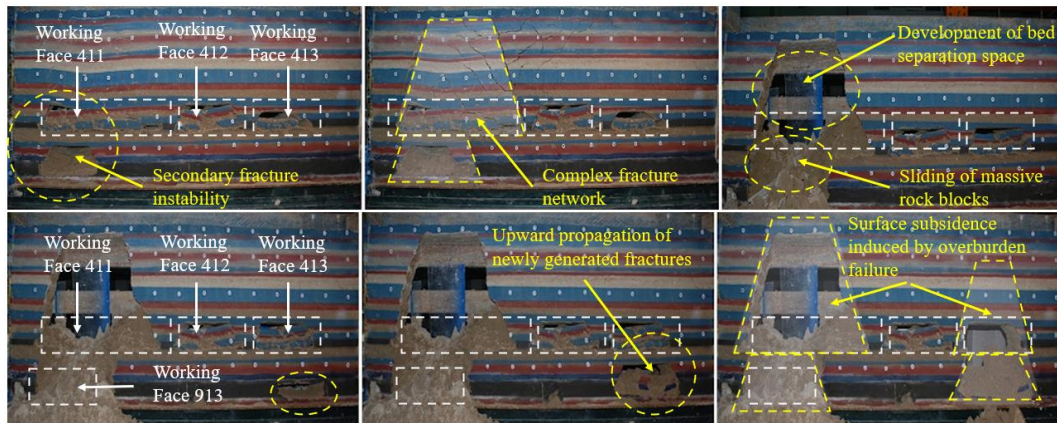


Fig.10 Fracture Characteristics of Overlying Strata in No.9 Coal Seam Mining and Secondary Activation of Key Strata in No.4 Coal Seam

Stage 1 (advancing 60–100 m)--Disturbance triggering period. The advanced abutment pressure of the No. 9 coal seam penetrates the interlayer rock pillar, and the residual voussoir beam structure of the key stratum located within the fractured zone of the No. 4 coal seam experiences secondary unloading disturbance from below. The inter-block occlusal friction force decays rapidly. The dynamic load coefficient measured by field monitoring increases from 1.2 to 1.5.

Stage 2 (100–120m)--Fracture connection period. Newly generated fractures from the collapse of the No. 9 coal immediate roof propagate upward at a breaking angle of $70^{\circ}\sim 85^{\circ}$ and connect with the pre-existing fracture zone of the No. 4 coal seam at the weak mudstone interlayer between the seams. The height of the water-conducting fracture zone increases abruptly from the original 28 m to approximately 47 m, an increase of about 68%.

Stage 3 (>120 m)--Secondary reactivation and instability period. The residual voussoir beam blocks of the No. 4 coal key stratum lose interlayer constraints and undergo secondary reactivation instability. The periodic weighting interval shortens from 40m to 25~30m, the dynamic load coefficient rises to 1.8~2.3, energy release becomes highly concentrated, and the fractured structure tends toward disordering.

This three-stage model is consistent with the empirical “stress transfer – structure reactivation” law summarized by Peng^[9] for longwall multi-seam mining in the United States. However, through similarity simulation, this study quantitatively provides the advanced distance thresholds, breaking angle, and dynamic load coefficient range for each stage, offering

more refined engineering criteria for support design and hazard warning.

5 Double-arch cooperative support mechanism and parameter design

5.1 Mechanical model of the double-arch cooperative load-bearing structure

Based on Hou Chaojong's surrounding rock strength reinforcement theory for rock bolting^[25] and Kang Hongpu's high-pretension powerful support theory^[26], and targeting the conditions of large-span dynamic-pressure composite roofs, this paper proposes the principle of “failure depth – arch stiffness matching”. Rock bolts penetrate the shallow failure zone of the roof (approximately 1.68m deep) and, through a high-density array (1m×1m), form a continuous compression zone, reinforcing the loose coal-rock mass into a shallow inner arch with a thickness of about 2.0m. High-pretension cables, 7.3m in length with a spacing of 3m×3m, pass through the shallow failure zone and extend into the deep stable bedrock, constructing an outer load-bearing arch with a span of 5.5m and a height of 3.5m, as shown in Fig.11. The stress transmission path is: roof load → inner arch compression zone → cable pretension anchorage section → outer arch compression zone → restraining reaction force from the deep surrounding rock.

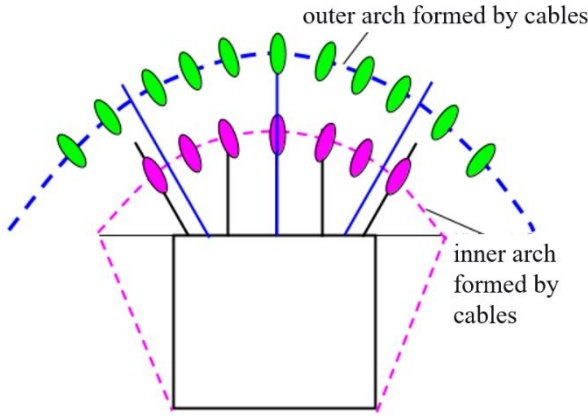


Fig.11 Schematic Diagram of Double-Arch Cooperative Bearing Mechanism

The inner arch enhances the frictional resistance at the bedding interface through the transverse restraint reaction force $\tau = q_b \tan \varphi + c$ of the rock bolts, thereby suppressing the transformation of bending deformation into delamination. The outer arch, through a cable pretension of 250–300 kN, “welds” the composite roof and the deep stable bedrock into an integral beam structure, eliminating the source of incompatible bending moments in the composite roof. Defining the bending stiffnesses of the inner and outer arches as $K_i = E_i t_i^3$ and $K_o = E_o t_o^3$ (where t_i and t_o are the characteristic thicknesses of the inner and outer arches, respectively), the stiffness matching condition for cooperative operation is:

$$\frac{K_i}{K_o} = \frac{E_i t_i^3}{E_o t_o^3} \in [0.1, 0.3] \quad (18)$$

This ratio interval ensures that the inner arch has sufficient deformation capacity to dissipate the surrounding rock's deformation energy. In contrast, the outer arch has sufficient stiffness to limit overall roof subsidence. Under the conditions of this project, the ratio $K_i / K_o \approx 0.18$ falls within the middle of the recommended range. Compared with the existing empirical ratio approach, this condition provides a clear quantitative constraint.

5.2 Support parameter calculation

Based on Protodyakonov's arch theory and the limit equilibrium method, the failure depth of the two sidewalls c and the failure depth of the roof b are respectively given by:

$$\begin{cases} c = \frac{h}{2 \tan \varphi} \ln \frac{K_{CX} \gamma H B + \sigma_c \cos \varphi / (1 - \sin \varphi)}{\sigma_c} \\ b = a / f \end{cases} \quad (19)$$

Taking $K_{CX} = 3$, $\gamma = 25 \text{ kN/m}^3$, $H = 250 \text{ m}$, $\sigma_c = 25 \text{ MPa}$, $B = 2$, $f = 2.5$, $h = 3.5 \text{ m}$, $\varphi = 35^\circ$, $a = 2.75$ and substituting into Equation (19) yields: $c = 1.45 \text{ m}$, $b = 1.68 \text{ m}$. The failure zone and the point of the resultant force are shown in Fig.12.

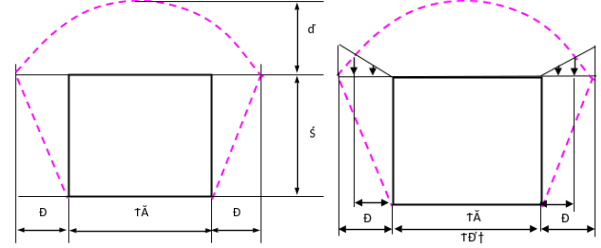


Fig.12 Failure Range of Coal Seam Roadway Surrounding Rock and Position of Resultant Force Action Point

The lower limit of the effective length of the sidewall rock bolts is taken as 2/3 of the failure zone (at the point of resultant force action), and the upper limit is taken as the entire failure zone; in practice, a length of 2.0m is adopted. Similarly, the effective length of the roof rock bolts is taken as 2.5m. The spacing of the roof rock bolts is determined by taking the smaller value from the two conditions: rock weight balance within the arch and shear strength:

$$d_1 = \sqrt{\frac{R}{k \gamma_m b}}, \quad d_2 = \sqrt{\frac{\pi d^2 \tau_f h}{4 k \gamma_m a}} \quad (20)$$

Taking $R = 70 \text{ kN}$, $\gamma_m = 14 \text{ kN/m}^3$, $k = 3$, $d = 20 \text{ mm}$, $\tau = 400 \text{ MPa}$, $f = 0.3$, we obtain $d_1 = 1.0 \text{ m}$, $d_2 = 1.58 \text{ m}$, and then take $\min(d_1, d_2) = 1.0 \text{ m}$. The sum of the shear resistances of the sidewall bolts should be greater than the weight of the failed coal mass in the sidewall, $nQ \geq k_1 \gamma_m c h a$. When the safety factor is $k_1 = 2$, $nQ \geq 50 \text{ kN}$. For the sidewall, either two fiberglass-reinforced plastic bolts or one steel bolt can be used. The scheme is shown in Fig.13.

The number of cables should be sufficient to ensure that the sum of their breaking forces exceeds the weight of the rock mass within the roof failure zone. The rock weight per meter along the strike is $w = 1.68 \times 5.5 \times 1.4 = 12.9 \text{ t}$. With a safety factor of 1.5 and a cable breaking load of 30t, each cable supports the rock weight over a length of 1.5m. Two cables are arranged per row, with a row spacing of 3m. The cable length is $L = 2(b + 1.5 + 0.5) = 7.36 \text{ m}$, taken as 7.3m. The final spacing is set at 3m×3m, with a pretension of 250–300kN.

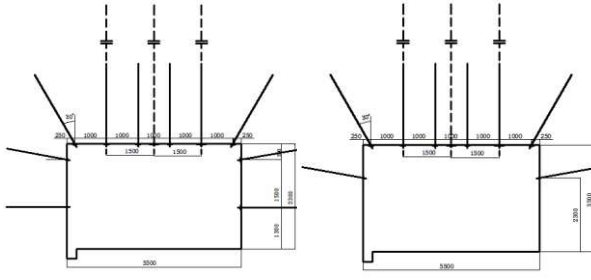


Fig.13 Cross-section of Double-Arch Cooperative Support Scheme for Gateway of Working Face 914

6 Field measurement and support effectiveness verification

6.1 Monitoring scheme

Fifteen comprehensive monitoring stations were arranged along the advancing direction of the headgate of Working Face 914 at 20m intervals. Stations 1# to 5# adopted the original support scheme (bolts: length 1.8m, spacing 0.8m×0.8m; cables: length 5.5m, spacing 2.5m×2.5m, pretension 150kN) as the control group, and stations 6#~15# adopted the optimized double-arch cooperative support scheme proposed in this paper as the experimental group. The two groups were located in adjacent sections with similar geological conditions to avoid systematic bias. The monitoring items included:

surface displacement (cross-point method); roof multipoint extensometers (four points at depths of 1.5m, 3.0m, 5.0m, and 7.5m); working resistance of rock bolts and cables (instrumented monitoring of three bolts and two cables at each station); and local displacement vector fields (stations 5# and 10# were additionally equipped with 12-channel displacement direction angle sensor arrays). All instruments were electronic displacement gauges (resolution 0.1mm) and strain-type load cells (range 400kN, accuracy ±1%FS). Data were sampled at a rate of 4 times per hour, and continuous monitoring was carried out for six months (from 200m ahead of the working face advance to 100m after the working face passed the station).

6.2 Surface displacement and roof separation

The surface displacement curves at typical monitoring stations exhibit a typical "three-stage" growth pattern. For the control station 2#, the peak roof subsidence was 236mm; for the experimental station 12#, the peak value was 89mm. The final deformation data for all 15 monitoring stations are listed in Table 2.

Table 2 Final Deformation Monitoring Data of 15 Monitoring Stations

Monitoring station	Scheme	Roof subsidence/mm	Sidewall convergence/mm	Floor heave/mm	R_{rs}
1#	Original scheme	228	186	24	1.94
2#	Original scheme	236	178	22	1.95
3#	Original scheme	198	165	21	1.84
4#	Original scheme	246	192	28	1.88
5#	Original scheme	272	198	31	1.95
6#	Optimization	92	78	9	1.93
7#	Optimization	78	68	7	1.85
8#	Optimization	98	82	11	1.88
9#	Optimization	85	71	8	1.90
10#	Optimization	94	79	10	1.86
11#	Optimization	72	65	6	1.82
12#	Optimization	89	75	9	1.89
13#	Optimization	108	88	12	1.96
14#	Optimization	82	72	8	1.84
15#	Optimization	96	80	10	1.88

For the control group ($n=5$), the mean roof subsidence was 236mm (SD=28.5mm, 95% CI [201, 271]); for the experimental group ($n=10$), the mean was 89mm (SD=10.6mm, 95% CI [82, 97]). Welch's t -test: $t=11.36$, $df=4.7$, $p<0.001$, indicating a highly statistically significant difference. The reduction in roof subsidence was 62.3%, the reduction in sidewall

convergence was 57.9%, and the reduction in floor heave was 64.2%. The dominant deformation ratio R_{rs} had a mean of 1.89 across the two groups, which validates the theoretical value of 1.9 given in Section 3.2. The roof multipoint extensometer showed that in the control group, the peak separations in the middle part (1.5~3.0m) and the deep part (3.0~5.0m) were 45mm

and 32mm, respectively; in the experimental group, these values decreased to 12mm and 6mm, reductions of 73.3% and 81.3%, respectively. In the bedrock layer (5.0~7.5m), both groups maintained small separations (<3mm), indicating that the design, in which the cable anchorage section is placed into the stable rock mass, is effective.

6.3 Field validation of the displacement vector mutation criterion

The monitoring data from control station 5# show that when the working face was 35m away from the station, the displacement direction angle at the deep crown (2.7m from the free surface) began to rotate rapidly from its initial value (nearly vertical downward). When the working face approached 15 m from the station, the mutation angle θ reached 29.4° , corresponding to a vertical stress of approximately 15.8MPa; simultaneously, the deep roof separation increased abruptly from 18mm to 32mm, and the subsidence rate increased from 2.5mm/d to 6.8mm/d. These field measurements are in excellent agreement with the previously proposed theoretical criterion $\theta_c = 27^\circ$ and the stress threshold of $q = 15$ MPa (deviation <10%). At the experimental station 10#, the maximum rotation of the displacement direction angle at the deep crown was only 18.6° over the same range

of working face distances; throughout the entire mining period, θ remained below 22° , indicating that the double-arch cooperative support effectively prevented the surrounding rock from entering the precursor stage of localized instability. This directly validates, from the perspective of in-situ field monitoring, the effectiveness of the proposed criterion and the support system.

6.4 Comprehensive evaluation of support effectiveness

Continuous application observations over nearly one year show that after adopting the double-arch cooperative support scheme proposed in this paper for the large-span headgate of Working Face 914 in the No. 9 coal seam, no safety incidents such as roof falls, rib spalling, or roof instability have occurred, and the roadway has maintained a usable cross-section over the long term. A comprehensive comparison is provided in Table 3, where all roadway indicators show significant improvement. In terms of engineering cost accounting, the optimized scheme results in a slight increase in material cost per linear meter compared with the original scheme (mainly due to the increased cable length and number). However, because the deformation is small and no repairs are required, the overall cost is reduced, yielding notable social and economic benefits.

Table 3 Comprehensive Comparison of Support Effects Between Original Scheme and Optimized Scheme

Evaluation indicator	Original scheme (1#~5#)	Optimized scheme (6#~15#)	Reduction/improvement
Mean maximum roof subsidence/mm	236	89	-62.3%
Mean sidewall convergence/mm	178	75	-57.9%
Mean floor heave/mm	25	9	-64.2%
Mean deep roof separation/mm	32	6	-81.3%
Peak displacement mutation angle/(°)	29.4	18.6	-36.7%
Stable working resistance of cables/kN	95~120	180~220	+83.3%

7 Discussion

7.1 Comparison between this study and classical support theory systems

The lateral abutment pressure analytical results of this study ($\sigma_{\max} \approx 12$ MPa, peak distance 16m) are significantly smaller than the peak distance of 22~26m observed by Peng^[34] in the shallow coal seams of the Powder River Basin. This is mainly attributed to the fact that the gob height in Chinese fully mechanized top-coal caving (approximately 15 m) is much larger than that in the Powder River Basin (7~9m), as well as the difference in key stratum lithology stiffness (18.2GPa vs. 8GPa), resulting in a shorter stress transmission path and

a more compact stress concentration. Wilson's limit equilibrium formula^[35] requires the introduction of a coal seam thickness correction factor (in this paper, $x_0 = (M / 2f) \ln(\cdot)$) under Chinese shallow-buried top-coal caving conditions.

The incremental contributions of the double-arch cooperative load-bearing structure proposed in this paper relative to previous theories can be expressed in four aspects: ① Layered reinforcement instead of uniform reinforcement – Hou Chaojiong's theory^[25] is based on the assumption of uniform bolt constraint; this study extends it to a three-layer structure of “failure zone – transition zone – stable zone”, where the inner bolt

arch, interfacial compressive stress, and outer cable arch respectively assume different mechanical functions; ② Stiffness matching instead of empirical values – Kang Hongpu^[6] emphasizes the superposition of the prestress field, but the relationship between the inner and outer arch stiffnesses is mainly based on empirical values; this study provides for the first time the quantitative constraint of $K_i / K_o \in [0.1, 0.3]$ (Eq. 17).; ③ Precursor identification instead of pattern judgment – Zhao Zhiqiang's^[33] butterfly-shaped plastic zone control relies on numerical simulation; this study takes the abrupt change in displacement direction as the monitoring target, which can be directly identified through routine field displacement measurements; ④ Supplement for shallow static-dynamic combined loading conditions – He Manchao's^[27] NPR cables are mainly designed for deep rockburst-prone conditions; this study fills the design gap for shallow, non-rockburst-prone but large-span top-coal controlled conditions under static-dynamic combined loading.

7.2 Theoretical position and application boundaries of the displacement vector mutation criterion

The displacement vector mutation criterion (Eq. (16)) is strictly consistent with the Rudnicki–Rice strain localization bifurcation theory in its theoretical basis; both treat the onset of shear banding as a transition from a continuous to a discontinuous velocity field. However, their descriptions differ: Rudnicki–Rice takes the strain-rate jump as the fundamental variable, with the criterion expressed as the singularity of the acoustic tensor, which is suitable for theoretical studies at the constitutive level. In contrast, this paper takes the reorientation of the displacement vector as the fundamental variable, establishing a one-to-one correspondence with the field-monitorable displacement field, thereby offering greater engineering operability. Compared with the displacement mutation criterion based on the finite element strength reduction method by Griffiths and Lane^[36], the criterion in this paper does not rely on the numerical convergence judgment of the reduction process. Instead, it is based on the topological evolution characteristics of the displacement field itself, reducing sensitivity to mesh density and constitutive parameters. Compared with the energy-release-rate-based rockburst criterion by Kaiser

et al.^[37], this paper's criterion establishes a connection through the energy dissipation ratio η_p and extends its applicability to slowly deforming instability scenarios without rockburst tendency.

It should be noted that the criterion has the following application boundaries: ① For purely elastic-brittle surrounding rock (where the residual weakening amplitude of the friction angle φ is small), the mutation amplitude of θ may be insufficient; the critical threshold needs to be calibrated individually based on the degree of residual internal friction angle weakening; ② In heterogeneous areas containing large geological structures (faults, collapsed columns), the structural geometry dominates the displacement field, and the distribution of the mutation angle may deviate from the theoretical value derived under the homogeneity assumption; ③ Under special environments such as strong water hazards or high temperatures, joint analysis with fluid-solid coupling and thermo-mechanical coupling effects is required. Furthermore, in this paper, field displacement direction-angle sensors were installed at only two monitoring stations; future work should extend this to additional measurement points to establish a regional validation database for the criterion. The numerical simulations were primarily two-dimensional plane-strain; three-dimensional simulations of the full mining process need to be systematically conducted.

8 Conclusion

(1) Based on the key stratum theory, it is derived that the lateral abutment pressure for a shallow-buried fully mechanized top-coal caving face exhibits a unimodal distribution, $\sigma_{\max} \approx 12$ MPa, a peak distance of 16m from the gob boundary, and an influence range of 34m. The peak of the advance abutment pressure in the strike direction is located 14~20m ahead of the coal wall, with an influence range of 28~40m. The peak abutment pressure is lower than the coal mass's compressive strength (20MPa), providing a mechanical basis for the design of gob-side entry driving and small coal pillars.

(2) Within the framework of the Rudnicki–Rice strain localization bifurcation theory and associative flow plasticity, an instability criterion for surrounding rock based on an abrupt change in displacement vector

direction is rigorously derived. The quantitative relationships among the displacement-direction reorientation, the direction of the principal plastic strain, and the acoustic tensor singularity are clarified. The critical mutation angle $\theta_c \approx 27^\circ$, the corresponding vertical stress threshold of 15MPa, and the critical spatial gradient $\zeta \approx 15^\circ/\text{m}$ are given. This criterion provides a directly monitorable and identifiable instability precursor indicator from the perspective of the displacement field.

(3) The deformation of surrounding rock in large-span roadways under dynamic pressure exhibits a roof-settlement-dominated mode ($R_{rs} \approx 1.9$). Based on the Hou–Kang support system, the principle of “failure depth – arch stiffness matching” ($K_i / K_o \in [0.1, 0.3]$) is proposed, and a double-arch cooperative load-bearing structure consisting of a “bolt-mesh inner arch + cable outer arch” is constructed. This forms an integrated quantitative design method that spans from determining the failure zone to matching bolt and cable parameters and coordinating prestress.

(4) Under multi-seam mining conditions, extraction of the No. 9 coal seam induces secondary reactivation instability of the key stratum of the No. 4 coal seam, exhibiting a three-stage evolution of “disturbance triggering – fracture connection – secondary reactivation”. After mining the No. 4 coal seam, the initial caving step distance of the main roof of Working Face 914 increases from 40m to 60m, and the intense influence range reduces from 50~70m to 40~50m, which is beneficial for surrounding rock control. The height of the water-conducting fracture zone increases from 28m to 47m, imposing higher requirements for water hazard prevention.

(5) Based on field monitoring at 15 stations over 6 months: the mean maximum roof subsidence decreased from 236mm to 89mm (a reduction of 62.3%, $p < 0.001$), the mean sidewall convergence decreased from 178mm to 75mm (a reduction of 57.9%), the mean deep roof separation decreased from 32mm to 6mm (a reduction of 81.3%), and the cable working resistance stabilized at 180~220kN. The measured displacement direction angle at Station 5# ($\theta = 29.4^\circ$, $q = 15.8$ MPa) deviated from the theoretical criterion ($\theta_c = 27^\circ$, $q = 15$ MPa) by less than 10%, comprehensively validating the

effectiveness of the double-arch cooperative support system and the displacement vector mutation criterion.

Data Availability

The datasets used and analysed during the current study are available from the corresponding author on reasonable request.

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