

Topical Collection Reprint

Sustainable Development of Coal Based Energy

Technology, Environment, Humanities, Economy, and Education: Prevention and Control of Dynamic Disasters

Edited by
Cun Zhang, Fangtian Wang, Shiqi Liu and Erhu Bai

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Sustainable Development of Coal Based Energy: Technology, Environment, Humanities, Economy, and Education

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Prevention and Control of Dynamic Disasters

Collection Editors

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About the Editors

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Cun Zhang is a Professor affiliated with the School of Energy and Mining Engineering at China University of Mining and Technology, Beijing. His research interests primarily include mine pressure and strata control, mine water resource utilization, coal and gas co-mining, and the reuse of abandoned mines. He has presided over more than 10 national and provincial-level vertical projects. His notable achievements include being a recipient of the Excellent Doctoral Dissertation Award from the Chinese Society of Rock Mechanics and Engineering, being accepted to the Young Talent Promotion Project of the China Association for Science and Technology, and being elected as a Fellow of the International Association of Advanced Materials (IAAM Fellow). Furthermore, he has been continuously listed in the Stanford University's Top 2% of the World's Most-Cited Scientists since 2022. In terms of academic contributions, he has received one first prize award for provincial-level teaching and five first prize awards for provincial-level scientific and technological progress, among other awards. He has produced over 200 publications, including more than 50 JCR Q1 papers as first or corresponding author, and 8 ESI highly cited papers, and has published 4 monographs and holds more than 10 invention patents as the first inventor. In addition to this, he serves as an Associate Editor for the *SCI Journal of Geophysics and Engineering* and is an Editorial Board Member for *Rock Mechanics Bulletin* and *Sustainability*.

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Preface

As coal mining progresses to greater depths and higher intensity levels, dynamic disasters have become a major threat to mine safety. Dynamic phenomena such as rock bursts, coal and gas outbursts, and roof water inrushes not only cause equipment damage and production interruptions but also directly endanger miners' lives. In recent years, frequent occurrences of dynamic disasters during deep resource extraction globally have posed new challenges to traditional mining science and engineering practices, given their complex mechanisms, difficulties in prediction, and the high complexity of their prevention and control.

This Reprint focuses on the core topic of "Prevention and Control of Dynamic Disasters", showcasing 12 innovative research papers that reveal the mechanisms behind dynamic disasters and explore advanced prevention technologies from multiple perspectives. These studies cover critical areas such as roof control, surrounding rock stability, pressure relief mechanisms, and hydrogeological impacts, forming a chain of innovations from theoretical exploration to engineering practice. Through integrated methods including theoretical modeling, experimental testing, field monitoring, and numerical simulations, this collection of research systematically elucidates cutting-edge issues such as the principles of strong ground pressure control under multiple key strata, the mechanisms of segmented enlarged-diameter borehole pressure relief, rock burst characteristics induced by TBM excavation in deep hard and soft compound strata, and energy accumulation laws under multiple mining disturbances. Innovatively, it also proposes new research pathways, such as risk assessment methods for floor water inrush based on the development patterns of pre-mining microseisms and the characteristics of acoustic emission and energy dissipation in sandstone under confined-pressure soaking.

This Reprint aims to provide a groundbreaking academic reference for researchers, engineers, and policymakers in fields such as mining engineering, rock mechanics, and safety engineering. It is particularly suited for professionals engaged in deep resource development and the prevention and control of dynamic disasters in mines. By presenting the latest theoretical insights into disaster mechanisms and innovative practices in prevention technologies, we hope to advance the improvement of technical systems for mine dynamic disaster prevention and control, providing theoretical support and technical assurance for building intrinsically safe mines.

We extend our gratitude to all the authors who contributed their research findings. Their outstanding work lays a solid foundation for deepening the understanding of dynamic disasters and breaking through technical bottlenecks in their prevention. Against the backdrop of ongoing global deep mineral resource development, the publication of this Special Issue will infuse the field with new academic momentum in the goal of promoting safe and sustainable development in the mining industry.

Cun Zhang, Fangtian Wang, Shiqi Liu, and Erhu Bai
Collection Editors

Article

The Principle and Practice of Strong Mine Pressure Control in the Initial Mining and Caving Stages under Multiple Key Strata

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Abstract: The mechanism of strong mine pressure control in the initial mining and caving stages under multiple key strata (MKS) was studied by theoretical analysis, numerical simulation and field measurement in order to avoid the dynamic pressure accidents caused by the MKS breaking in interactions in the initial mining and caving stage. Panel 13103 (P13103) in the Shan Xi Jinshen Shaping Coal Mine was used as the study site. The overlying structure and the breaking characteristics of the key strata were analyzed and the overlying structure model of longwall top-coal caving (LTC) under MKS was established. Reasonable initial mining heights and initial caving positions were determined and the spatial and temporal characteristics of ground pressure and roof structure of the working face in the mining process were obtained. The results show that, in the scheme of starting top-coal caving after the working face advanced 20 m, the support damage ratios were all greater than 50% with roof weighting, which is prone to induce prop crash accidents. The dynamic load formed by the second weighting was greater than first weighting under the MKS. The duration of mine pressure tended to be stable with fully developed caving zone heights. With the designed mining scheme, the safe and efficient mining of P13103 is guaranteed.

Keywords: multiple key strata; top-coal caving with large mining height; overburden structure; first roof caving; interactive roof breaking; dynamic pressure accident

1. Introduction

The coal seams in the northwestern region of China are generally shallow buried and superior deposits. A large number of modern mines have been built with high-intensity mining as the obvious indigenous characteristic [1]. In recent years, with the development of large mining heights and the technology and equipment of LTC, longwall top-coal caving with large mining heights (LTCLMH) has been widely used in extra-thick coal seams [2]. The stope space formed by LTC is large and as the disturbance distance overlay multiplies, dynamic disasters occur frequently. Among them, the strong strata behaviors of the roof in the initial mining and caving stages often cause supports to shrink sharply, with the rib spalling of the coal wall and the roof falling before the canopy, and even leads to support crushing accident, which threaten the safety of personnel and equipment in the working face and seriously affect production efficiency [3–6].

The LTC working face is generally characterized by intense mining pressure, large dynamic load coefficients, frequent opening of safety valves and severe mine pressure in the middle of the working face. To reveal the theoretical rock structure characteristics and the formation mechanism of severe rock pressure in LTCLMH, a series of mining-induced overlay strata structure models were proposed, such as ‘the up the masonry beam and

down the inverted step combination type of cantilever beam' [7], 'the combined short cantilever beam/articulated rock beam' [8] and 'the horizontal O–X breakage of far-field key strata' [9]. Because of the severe strata behavior conditions of fully mechanized cave mining under multi-gob and MKS, the reasonable selection of support and the efficient application of roof crack prevention and control measures are necessary to ensure safe mining [10,11]. In addition, the study of mine pressure under the conditions of single-key strata and MKS in the western mining area shows that the migration and breaking of the key strata determine the mine pressure behavior of the working face [12]. However, there are few studies on the prevention and control of strong mine pressure in the early stages of LTC under MKS.

In this paper, based on a case study in a Chinese coal mine, we studied the principle of strong mine pressure control in the initial mining and caving stages under MKS. To avoid the interaction among broken MKS in the initial mining stage, key strata were evaluated and the roof breaking step was analyzed, and the rock structure model of coal caving after the initial weighting under MKS was established. The simulation analysis and practice verified the prevention and control effects of coal caving on initial mining dynamic pressure accidents after the initial weighting under MKS, and the special mine pressure law in initial mining and initial caving under MKS is summarized.

2. Background

Shaping Coal Mine is located in the southeast of Hequ County and is located in the Hebaopian Mining Area of the Hedong Coalfield. The average coal thickness of the number 13 coal seam in the first panel is 13 m, and it is part of an extra-thick coal seam. The detection of coal thickness from the two entries shows that most of the areas of the first panel of 13103 branch into seam numbers 13-1 and 13-2. Due to the large change in the thickness of the number 13-1 coal seam, the middle sandwich is medium sandstone with an average thickness of 8 m, and only coal seam 13-2 is mined; the two entries into the area are arranged along the floor of the number 13-2 coal seam. P13103 is 1674 m long and 240 m wide, with a total of 140 hydraulic supports, an average inclination angle of 3° and a burial depth of 200 m. The equipment used in LTCLMH adopts ZFY13600/27/45D-type hydraulic supports and MG650/1620-WD-type shearers from Beijing Coal Machinery Factory, with a machine mining height range of 2.7–4.5 m. In situ stress measurements by the downhole multi-point method at the site showed that the initial stress direction is consistent and belongs to the slip type [13]. The vertical stress (σ_v) is formed by the self-weight of the overlying strata. The maximum horizontal principal stress (σ_H) is 1.7 times the vertical stress, the direction is NW43.8°, which is approximately parallel to the working face's advance direction and the minimum horizontal principal stress (σ_h) is 0.9 times the vertical stress. The overlying structure according to drill hole ZK902 inside P13103 is shown in Figure 1. The basic roof of P13103 is 8.0-meter thick medium sandstone, and there are multiple thick and hard layers above it; see the red rock layer in the columnar in Figure 1. Since the mine pressure in the initial mining stage of the adjacent mine under similar geological engineering conditions is intense, and prop crushes by dynamic pressure and roof collapse accidents have occurred, it is necessary to study the roof structure and mine pressure characteristics under similar conditions [14,15].

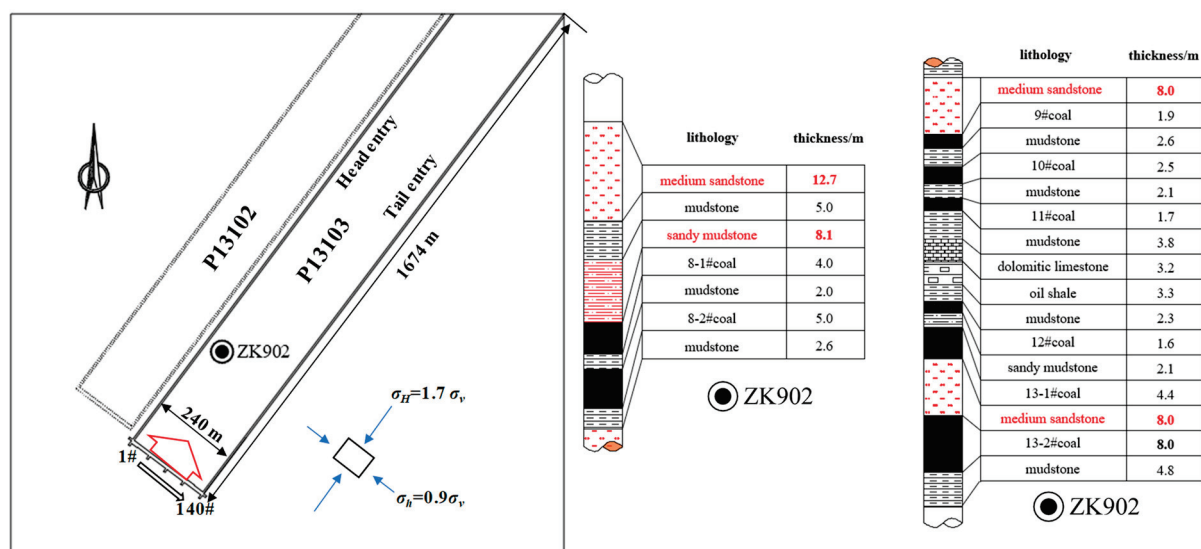


Figure 1. P13103 layout and geological conditions.

3. Roof Structure and Breaking Characteristics

The pressure behavior of the stope is dominated by the breaking structure of the upper strata. The previous mining pressure theory for stope mainly studied the influence of the direct roof and the basic roof near the stope [16,17]. However, the practice of large mining-height mining and top-coal caving cases show that, with the increase of the total mining height, the development heights of the caving zone and fracture zone increase in an exponential manner, and the high-level key strata (HKS) also affect the mine pressure of the working face [11]. The physical and mechanical parameters of coal and rock according to the ZK902 borehole in P13103 are shown in Table 1.

Table 1. Physical and mechanical parameters of strata.

Lithology	h^a (m)	ρ^b (kg·m ⁻³)	E^c (Gpa)	μ^d	T_l^e (Mpa)	RQD ^f (%)
Medium sandstone-D	12.7	2385	6.47	0.21	4.1	60
Mudstone	5.0	2600	8.25	0.31	1.6	55
Sandy mudstone-C	8.1	2560	15.46	0.23	3.6	45
8-1 coal (gob)	4.0	1340	7.93	0.25	1.2	55
Mudstone	2.0	2500	8.68	0.28	0.9	75
8-2 coal (gob)	5.0	1340	1.80	0.33	0.6	35
Mudstone	2.6	2580	10.01	0.31	1.1	48
Medium sandstone-B	8.0	2380	5.15	0.26	4.2	75
9 coal	1.9	1360	7.93	0.32	1.3	23
Mudstone	2.6	2580	8.68	0.28	1.1	75
10 coal	2.5	1350	9.28	0.38	1.3	32
Mudstone	2.1	2580	12.64	0.28	1.0	62
11 coal	1.7	1380	6.68	0.34	1.1	32
Mudstone	3.8	2480	9.82	0.31	1.2	60
Limestone	3.2	2730	22.75	0.26	4.9	45
Oil shale	3.3	2560	6.50	0.30	1.8	18
Mudstone	2.3	2560	10.28	0.32	2.1	0
12 coal	1.6	1420	4.20	0.33	0.6	5
Sandy mudstone	2.1	2590	15.46	0.23	3.4	81
13-1 coal	4.4	1440	7.50	0.32	1.2	21
Medium sandstone-A	8.0	2380	6.46	0.21	3.8	50
13-2 coal	8.0	1460	9.29	0.32	1.2	45
Mudstone	4.8	2650	10.01	0.28	2.2	58

^a h is thickness, ^b ρ is density, ^c E is elastic modulus, ^d μ is Poisson ratio, ^e T_l is tensile strength, ^f RQD is rock quality designation.

3.1. Overburden Key Strata Evaluation

For each rock layer of a different thickness and strength in the overburden rock, the basic roof load is calculated according to combination beam theory [18,19]:

$$(q_n)_1 = \frac{E_1 h_1^3 (\gamma_1 h_1 + \gamma_2 h_2 + \cdots + \gamma_n h_n)}{E_1 h_1^3 + E_2 h_2^3 + \cdots + E_n h_n^3} \quad (1)$$

where $(q_n)_1$ is the load accumulated on the basic roof imposed by the n -layer overburden, MPa; and γ_n , h_n , E_n is the volume weight (MN/m^3), thickness (m) and elastic modulus (MPa) of the n th-layer overburden, respectively.

If the conditions $(q_{n+1})_1 < (q_n)_1$ are met, the $n + 1$ strata is the key strata of the overlay. According to the parameters in Table 1, the parameters of middle sandstone B, sandy mudstone C, and medium sandstone D in the overlay above the number 13 coal seam do not contribute to the load of the basic roof, middle sandstone A. The calculation results are shown in Table 2, and the four key strata are labeled as K1 to K4 for the convenience of the following description.

Table 2. Key strata evaluation.

$n + 1$ Strata	Medium Sandstone-A	Medium Sandstone-B	Sandy Mudstone-C	Medium Sandstone-D
$(q_{n+1})_1/\text{MPa}$	0.19	0.35	0.12	0.10
$(q_n)_1/\text{MPa}$	/	0.44	0.40	0.12
label	K1	K2	K3	K4

3.2. The Breaking Step of the Key Strata

Practice shows that the height of the caving zone in fully mechanized cave mining ranges from 4 to 6 times the mining height. Therefore, the key strata K1 and K2 are mainly considered when analyzing the collapse of the roof. The first panel is abstracted as the condition of a four-sided fixed support, and the pressure step of the first mine is calculated according to [20]:

$$a_1 = \frac{b}{\sqrt{2}l_m} \sqrt{b^2 - \sqrt{b^4 - 4l_m^4}} \quad (2)$$

where b is the width of the panel, m and l_m is the pressure step coefficient of the roof.

The key strata K1 and K2 are used as the basic roof, and the pressure steps where the first break occurs are 34.6 m and 41.2 m, respectively. After the initial break, the corresponding roof is calculated as a cantilever beam, and the periodic pressure step distances are 14.2 m and 16.8 m, respectively.

According to the theoretical calculation results above, the breaking steps of K1 and K2 above P13103 are similar when the initial weighting is applied. Compared with production experience under similar conditions, the LTC working face has a longer weighting time, which can last for 1–2 days. The mine pressure appears to be severe, with the dynamic load factor reaching 1.8 [7]. The average daily advance distance of P13103 was designed to be 4 m. Considering the hysteresis of roof collapse, the two key strata will break and interact, resulting in the superposition of mine pressure, which in turn leads to dynamic crush accidents [21].

4. Key Strata Structure Control

When pressure is applied for the first time, the span of the basic roof is generally relatively large and the scope of influence is far, which is the key stage in preventing roof collapse accidents. Especially with the combination of LTCLMH and MKS conditions, where the superimposed near-field MKS breaking-step distance is similar, it is necessary to control the overlying rock structure from the initial mining in order to avoid the direct impact on the working face caused by the breaking of the basic roof block after coal seam

mining, which acts by pushing down on the stope space and causes crush accidents. Although many mines have achieved good results by using deep-hole blasting and hydraulic fracturing for the initial roof caving, the efficiency of roof caving under the conditions of MKS is low and the effect is difficult to guarantee [22]. In addition, Shaping Coal Mine is a resource-integrated mine. There is an old mine fire area within 300 m of the open cut in front of the first mining working face, and the safeguarding measures of ground nitrogen injection and underground ventilation regulation were adopted. Avoiding the sudden increase in the height of the cave zone during the initial caving results in a connection between the working face and the fire area. From the aspect of the mining process parameters, the main consideration is to realize the control of the strata overlying the stope [23].

4.1. Low-Level Double Key-Strata Structure Control

The practice of backfill mining and slice mining shows that mining thickness is one of the most important factors affecting the migration of overlying rock [24]. Due to the caving and dilatation characteristics of the overlying rock above the gob [25,26], the thickness of the direct roof required to fill the gob is Σh .

$$\Sigma h = \varphi m / (k_p - 1) \quad (3)$$

According to the mining conditions at P13103, when the mining thickness m is 8.0 m, the recovery rate φ of the fully mechanized caving face is taken as 80%, the roof is medium hard, the bulking coefficient k_p is taken as 1.25, and the caving roof height is 25.6 m. The low-level K1 is not able to form a masonry beam structure and then transform into a direct roof, forming a cantilever beam structure near the stope. When K2 breaks, it is difficult for the K1 layer to form a stable structure to protect the stope space. According to this, it is necessary to reduce the mining height in the initial mining stage, use top-coal fragmentation filling to reduce the roof collapse angle of the goaf and promote the formation of a masonry beam structure in the low-level key strata (LKS) so the broken rock blocks are hinged to form an arch towards the front of the coal wall and mining face. The gravel in the open area transmits the load. When the HKS breaks for the first time, the low-level masonry beam structure is used to shield the stope space. Figure 2 shows the overburden structure models of the different initial mining schemes of LTCLMH.

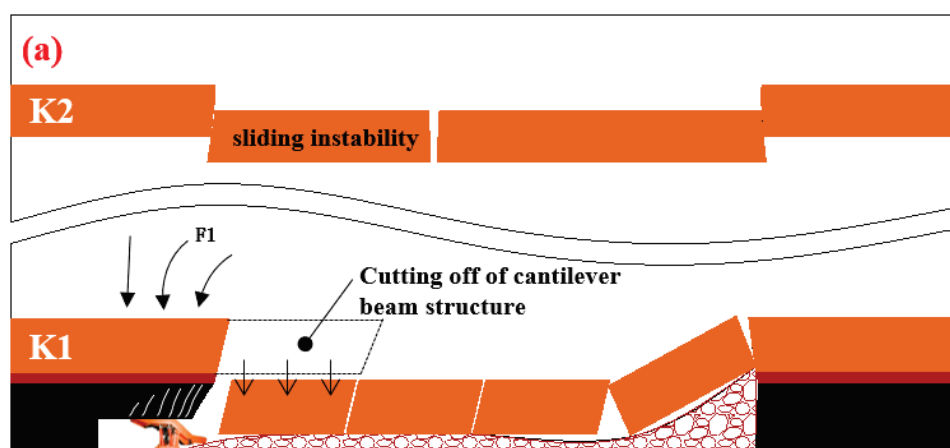


Figure 2. Cont.

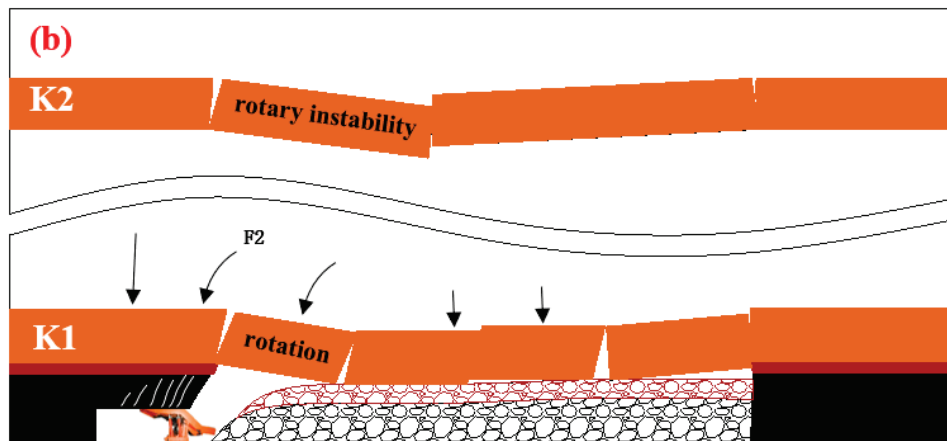


Figure 2. Rock structure model of different top-coal caving mining schemes under double key strata (a) Cantilever beam structure formed by top-coal caving with initial mining (b) The masonry beam structure formed by top-coal caving after the first weighting.

4.2. The Initial Mining Height Determined

According to the caving and dilation characteristics of the near-stope key strata [27–29], to promote the formation of a masonry beam structure in the LKS, it is necessary to satisfy the requirement that the thickness of the caving roof is less than the sum of the thickness of the LKS and the mining layer, as shown in Equation (4).

$$\frac{m\varphi}{k_p - 1} < \sum_{i=1}^n h_i + M - m\varphi \quad (4)$$

where M is the thickness of the coal seam, m ; m is the actual mining thickness, m ; φ is the recovery rate of the working face, k_p is the bulking coefficient of the rock, 1.15–1.30; and h_i is the thickness of each rock strata above the recovery seam, m .

According to the actual conditions of Shaping Coal Mine, the average thickness of seam number 13 in the initial mining area of 90 m is 8.0 m, and the average thickness of K1 is 8.7 m. The working face recovery rate φ is 93% for fully mechanized caving and 80% for LTC; for the existence of top coal, the bulking coefficient k_p is taken as 1.30, i.e., the mining thickness needs to be less than 4.1 m. Combined with the equipment conditions of P13103, it is finally determined that the mining height is designed to be 4.0 m without the top-coal caving process in the initial mining stage.

5. Numerical Simulation

Coal caving generally starts at about 20 m in the initial mining stage. According to Section 3.2 and 4.2, the initial mining plan for the prevention and control of severe weighting under MKS in P13103 of Shaping Coal Mine is as follows: the initial mining range is about 40 m and the height is 4 m without top-coal caving. After first weighting, the top-coal caving was gradually started, fully caving at 60 m with the mining–caving ratio of 1:1. Based on this, numerical simulations were used to analyze the strata movement characteristics.

In order to reflect the characteristics of the undulating surface and the mining height transition of the working face in detail, the FLAC3D numerical model was established based on the strata histogram of P13103 and the surface topography. The model was 350 m in the X direction and 350 m in the Y direction, the Z direction was modeled on the surface, the displacement was limited around the area and at the bottom, and a 50 m boundary coal pillar was left. Stress boundary conditions were set according to the in situ stress test results. The model is shown in Figure 3. In Scheme A, with the initial advance distance of 20 m, the top coal was mined for 4 m without caving, after which the coal caving height gradually increased, with full caving beginning as the advance distance reached 40 m

with a mining–caving ratio of 1:1. In Scheme B, with the initial advance distance of 40 m, the top coal was mined for 4 m without caving, after which the coal caving height gradually increased, with full caving beginning as the advance distance reached 60 m with a mining–caving ratio of 1:1.

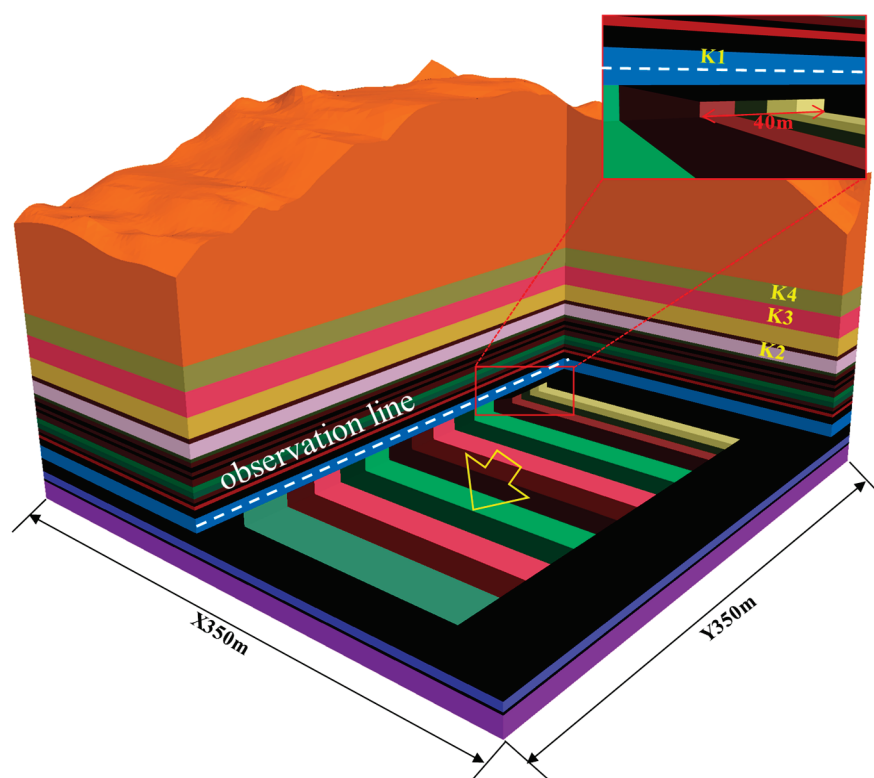


Figure 3. Numerical model and grouping.

The Mohr–Coulomb model was used for the coal strata. Combined with Table 1, the parameters of the complete rock sample were converted into rock mass parameters by RQD to characterize the mechanical response of the original rock [4,30]. The Null model was used for step-by-step excavation, the large strain calculation mode was turned on, all calculations were carried out to equilibrium and the displacement and stress were transferred between layers through the ‘interface’ [31].

5.1. Initial Mining Pressure Characteristics

In the step-by-step excavation process, hydraulic supports are grouped in the range of 2–8 m behind the coal wall by FLAC3D. The top and bottom beams are elastic models [32]. The Mohr–Coulomb constitutive model for characterizing the support performance of hydraulic supports was used [33]. The support strength was calibrated to 1.4 MPa, according to the support parameters, as shown in Figure 4.

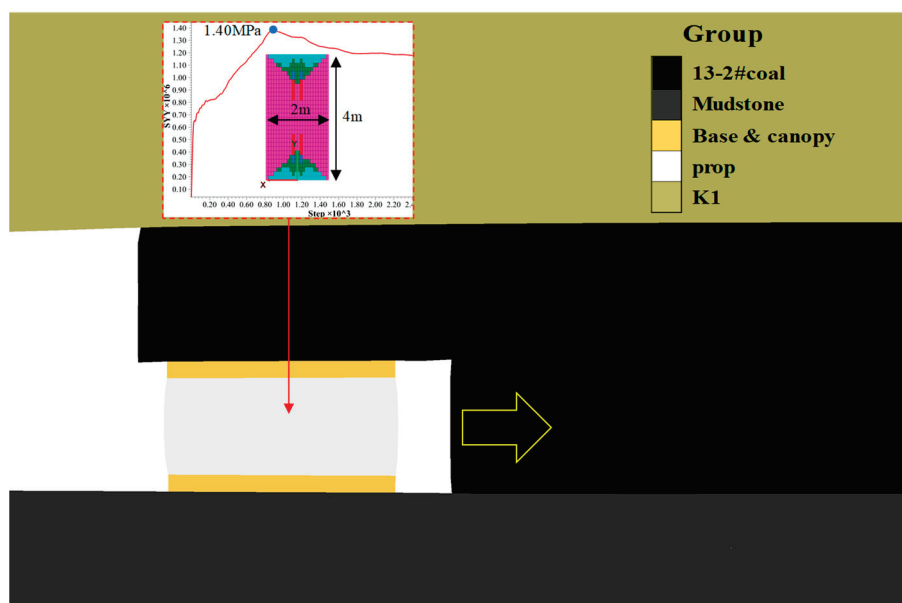


Figure 4. Hydraulic support grouping and mechanical calibration in numerical simulation.

The middle part of the working face with the most severe mining pressure was selected for monitoring. The results of the mining pressure characteristics of the stope corresponding to Scheme A and Scheme B are shown in Table 3. Comparing the two schemes at the same advance distance, it can be seen that the advance abutment pressure and the damage ratio [34–36] of the support in Scheme A are significantly greater than those in Scheme B. In Scheme A, the damage ratio of support is as high as 74% when recovery is 60 m, which is prone to inducing support crushing accidents. However, the damage ratio of support does not exceed 50% in Scheme B, and the vertical stress of the coal wall in the advance position does not increase significantly.

Table 3. Characteristics of ground pressure in the initial mining stage.

Scheme	Advance Distance/m	Damage Ratio/%	Advance Abutment Pressure before Coal Wall 5 m/MPa
A	40	58	15.8
	60	74	20.2
B	40	44	12.1
	60	50	13.7

5.2. Mining Response Characteristics of the Key Strata of Scheme B

Since Scheme B had a better effect on the prevention and control of severe mining-induced pressure, the mining response of the key strata in the initial mining stage was analyzed. During the initial mining process of 60 m, the K1 had a plastic zone from the open cut. When the mining distances were 20 m and 30 m, shear failure continued to occur behind the working face. After 40 m of mining, the central part of the gob experienced a tensile-shear failure, the boundary of the gob mainly shear failure, and the roof collapsed, corresponding to the first weighting when the mining height was 4 m. After the working face was mined for 60 m, the roof plastic zone developed ahead of the working face, which is generally considered to be caused by the cantilever beam structure, as shown in Figure 5.

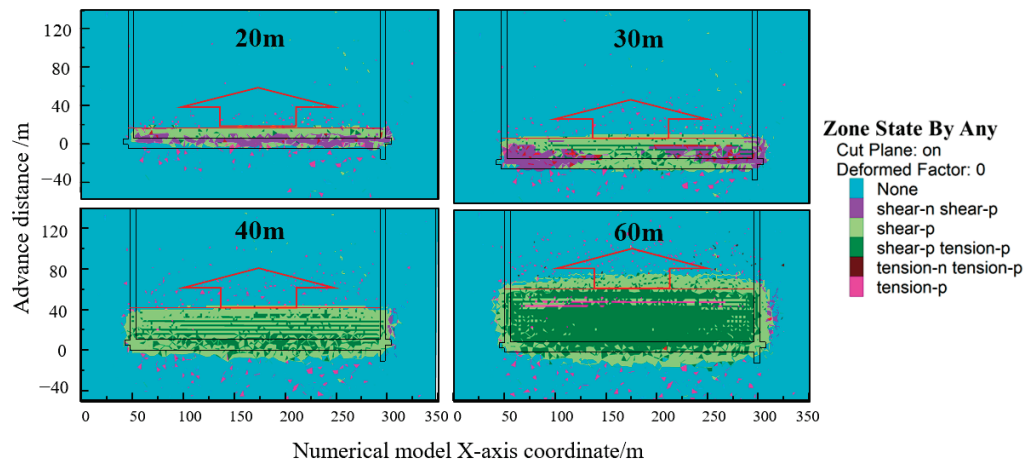


Figure 5. Development characteristics of mining plastic zone in K1 from open cut.

For the K2 strata, when the advance distance was 40 m, the plastic zone developed sporadically, resulting in damage. When the advance distance was 60 m, the roof suffered plastic failure. The lag of the working face was about 15 m, and the damage occurred in the middle of the goaf, which proved that the K2 did not suffer from shear-off failure, as shown in Figure 6.

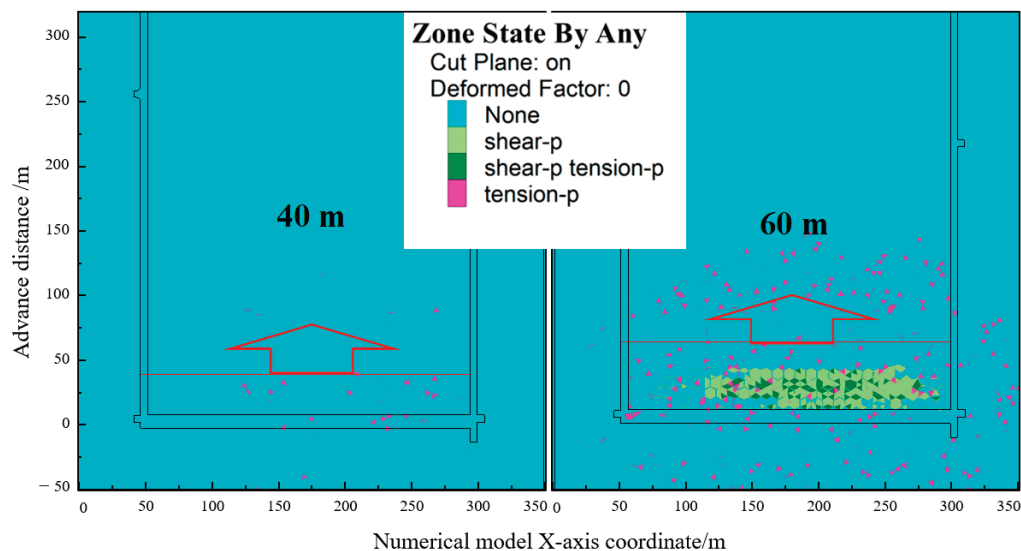


Figure 6. Development characteristics of mining-induced plastic zone in K2.

The observation line was arranged in K1 by Tecplot to monitor the vertical stress and displacement of the mining state with the typical plastic failure of the key strata in the initial mining process of P13103 (Figure 3). The results are shown in Figure 7.

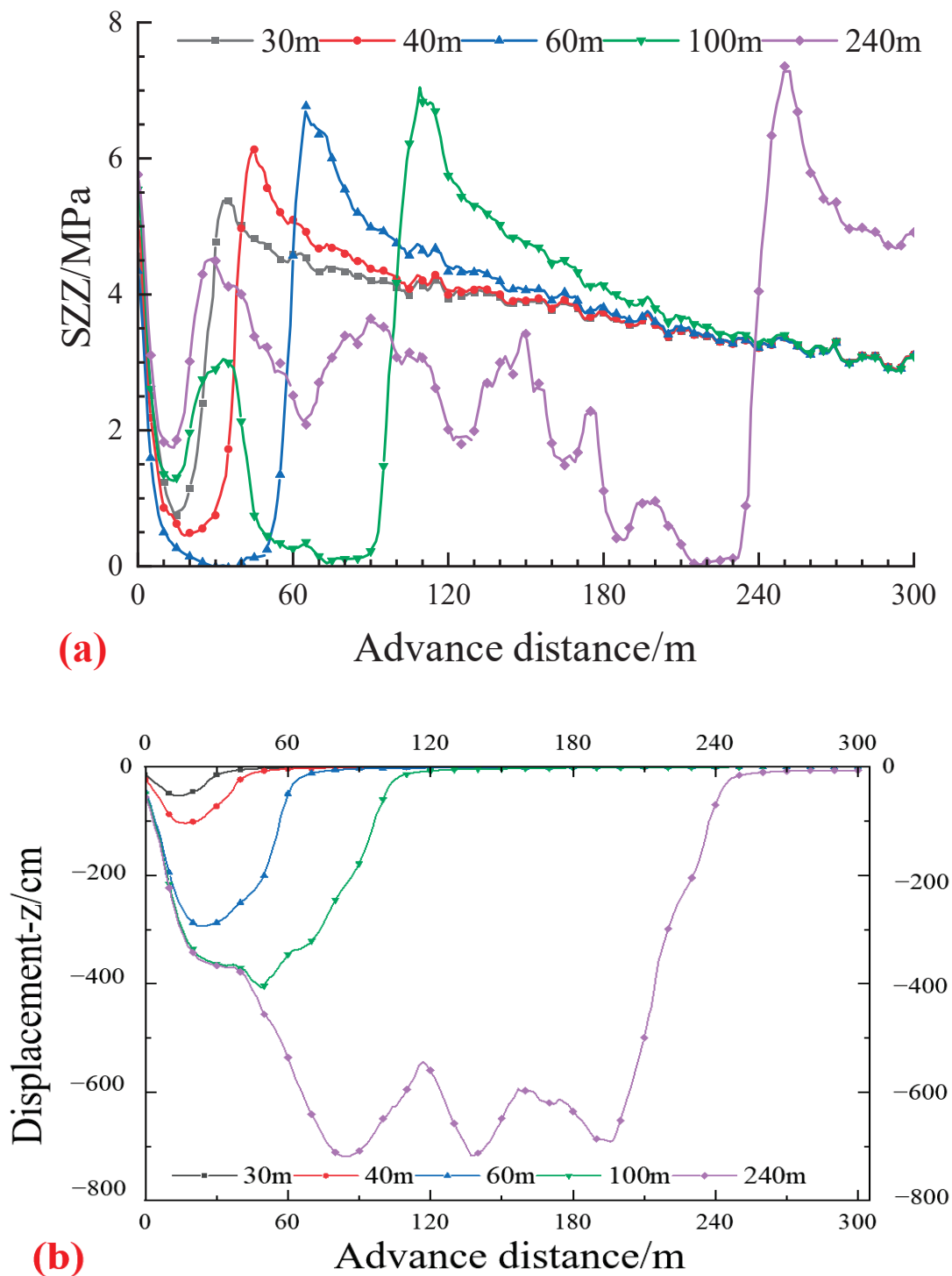


Figure 7. Mining response characteristics of K1 above the middle area of P13103 (a) vertical stress distribution (b) vertical displacement distribution.

From Figure 7a, it can be seen that with the working face advancing, in the process recovery distance of 30–60 m, the peak value of the advanced abutment pressure increases, and the influence range of abutment pressure expands gradually. The peak value of stress in the advance position changed little after the recovery distance to 60 m and stabilized. When the recovery distance reached 100 m, the gob compaction stress at the open cut side gradually recovered. When recovery distance reaches 240 m, the gob presented a square shape and the ground pressure appeared violently. The gob compaction stress at the

open cut side basically recovered to the original rock stress, and the stress concentration coefficient at the advanced position was greater than 2. In Figure 7b, in the process of the recovery distance of 30–60 m, the vertical displacement increased rapidly, the displacement of recovery distance to 30–40 m increased to twice the value, the maximum subsidence reached 1 m, and K1 first collapsed. When the displacement of 40–60 m was doubled, the maximum subsidence was 3 m. Compared with the development characteristics of the plastic zone, when K2 was first broken, the caving zone increased rapidly, and the mine pressure behavior was more intense than the initial weighting. When mining to 100 m, the maximum roof subsidence was close to the initial mining height, with the roof caving fully. The gob behind the working face was gradually compacted when mining to 240 m, and the maximum displacement in the middle of the gob was greater than 7 m.

The analysis shows that under the current equipment conditions, Scheme A is not conducive to the stability of the roof structure of the LKS, as it is easy to induce dynamic load accidents such as support crush accidents. It is an effective means for the prevention and control of strong mine pressure to select the top-coal caving after the initial weighting in the initial mining and caving stage of the extra-thick coal seam. The bedding rock was broken from the bottom to the top, and the MKS were broken layer by layer, resulting in successive weighting. The second weighting intensity was greater than the first weighting, and then the strata behavior of the working face tended to be stable.

6. Field Measurement of Mine Pressure

The initial mining was organized according to Scheme B, and the prop resistance of the hydraulic support in the working face was monitored. It was observed that the pressure changes near entries of the working face were not obvious, and the pressure in the range of 30–110 # support in the middle was obvious. The mine pressure was monitored every 10 supports. P13103 stops from the open cut; the first four weightings are shown in Table 4.

Table 4. Working face pressure in the initial mining stage.

Roof pressure order	1	2	3	4
Average pressure step/m	41.0	14.7	12.3	10.7
Duration/h	6–26	5.5–48.6	9–49.2	4.5–46.2
Support number	Dynamic load factor			
30	1.35	1.45	1.14	1.10
40	1.33	1.46	1.22	1.24
50	1.36	1.59	1.27	1.43
60	1.50	1.62	1.36	1.37
70	1.41	1.32	1.34	1.19
80	1.27	1.41	1.16	1.29
90	1.25	1.50	1.35	1.33
100	1.18	1.28	1.32	1.28
110	1.28	1.54	1.38	1.40
Average dynamic load factor	1.32	1.46	1.28	1.29

According to the statistical data, the initial breaking step of K1 is 41 m, which is consistent with the theoretical analysis result. Affected by the coal seam dip angle, the maximum dynamic load coefficient along the working face direction is distributed in supports 40–70, which is located in the middle and lower parts of the working face. In the process of mining 90 m forward from open cut, the statistical average dynamic load coefficient first increases and then decreases, and the average dynamic load coefficient of the second weighting is the largest. After the first pressure, the pressure duration

increased. The rock pressure characteristics show that the height of the caving zone increases significantly and tends to be stable after the first weighting. The dynamic load formed by the first breaking of K2 is greater than K1. The rock pressure of the second weighting stope is the most intense. The safety valve of the support in the middle of the working face is partially opened, and the maximum depth of the rib spalling of the coal wall is 0.6 m. There is no roof falling in front of the support and shrinkage of props. It effectively avoids the dynamic pressure disaster when the first roof caves, and realizes the safe mining of the working face.

7. Discussion

The findings of this paper clearly show the formation principle, prevention and control practice of strong mine pressure in the initial mining and caving stage with MKS in a shallow, extra-thick coal seam. The analysis results show that the breaking distance of low double key strata is similar. Because of the strong mine pressure that occurred in adjacent mines with similar geology and equipment conditions, a targeted mining scheme is proposed.

Generally, the basic roof presplits before first weighting, which can effectively reduce the breaking step of the roof and realize the prevention and control of severe mine pressure. Under complex conditions, such as MKS and mining under fire area, top-coal caving after the initial weighting should be adopted, and the initial mining height should be determined according to the overlying structure, breaking roof structure and bulking characteristics so as to promote the transformation of the broken basic roof from cantilever beam to masonry beam structure that forms a shield and avoids the strong mine pressure impact stope formed by the MKS breaking interaction.

The interaction of low-level double key strata is closely related to factors such as mining thickness, interlayer height and strata structure. In the case of small mining thickness and large interlayer height, the traditional near-field key-strata-breaking dominant mine pressure is still applicable. The study of the MKS interaction criteria is a topic worthy of exploration in LTCLMH.

In the process of the numerical simulation analysis, the subsidence value of the basic roof is almost equal to the mining height, which indicates that the finite element simulation method is insufficient to characterize the bulking characteristics of the gob and the caving zone. The strata in the caving zone that adopt the double-yield constitutive model can better reflect the surface subsidence and the front abutment pressure distribution characteristics, while the mining response of the basic roof inside the caving zone is ignored. Therefore, the mining response in the caving zone is worth exploring in the future.

The research results are limited by the finite element numerical simulation method. Although the large strain and the contact problem between the layers are fully considered in the calculation, there are deficiencies in the analysis of the dilatancy in the caving zone and the breaking roof structure. The promotion of the new finite discrete element calculation method is expected to improve the accuracy of the analysis of such problems.

8. Conclusions

In this paper, the prevention and control principle of strong mine pressure in the initial mining and caving stage under the condition of MKS was analyzed. The engineering application under the geological conditions of MKS and the overlying fire zone shows that the strata behavior of the mining scheme is relaxed. The main conclusions are as follows:

- (1) The basic roof structure is the key to the control of strong mine pressure in the initial mining and caving stage;
- (2) The calculation formula of the initial mining height to form the masonry beam structure after the initial breaking of the basic roof is proposed and the breaking roof structure model under two mining schemes was established;
- (3) The duration of mine pressure tends to be stable with the height of the caving zone fully developed;

- (4) The mining scheme of gradual coal caving after the first weighting of P13103 realizes safe mining in the initial mining and caving stage with MKS.

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Glossary

h	thickness, m
ρ	density, m
E	elastic modulus, GPa
μ	Poisson ratio, 1
T_l	tensile strength, MPa
b	width of the panel, m
l_m	the pressure step coefficient of the roof, 1
φ	the recovery rate, %
k_p	the bulking coefficient, 1
m	the mining thickness, m
M	the thickness of the coal seam, m
h_i	the thickness of each rock strata above the mining coal seam, m
MKS	multiple key strata
P13103	the 13103 Panel
LTC	longwall top-coal caving
LTCLMH	longwall top-coal caving with large mining height
HKS	the high-level key strata
RQD	rock quality designation
LKS	the low-level key strata

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Article

The Spatial Evolution Law and Water Inrush Mechanism of Mining-Induced Overburden in Shallow and Short Coal Seam Group

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Abstract: In order to grasp the overburden evolution law of the shallow and short coal seam group, based on the key bed theory, a mechanical analysis of the longitudinal expansion of mining-induced overburden fractures in the coal seam group was carried out, and the spatial evolution characteristics of mining-induced overburden fractures were simulated by the numerical simulation method. The results show that in the process of working face advancement, with the continuous instability and failure of the overburden, the size and shape of the fracture network are also changed. The repeated mining of the lower coal seam further causes the secondary activation of the upper overburden, which makes the roof fractures of the partially compacted goaf violently move again. The “channel source” and “space source” continue to carry out the process of “generation-expansion-compression-generation-expansion”, in combination with pore fracture elastic theory. The water inrush characteristics of the whole coal seam are divided into three “solid-liquid” coupling stages: the original gap seepage stage, the initial water discharge stage of mining fissures and the water inrush stage of fractured rock mass. The steady value of water inflow and its variation characteristics with time are predicted by using the formula of deep well flow in a confined aquifer.

Keywords: shallow coal seam; overburden failure; water inrush factor; numerical simulation; water inrush mechanism

1. Introduction

The coal seams in the mining areas of central and western China generally have the characteristics of shallow buried depth, thin bedrocks, multi-layer coal and small seam spacing. These characteristics can be collectively referred to as shallow and short coal seam group extract. First, the underlying bedrock thickness of coal seam is small, and the burial depth is no more than 150 m, so it is easy to break through under the influence of coal mining. Secondly, due to the small interval between layers in the short coal seam group, the coal seam mining seams will have a great influence on each other, and the repeated mining will continue to affect the development characteristics of overburdening. Whether there will be a water inrush phenomenon in coal seam mining is mainly affected by the following main control factors: water inrush source, abundance of water source and water pressure, water inrush channel and stagnant water space, which can be referred to as the “four source elements”, including the water inrush source, dynamic source, channel source and spatial source. When the “four source elements” constitute a complete relationship, water inrush will occur in the working face. For the problems of overburden failure and

water inrush in coal seam mining under shallow buried conditions, Huang Qingxiang [1–3] studied the ore pressure characteristics of shallow buried coal seams and proposed the theory of “double key layers” in the roof of a near-shallow buried coal seam with large mining height. Jia Mingkui [4] studied the failure height of the overburdening after mining of thin bedrock working face and made clear the conditions of water and sand gushing. For shallow coal seam mining, Yang Dengfeng [5] studied the instability mechanism of overburdened rock under load using catastrophe theory and analyzed the main influencing factors of overburdened rock instability. Fan Limin et al. [6,7] studied the strata control and water conservation mining technology of coal seam mining under the condition of shallow burial in the ecologically fragile region of western China. Liu Quanming [8] studied the buried depth effect of the overburdened migration rule in fully mechanized caving mining under the condition of shallow buried depth and thin bedrock by using the method of a similar simulation experiment. More research focused on single coal seam mining under the condition of impact, for the shallow buried close distance coal seam group mining roof damage rule and the study of water inrush mechanisms is not very perfect, so that under the condition of close distance coal seam group occurrence, a shallow buried coal seam repeated mining group will cause the overburden rock fracture channels and severe changes of the space, causing a water inrush mechanism different from those of single coal seam mining conditions. Therefore, it is of great significance to study the failure law of roof overburdening and the water inrush mechanisms of shallow and short-distance coal seam group mining (Figure 1 research methodology flowchart) and grasp its evolution law to prevent the occurrence of water disasters.

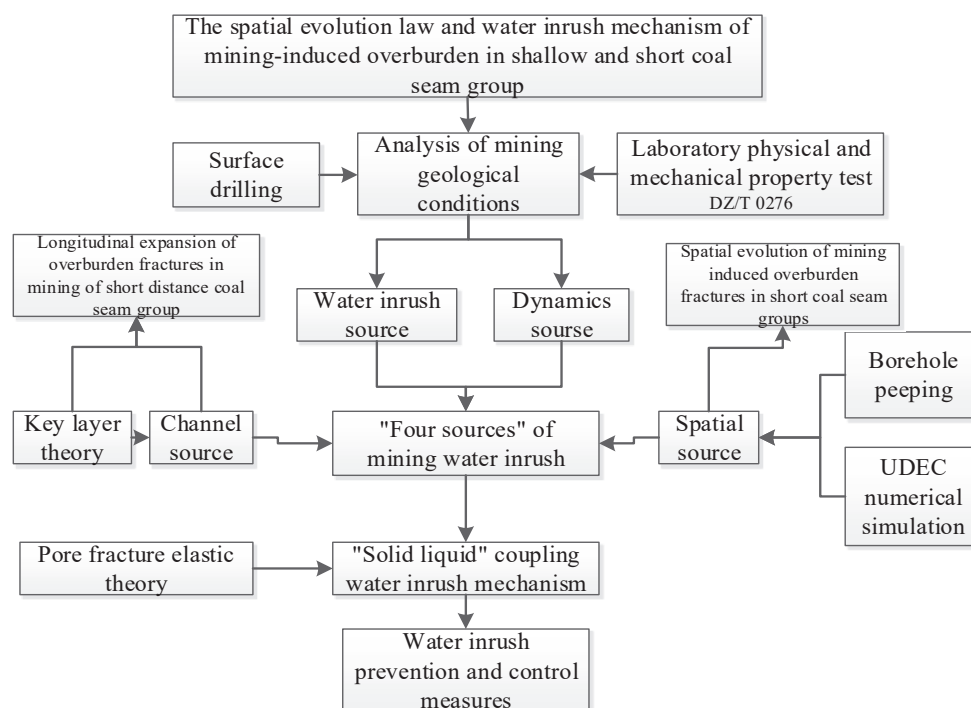


Figure 1. Research methodology flowchart.

2. Overview of the Study Area

Xialiyuan Coal Mine belongs to the Pingshuo Mining Area. Coal 4 and 9 of the Taiyuan formation are mainly mined in the mine field. The second mine area's first working face of 4 coal is a 40,201 working face. The extension length is 765 m, the dip length is 150 m and the average thickness of the coal seam is 11.2 m with a dip angle of $2^{\circ}\sim 5^{\circ}$. The structure of the coal seam is simple and stable, the roof contains K3 and K4 sandstone aquifers, the water yield is weak, the average thickness of the bedrock is 100 m, the average thickness of the Quaternary is 30 m, the water yield is medium and the spring water is exposed in

the low-lying part of the ground. The average thickness of a 9 coal seam is 8 m, which, under the 4 coal and the coal seam spacing, is 30.50 m. The fully mechanized top coal caving mining method is adopted in the working face. According to the construction drilling results and the laboratory physical and mechanical property test results of the construction rock samples during the mine field exploration (the test is based on DZ/T 0276 rock physical and mechanical property test specification), the statistical regional borehole columnar characteristics and the physical and mechanical parameters of each coal and rock stratum are shown in Table 1.

Table 1. Physical and mechanical parameters of coal and rock.

Rock Number	Lithology	Thickness (m)	Density (kg/m ³)	Elastic Ratio (GPa)	Poisson's Ratio	Cohesion (MPa)	Internal Friction Angle (°)	Uniaxial Tensile Strength (MPa)
1	Loose strata	30	1612	0.79	0.33	0.6	45	0.43
2	Sandy mudstone	16	2364	1.56	0.35	1.37	26	0.89
3	Medium grained sandstone	4.23	2511	3.12	0.39	2.55	48	2.21
4	Siltstone	4.56	2383	2.27	0.33	1.65	27	1.27
5	Sandy mudstone	12	2364	1.56	0.35	1.37	26	0.89
6	Siltstone	11.5	2383	2.27	0.33	1.65	27	1.27
7	Sandy mudstone	10.4	2364	1.56	0.35	1.37	26	0.89
8	Fine grained sandstone	3.9	2420	5.1	0.29	2.36	23	1.83
9	Sandy mudstone	24.8	2364	1.56	0.35	1.37	26	0.89
10	Medium grained sandstone	9.63	2511	3.12	0.39	2.55	48	2.21
11	Sandstone	2.2	2464	1.26	0.35	1.37	23	0.67
12	4 coal	11.2	1320	1.26	0.36	0.83	19	0.72
13	Sandy mudstone	12	2364	1.56	0.35	1.37	26	0.89
14	Medium grained sandstone	6.75	2511	3.12	0.39	2.55	48	2.21
15	Fine grained sandstone	4.6	2383	2.27	0.33	1.65	27	1.27
16	Sandy mudstone	7.15	2364	1.56	0.35	1.37	26	0.89
17	9 coal	8	1320	1.26	0.36	0.83	19	0.72
18	Sandy mudstone	17.6	2364	1.56	0.35	1.37	26	0.89

According to the above “four source elements” of water inrush. K3, K4 and the Quaternary aquifer in Xialiyuan coal mine are all water inrush sources. The Quaternary aquifer has a medium water yield and is a stable drastic source. Once qualified channel and spatial sources are produced in coal seam mining, water inrush may occur in the working face. Therefore, by studying the spatial evolution characteristics of mining overburdening in the coal seam, the occurrence process of channel source and spatial source can be obtained, and then the law of water inrush can be mastered and prevented in the future.

3. “The Channel Source”—Mechanical Analysis of Longitudinal Propagation of Mining-Induced Overburden Fractures in Short Coal Seam Group

The strata above the direct roof of coal seams are the key strata that control the activities of the whole overburden in the stope or to the surface, so the structural position of the key strata can affect the overall development height of the mining-induced overburden. When the main key strata is located under the overburden and close to the coal seam, and is less than a critical distance, because the compressible rotary space resistance of the lower part of the main key strata is relatively large, the subsidence and rotation of the structure block are larger when the key strata is broken, and the fracture of the key strata can lead to the larger opening of the rock fracture of the strata, which extends to the top of the overburden.

3.1. Key Strata Structure of Overburden in Coal Seam Group

According to the key strata theory [9–11], when in the rock strata above the coal seam $(q_{n+1})_1 < (q_n)_1$, the first hard rock layer and the $N + 1$ rock layer can both be the key strata, controlling the synchronous fracture and subsidence of other soft rock strata. When $L_{n+1} < L_1$, above the coal seam, only the first hard strata is the key strata, which is a single key stratum. When $L_{n+1} > L_1$, there are at least two key layers in the overburden: the upper one controlling the upper layer to the surface of the hard rock as the main key layer and the lower hard rock as the sub-key layer. According to the judgment criterion of key strata, and referring to the columnar structure of boreholes and the physical and mechanical parameters of strata in Table 1, quantitative analysis and judgment are made on all strata in the working faces of Xialiyuan Coal Mine. The judgment results are as follows: the rock No. 10 (medium-grained sandstone, 9.63 m thick) is the main key layer, and the rock No. 14 (medium-grained sandstone, 6.75 m thick) is the sub-key layer.

In the coal seam No. 4 mining, the main key layer (rock No. 10, referred to as key layer 1) plays a dominant role in controlling the activities of the overburden (Figure 2). When the coal seam No. 4 is exhausted, the coal seam No. 9 is mining, and the sub-key layer (rock layer No. 14, referred to as key layer 2) becomes the main key strata of the overburden of the coal seam No. 9. The key strata structure has changed a single key layer of the upper structure (Figure 3), which can present typical rock pressure characteristics of strata behavior.

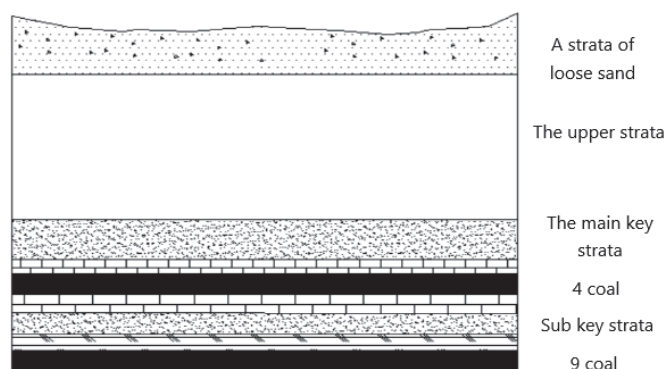


Figure 2. Key strata structure of overburden of coal seam group before mining.

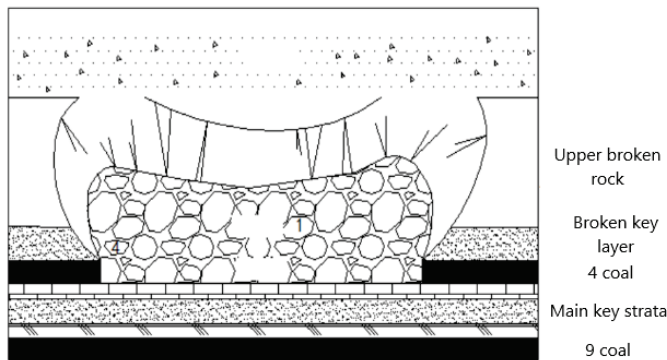


Figure 3. Key strata structure of overburden in coal seam No. 4 after mining.

3.2. Failure Mechanism of Key Strata in Coal Seam Group

There are two types of overburden strata failure instability [12]: Structure slide instability (S instability, shear failure) and structure deformation instability (R instability, tension bending failure).

3.2.1. Shear Failure Condition

The shear failure condition of overburden is when the working face is pushed to the cracking end of overburden, the middle part of the strata has not cracked and the shear stress of the remaining section has exceeded the ultimate strength. In this case, when there is a small amount of space under the strata, the overburden will be cut off as a whole, resulting in the overburden being cut off as a whole in the goaf [13,14]. According to the rock beam stress analysis and material mechanics [15], the shear failure expression formula is as follows:

$$L_c = \frac{4M_c[\tau]}{3(M + \sum_{i=1}^n M_i)\gamma} \quad (1)$$

If $\sum_{i=1}^n M_i = 0$, there is:

$$L_c = \frac{4M_c[\tau]}{3M\gamma}$$

where L_c —shear distance of rock strata, m; M_c —residual thickness of rock beam end after cracking, m; $[\tau]$ —Shear strength of rock, MPa; M —actual thickness of rock beam, m; M_i —thickness of the following layer above the rock beam, m; γ —average bulk density of rock strata, MPa.

3.2.2. Tensile Bending Failure Condition

The bending failure condition of the overburden on the roof is as follows: With the continuous advance of the stope, the overburden (assumed to be plastic strata) is exposed, and under the action of overlying weak strata and its own gravity, the strata are bent and subsided. The original fixed beam is transformed into a simply supported beam. With the maximum bending moment transferred from the end of the beam to the middle of the beam, the middle part of the beam begins to crack and fall, and finally forms a “pseudo plastic rock beam”; if the height of the lower free space of the rock beam exceeds its allowable settlement value, the rock beam will collapse and the tensile bending failure and instability will occur. The expression formula is:

$$S_0 < S_n = h - \sum_{i=1}^{n-i} M_i(K_A - 1) \quad (2)$$

where S_0 —Allowable settlement value of “pseudoplastic beam”, m; S_n —The free space height that allows the movement of the rock, m; K_A —Dilatancy coefficient of caved rock; M —Thickness of caving rock, m; h —Mining height, m.

3.2.3. Theoretical Criterion of Overburden Failure form

When $L_c > L_0$, the rock strata are pulled away from the middle.

When $L_c = L_0$, the strata are sheared.

Where L_c —Rock shear distance, m; L_0 —Limit span at first crack of beam end.

3.3. Mechanical Analysis of Longitudinal Propagation of Overburden Fracture in Coal Seam Group Mining

According to the analysis of the strata structure in Xialiyuan coal mine, there are two key strata that play a key role in controlling the whole overburden in the coal seam group working face. The fracture instability characteristics of two key layers, key layer 1 and key layer 2, are analyzed.

In coal seam mining, when the overlying key layer reaches the limit span, it will not collapse immediately, but will form a three hinged arch structure. During the continuous advancement of the mining face, the stability of the three hinged arch structure is mainly controlled by two key blocks. The interaction of the two key blocks presents two failure forms: sliding instability (s) and deformation instability (R).

3.3.1. Failure Judgment of Key Layer “S-R”

Judgment of Key Stratum Sliding Instability (S)

After the initial fracture of the key strata, the two rock blocks form an arch equilibrium state (Figure 4). The horizontal extrusion force T can be considered as the action point at $a/2$ (Figure 5), the load borne by the key strata is p and the length and height of the rock block are l and h , respectively. The moment balance is obtained for point O to obtain the balance curve formula without sliding instability:

$$i \leq \tan \varphi + \sin \alpha \quad (3)$$

$$\sin \alpha = [M - \sum h(k_p - 1)] / l \quad (4)$$

where i —Fracture degree of rock block, ratio of height H to length L of fracture block; $\tan \varphi$ —The friction coefficient between rocks is 0.3; α —Block rotation angle, M —Mining height; H —Height of rock strata falling; K_p —Breaking expansion coefficient, take 1.5; L —Length of fractured rock block.

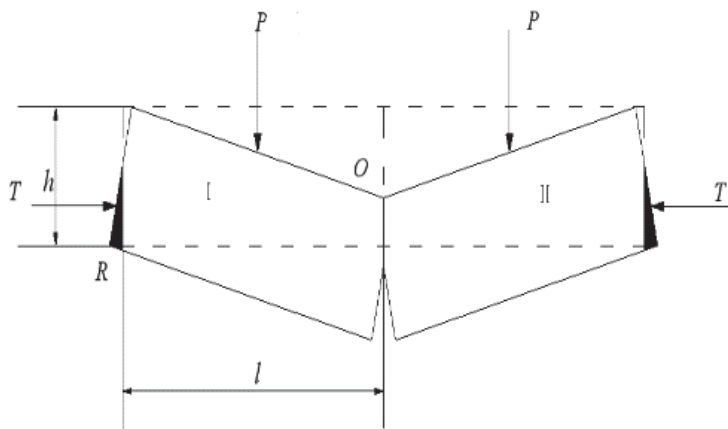


Figure 4. Schematic diagram of three hinged arch structure.

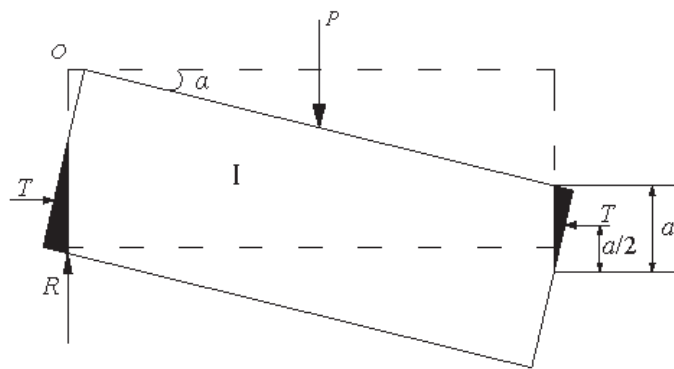


Figure 5. Schematic diagram of stress analysis of key blocks.

Rotation Instability of Key Layer (R)

According to the relationship between T/P and the rotation angle α , when the fracture degree is constant, the horizontal thrust T between the key blocks increases with the increase of the rotation angle, which will lead to the crushing of the corner blocks and deformation and rotation instability. In order to avoid the deformation and instability of the key blocks, the following conditions should be met:

$$\frac{T}{a} \leq \eta \sigma_c \quad (5)$$

where $\sigma_p = T/a$ —The extrusion stress formed at the bite; η —The extrusion coefficient at the end angle of the key block, $\eta \sigma_c$ is the extrusion strength, η is 0.3.

According to the above formula, the judgment curve formula of rock block without rotary instability is:

$$i \geq \sin \alpha / (1 - \sqrt{\frac{nk\eta}{3}}) \quad (6)$$

where n —Ratio of compressive strength to tensile strength of rock; k —Calculated as simply supported beam, $k = 0.3$; η —Ratio of extruding strength to compressive strength of rock block, η is 0.3.

According to the above analysis, the instability form of rock block mainly depends on the fracture degree and rotation angle of the block.

Judgment of Instability form of Key Strata

① Judge the instability form of key strata 1 and substitute the relevant parameters of the above judgment formula: $\tan \varphi$ is 0.3; H is 9.63 m; M : The mining height of coal seam No. 4 is 11.2 m; h is 14 m; k_p is 1.5; l is 12.14 m.

Calculated $i = 0.793$, $\tan \varphi + \sin \alpha = 0.646$, $i > \tan \varphi + \sin \alpha$, Therefore, key strata 1 will slip and lose stability.

$$\sin \alpha / (1 - \sqrt{\frac{nk\eta}{3}}) = 0.544,$$

$$i \geq \sin \alpha / (1 - \sqrt{\frac{nk\eta}{3}})$$

Therefore, there will be no rotary instability in key strata 1.

② Judge the instability form of key strata 2, and substitute the relevant parameters of the above judgment formula: $\tan \varphi$ is 0.3; H is 6.75 m; M : The mining height of coal seam No. 9 is 8 m; h is 11.5 m; k_p is 1.5; l is 10.3 m.

Calculated $i = 0.655$, $\tan \varphi + \sin \alpha = 0.518$, $i > \tan \varphi + \sin \alpha$, Therefore, key strata 2 will slip and lose stability.

$$\sin \alpha / (1 - \sqrt{\frac{nk\eta}{3}}) = 0.334$$

$$i \geq \sin \alpha / (1 - \sqrt{\frac{nk\eta}{3}})$$

Therefore, there will be no rotary instability in key strata 2.

3.3.2. Longitudinal Development Characteristics of Mining-Induced Overburden Fractures in Close Coal Seam Groups

Through the instability of key strata fracture mechanism analysis, the mining face in the working face end composite beams above will produce open fractures, and gradually, from the bottom up, when the rotary rock fracture gradually increases, the main roof structure sliding instability occurs, being exposed for the first time to pressure, formatting a water bursting fracture source “channel”, pushing the continuous mining face forward. As a result, the key rocks in the overburden roof break periodically quickly, and the longitudinal fractures in the overburden rock continue to develop upward through the wave base rock. During the mining of coal seam No. 9 in the lower group, the fracture instability of the key strata will directly lead to the fracture of the roof rock in the goaf of coal seam No. 4, resulting in the secondary activation of the upper overburden, resulting in the violent activity of the roof cracks in the partially compacted goaf again, as well as the expansion of the crack channel and the secondary development of overburden cracks. Longitudinal development model of overlying rock fissure in close coal seam mining (see Figure 6).

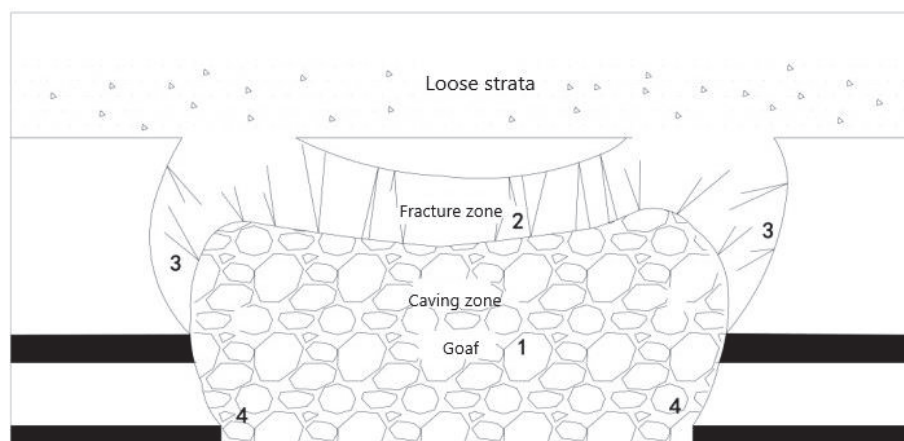


Figure 6. Longitudinal development model of overlying rock fissure in close coal seam mining. 1. Tensile failure zone. 2. Tensile fracture zone. 3. Shear failure zone. 4. Tensile shear failure zone.

4. “Spatial Source”—Spatial Evolution Characteristics of Mining-Induced Overburden Fractures in Short Distance Coal Seam

The structure of goaf and overburdening after mining can be divided into relatively complete layered structure, massive structure and broken structure, according to the degree of fragmentation and form, and the overall combination mode is closely related to the size of the water accumulation space. In the original strata of mining, the water accumulation space mainly exists in rock micro fractures. After coal mining, the violent development of overburden fractures leads to the rapid expansion of the water accumulation space, and rocks in the overburden fracture zone are massive structure rocks (see Figure 7), so the water accumulation space is mainly distributed in the pore fracture channel. The rock mass in the caving zone, which has been seriously damaged, presents a broken structure, and the overburden space is fully developed, as is the main water accumulation area. The spatial evolution characteristics of mining-induced overburden fractures are simulated by numerical simulation.

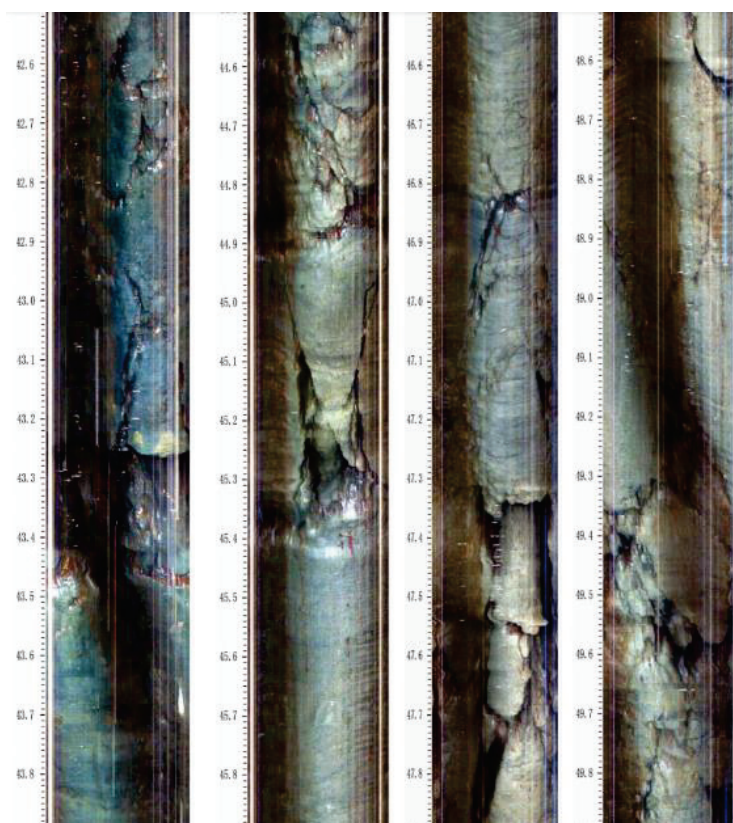


Figure 7. Borehole television map of fracture development in overburden after mining.

The following 40,201 working face area of Xialiyuan Coal Mine is taken as an example. According to the strata distribution and physical and mechanical parameters of coal and rock in Table 1, UDEC numerical simulation software is used to establish a numerical analysis model to simulate the spatial variation characteristics of overburdened fractures in the process of coal seam group mining (see Figures 8 and 9) [16].

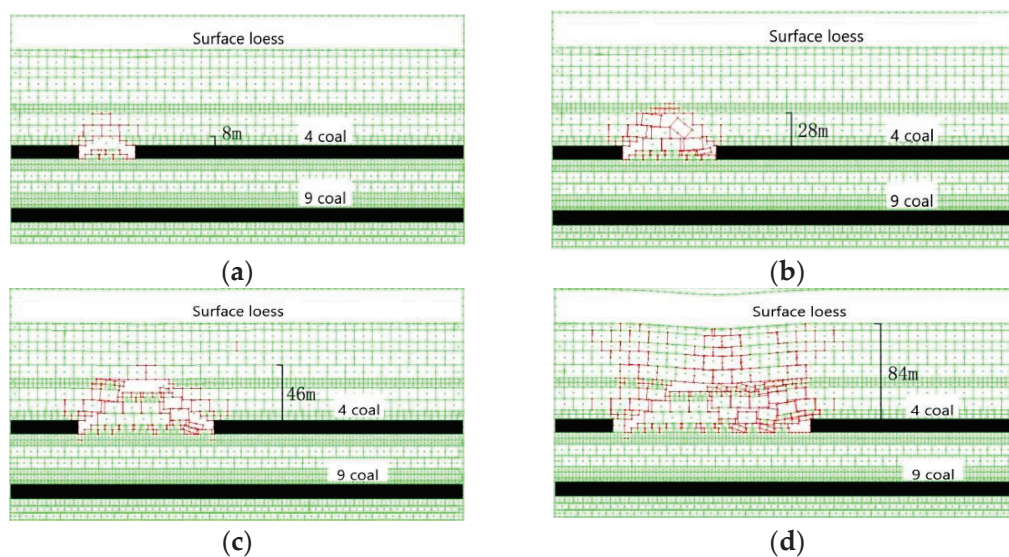


Figure 8. Spatial variation characteristics of overburden in single coal seam of coal seam No. 4. (a) Working face advance 20 m. (b) Working face advance 40 m. (c) Working face advance 100 m. (d) Working face advance 160 m.

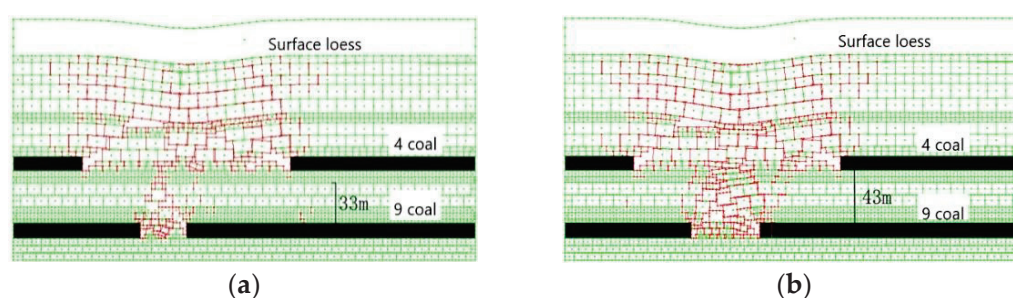


Figure 9. Spatial variation characteristics of overburden under the influence of repeated mining of coal seam No. 9. (a) Working face advance 40 m. (b) Working face advance 50 m.

In the mining of single coal seams, when the working face advances 20 m, the direct roof collapses and begins to form a water storage space. The overhanging area of the main roof is large, local cracks appear in the lower part of the main roof and fracture instability begins to appear. At this time, after the direct roof collapses, the main roof does not completely collapse under the support of coal pillars on both sides of the goaf, so the goaf is not fully filled, and there is a large space on its left and right. When the aquifer is connected with the goaf, this area becomes the main water accumulation space. When it is excavated to 40 m, initial breaking of main roof occurs. Therefore, it can be inferred that the initial pressure step distance is between 35 and 40 m. In terms of fracture form, due to the large lower space after coal caving, the whole stratification under the main roof first cuts off, the cracks continue to develop in the main roof top space and the water accumulation space keeps increasing. When the working face reaches 160 m, spatial fractures in overburdened rock are basically fully developed, and overburdened rock fractures have spread to the surface. At this time, the middle zone of the goaf fully sinks, and the rock in the caving fracture zone is compressed due to the vertical compressive stress. The water source cannot gather in this area, mainly in the incompletely compacted cracks.

In lower coal seam No. 9, repeated mining and the advancement of the working face to 40 m have caused the overburden fracture to spread to the 4 coal floor; when the working face advances to 50 m, the overburden fracture breaks completely through the 4 coal floor, which occurs as a result of repeated 9 coal mining and has carried on the secondary development of overburdening, which increases in a crack in the channel, increasing the water space, making the risk of water inrush bigger.

In the process of the working face advancing, with the continuous instability and failure of overburdening, the space, size and shape of the fracture network changes accordingly. The general evolution process of the fracture network space is compaction and generation at the same time.

5. “Solid-Liquid” Coupling Water Inrush Mechanism in Coal Seam Mining

As mentioned above, water inrush in coal seam mining is the interaction of water inrush source, dynamic source, channel source and spatial source. The sudden water source and dynamic source are the original conditions that existed before the mining of the coal seam, that is, there is a strong aquifer. The mining of the coal seam leads to the change of the structure of the upper overburden, and then the channel source and spatial source are generated. Overburden fracture is the channel of water seepage, migration and water storage space. The seepage disaster process of rock strata is the result of the coupling effect of a water-bearing body and consolidated rock mass [17]. According to the deformation and instability process of roof strata, the elastic theory of pore-fissure is referred to [18–21], and the water inrush process of coal seam mining is divided into three “solid-liquid” coupling action stages:

5.1. Original Gap Seepage Stage

Before the coal seam is mined, there is an original fissure in the overburden mass. At this time, the water channel and water accumulation space are insufficient, and the aquifer carries out low-speed seepage under the action of water flow. The solid-liquid coupling equation is as follows:

$$\begin{cases} Gu_{i,jj} + (\lambda + G)u_{k,ki} + \sum_{m=1}^2 \alpha_m p_{m,i} = 0 \\ -\frac{\mu}{k} p_{m,kk} = \alpha_m \varepsilon_{kk} - c^* p_m \pm \Gamma(\Delta p) \end{cases} \quad (7)$$

where $m:m = 1$ represents rock foundation, $m = 2$ represents rock gap; G is the shear modulus, Pa; λ is the lame constant; $u_{i,jj}$, $u_{k,ki}$ is displacement, m; α_m is Biot constant; $p_{m,i}$ is the fluid pressure, Pa; k is the equivalent permeability value or the average permeability of the overall system, m^2 ; μ is the dynamic viscosity of fluid, m^2/s ; c^* is lumped compressibility; Γ is the fluid exchange rate caused by differential pressure ΔP .

5.2. Initial Water Discharge Stage of Mining-Induced Fracture

During coal seam mining, under the influence of mining, the cracks of overburden rock mass expand and gradually form connectivity; under the action of the water pressure of the aquifer, the water from an aquifer can move down into the goaf through the cracks in the overburden, which is manifested as seepage in the roof of the working face. The solid-liquid coupling equation satisfied by the seepage of mining fractured rock mass in this process is:

$$\begin{cases} Gu_{i,jj} + (\lambda + G)u_{k,ki} + \sum_{m=1}^2 \alpha_m p_{m,i} = 0 \\ -\frac{k_m}{\mu} p_{m,kk} = \alpha_m \varepsilon_{kk} - c_m^* p_m \pm \Gamma(\Delta p) \end{cases} \quad (8)$$

where k_m is permeability of m phase, m^2 .

5.3. Water Inrush Disaster Stage of Fractured Rock Mass

With the advance of coal seam mining, the overburden fractures will further expand until the rock mass structure is damaged and collapses. At this time, the aquifer water has a sufficient migration channel and water storage space, and breaks into the goaf in a large area in the form of water inrush at the working face. The solid-liquid coupling equation satisfied by the seepage of broken rock mass in this process is:

$$\begin{cases} Gu_{i,jj} + (\lambda + G)u_{k,ki} + \sum_{m=1}^3 \alpha_m p_{m,i} = 0 \\ -\frac{k_{13}}{\mu} p_{1,kk} = \alpha_1 \varepsilon_{kk} - c_1^* p_1 + \Gamma_{12}(p_2 - p_1) + \Gamma_{13}(p_3 - p_1) \\ -\frac{k_{13}}{\mu} p_{2,kk} = \alpha_2 \varepsilon_{kk} - c_2^* p_2 + \Gamma_{21}(p_1 - p_2) + \Gamma_{23}(p_2 - p_3) \\ -\frac{k_3}{\mu} p_{3,kk} = \alpha_3 \varepsilon_{kk} - c_3^* p_3 + \Gamma_{31}(p_1 - p_3) + \Gamma_{32}(p_2 - p_3) \end{cases} \quad (9)$$

where $m = 1, 2$ and 3 represent pores, fissures and fractures, respectively; k_1 , k_2 and k_3 are pores, fractures and fracture permeability, m^2 ; k_{13} is the average permeability of rock foundation and fracture, m^2 ; Γ_{ij} is the fluid exchange rate between phase I and phase J, and it is assumed that there is interstitial flow caused by internal pressure difference between the two phases; others conform to the same meaning.

6. Prevention and Control Measures of Water Inrush in Mining Face

6.1. Forecast of Water Inrush and Inflow in Working Face

The accurate prediction of water inflow plays an important role in preventing and controlling the occurrence and control of water disasters and can guide mines to formulate prevention and control measures. According to the characteristics of water inrush, the inflow formula of a confined aquifer with a constant depth is used to predict the mine

water inflow [22–24], and the stable value of inflow and its variation characteristics with time are predicted.

6.1.1. Mathematical Model

In the early stage of coal seam mining, because the drainage time was not long enough, the falling funnel kept expanding and did not reach the stable flow state. However, the water level has been reduced to the roof of the coal seam, so the mathematical model of the water flow movement should be a definite solution of the variable flow with constant depth. Its mathematical model is

$$\begin{cases} \frac{1}{r} \frac{\partial}{\partial r} \left(r \frac{\partial s}{\partial r} \right) = \frac{S}{T} \frac{\partial s}{\partial t} & t > 0, 0 < r < \infty \\ s(r, 0) = 0 & 0 < r < \infty \\ s(\infty, 0) = 0 & t > 0 \\ s(0, t) = s_w & t > 0 \end{cases} \quad (10)$$

This mathematical model is obtained by Laplace transform and Darcy's law, and is:

$$Q = 2\pi T s_w G(\lambda) \quad (11)$$

where s_w is the drawdown in the well; $\lambda = \frac{Tt}{r_w^2 S}$ is dimensionless time; $G(\lambda)$ is the flow function of constant drawdown well flow in confined aquifer without overflow recharge.

6.1.2. Forecast of Water Inflow

In order to obtain the process $Q(t)$ of water inflow with time, different time points $t_1, t_2, \dots$ and t_n take in (6.17); the corresponding values at different time points can be obtained as $G(\lambda_1), G(\lambda_2), \dots$ and $G(\lambda_n)$ and $Q_1, Q_2, \dots$ and Q_n . The value of $G(\lambda)$ is shown in Table 2.

Table 2. $G(\lambda)$ values.

λ N	$N \times 10^{-4}$	$N \times 10^{-3}$	$N \times 10^{-2}$	$N \times 10^{-1}$	$N \times 10^0$	$N \times 10^1$	$N \times 10^2$	$N \times 10^3$
1	59.6	18.34	6.13	2.249	0.985	0.534	0.346	0.251
2	40.4	13.11	4.47	1.716	0.803	0.461	0.311	0.232
3	33.1	10.79	3.74	1.477	0.719	0.427	0.294	0.222
4	28.7	9.41	3.3	1.333	0.667	0.405	0.283	0.215
5	25.7	8.47	3	1.234	0.63	0.389	0.274	0.21
6	23.5	7.77	2.78	1.16	0.602	0.377	0.268	0.206
7	21.8	7.23	2.6	1.103	0.58	0.367	0.263	0.203
8	20.4	6.79	2.46	1.057	0.562	0.359	0.258	0.2
9	19.3	6.43	2.35	1.018	0.547	0.352	0.254	0.198
10	18.3	6.13	2.25	0.985	0.534	0.346	0.251	0.196
N	$N \times 10^4$	$N \times 10^5$	$N \times 10^6$	$N \times 10^7$	$N \times 10^8$	$N \times 10^9$	$N \times 10^{10}$	$N \times 10^{11}$
1	0.1964	0.1608	0.136	0.1177	0.1037	0.0927	0.0838	0.0764
2	0.1841	0.1542	0.1299	0.1131	0.1002	0.0899	0.0814	0.0744
3	0.1777	0.1524	0.1299	0.1131	0.1002	0.0899	0.0814	0.0744
4	0.1733	0.1449	0.1244	0.1089	0.0968	0.0872	0.0792	0.0726
5	0.1701	0.1426	0.1227	0.1076	0.0958	0.0864	0.0785	0.072
6	0.1675	0.1441	0.1213	0.1066	0.095	0.0857	0.0779	0.0716
7	0.1654	0.1393	0.1202	0.1057	0.0943	0.0851	0.0774	0.0712
8	0.1636	0.138	0.1192	0.10105	0.0937	0.0846	0.077	0.0709
9	0.1621	0.1369	0.1184	0.1043	0.0932	0.0842	0.0767	0.0706
10	0.1608	0.136	0.1177	0.1037	0.0927	0.0838	0.0764	0.0704

Water Inflow of Coal Seam No. 4

The calculation parameters of water inflow of coal seam No. 4 are shown in Table 3, and the value of time t is shown in Table 4.

Table 3. Coal No. 4 water inflow time value table.

	t_1	t_2	t_3	t_4	t_5	t_6	t_7	t_8	t_9	t_{10}
Time (d)	10	30	50	100	150	200	300	400	500	600

Table 4. Calculation of water inflow of unsteady flow coal No. 4.

Time t (d)	λ	$G (\lambda)$	Inflow $Q (m^3/h)$
10	0.00016	48.08	194.8
30	0.00049	25.40	102.9
50	0.00082	20.18	81.8
100	0.00164	15.20	61.6
150	0.00246	11.95	48.4
200	0.00328	10.43	42.3
300	0.00492	7.84	31.8
400	0.00656	7.50	30.4
500	0.00821	6.86	27.8

Bring the parameter values in Table 3 into formula 11 and take values according to Table 2 to obtain (Table 4 and Figure 10).

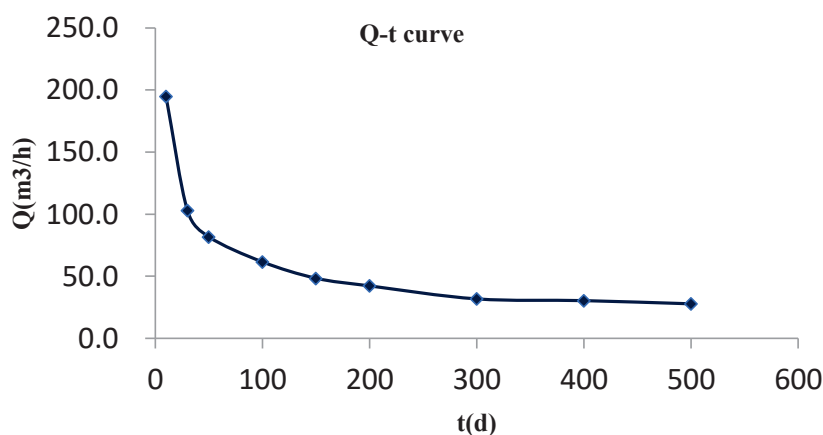


Figure 10. Coal No. 4 inflow process line.

It can be seen from Figure 10 that after about 300 days of drainage, the water inflow of coal 4 is basically stable, and the flow is $27.8 m^3/h$.

Water Inflow of Coal Seam No. 9

The calculation parameters of the water inflow of coal seam No. 9 are shown in Table 2, and the value of time t is shown in Table 5.

Table 5. Coal No. 9 water inflow time value table.

	t_1	t_2	t_3	t_4	t_5	t_6	t_7	t_8	t_9	t_{10}
Time (d)	10	30	50	100	200	300	400	500	600	700

Bring the parameter values in Table 5 into formula 11 and take values according to Table 2 to obtain (Table 6 and Figure 11).

Table 6. Calculation of water inflow of unsteady flow coal No. 9.

Time t (d)	λ	$G(\lambda)$	Inflow Q (m^3/h)
10	0.00041	28.40	775.3
30	0.00122	17.29	472.0
50	0.00204	13.11	357.9
100	0.00408	9.41	256.9
200	0.00817	6.72	183.5
300	0.01225	5.80	158.3
400	0.01633	5.13	140.0
500	0.02041	4.40	120.1
600	0.02450	4.11	112.2
700	0.02858	3.85	105.1

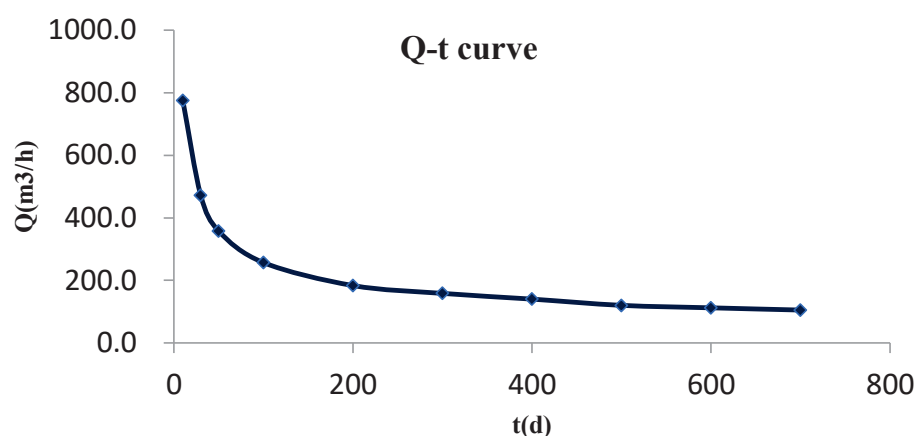


Figure 11. Coal No. 9 inflow process line.

It can be seen from Figure 11 that after about 500 days of drainage, the water inflow of coal 9 is basically stable, and the flow is $105.10 m^3/h$.

6.2. Water Inrush Prevention and Control Measures

- (1) The water-rich area of the working face should be appropriately accelerated. On the one hand, the processing speed can shorten the overburden dynamic deformation process time; the better the overburden compaction effect is, the more the compression of “spatial source”. On the other hand, the accelerating speed can shorten the development time of the overlying rock fissures and can quickly close, preventing the development of a “channel source”.
- (2) Drainage of water-rich aquifers in the upper working face before mining, as well as drilling and drainage in advance and drainage and pressure reduction to reduce the “water inrush source” and eliminate the “dynamic source”.
- (3) Thickness limited mining or local filling in the water-rich area of the working face. This method can restrain the development of longitudinal fractures in working face.

7. Conclusions

- (1) Water inrush from coal seam mining consists of “four source elements”, which are water inrush source, dynamic source, channel source and spatial source. The water inrush source and dynamic source are the original conditions existing before coal seam mining; the mining of the coal seam changes the upper overburden structure and then produces the channel source and spatial source; when the “four source elements” constitute a complete relationship, water inrush will occur in the working face.
- (2) Using the key strata theory, this paper carries out a mechanical analysis of the longitudinal expansion of overburden fractures during mining of short distance coal seam groups and grasps the development and evolution characteristics of a “channel source”. Through mechanical model analysis, the two key strata that play a key role in controlling the whole overburden in the mining of coal seam groups in the study area have the form of shear failure (sliding instability) during mining.
- (3) The UDEC numerical simulation method is used to simulate the spatial evolution characteristics of mining-induced overburden fractures. The results show that in the process of the working face advancing, with the continuous instability and failure of overburden, the spatial size and shape of fracture network are also changing, and the general evolution process is as follows: generation-compaction and generation-synchronous action.
- (4) The seepage catastrophe process of rock stratum is the result of the intercoupling of a water bearing body and a consolidated rock mass. According to the deformation and instability process of roof strata and the theory of pore-fracture elasticity, the water inrush process of coal seam mining is divided into three “solid-liquid” coupling action stages: the original gap seepage stage, the initial water discharge stage of a mining-induced fracture and the water inrush disaster stage of a fractured rock mass.

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Article

Study on the Pressure Relief Mechanism and Engineering Application of Segmented Enlarged-Diameter Boreholes

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Abstract: Ensuring the stability of surrounding rock while conducting pressure relief is key to roadway stability control and preventing rock burst. In this paper, segmented enlarged-diameter borehole destressing technology is proposed. The stress distribution around the borehole is analysed based on the theory of elasto-plastic mechanics, and the mechanism of pressure relief for segmented enlarged-diameter boreholes is studied. Secondly, The effects of the diameter of enlarged-diameter section, length of the enlarged-diameter section, and borehole space on pressure relief and roadway deformation were investigated. Finally, the optimal drilling parameters for Tangkou coal mine 6307 working face are analysed. The key parameters of segmented enlarged-diameter borehole pressure relief technology for the 6307 working face were determined and applied in field practice. Field monitoring results showed that the accumulated energy can be effectively reduced using segmented enlarged-diameter pressure relief boreholes, effectively controlling roadway deformation and providing reference for rock burst prevention and roadway stability control.

Keywords: rock burst; pressure relief; deformation; segmented enlarged-diameter borehole

1. Introduction

With the gradual depletion of shallow resources, deep mining in eastern China as well as parts of central and western China has become commonplace (the average mining depth has reached 800–1000 m) [1–6]. The occurrence frequency and damage degree of coal and rock dynamic disasters increase with increasing mining depth (e.g., rock burst, coal and gas outburst, etc.). Rock burst is a typical coal mine dynamic disaster, which violently releases deformation energy, throwing out coal and rock, causing hydraulic support damage, rib spalling, and roof collapse [7]. In recent years, several rock burst accidents have successively occurred in China [8]. Rock burst has become one of the primary disaster types seriously restricting the safety production of mines.

In order to prevent rock burst, numerous researchers have made important contributions to rock burst monitoring, early warning, and control [4,9–14]. Mohammadali [15] estimate the rockburst potentials in a real case study mine combined with conventional and numerical methods. Previous studies have showed that stress is the key factor of rock burst occurrence. Therefore, before the working face is laid out, the first step is to adopt a reasonable development layout and mining method to reduce the influence of geological formations on the working face [16,17]. Victor [18] modernised the pillarless system by introducing a third roadway as a pressure relief roadway to protect the working face roadway from the support pressure during mining of the adjacent slab. For the high stress area that has been formed, pressure relief should be conducted on any high stress areas, including coal seam blasting pressure relief, high-pressure water injection, and large diameter borehole pressure relief [19]. In order to reduce the degree of stress concentration

in the coal body, Dou et al. [20] proposed the theory of strength weakening using blasting pressure relief. Piotr [21] presents Polish experience in rockburst hazard assessment with the use of geomechanical method, as well as some solutions and examples of such analyses. Konicek [22] presented a case study of destress blasting and evaluated the effectiveness of rockburst prevention. Saeed [23] uses a fully coupled 3D finite element model of hydraulic fracturing in permeable rocks to investigate the interaction between multiple simultaneous and continuous hydraulic fractures. However, for mining roadways, improper blasting may also induce dynamic disasters such as rock burst and achieving the ideal pressure relief using coal seam water injection is difficult. Large diameter borehole pressure relief is an effective method to reduce the stress concentration for high stress areas. At present, this method has been widely used in coal mines and has become one of the primary methods of controlling rock burst disasters in coal mines.

Large diameter borehole pressure relief can transfer high stresses from the roadway side to the deep coal body. It has a relatively simple construction process, low cost, easy operation, and little impact on production [24]. Jia et al. [25] studied the pressure relief mechanism of large diameter boreholes using laboratory tests and numerical simulation. Zhang et al. [26] studied the variation of uniaxial compressive strength and energy dissipation index of coal after drilling and analyzed the characteristics of crack growth in coal. The above research is of great significance to understanding the mechanism behind large diameter borehole pressure relief.

The conventional large diameter borehole pressure relief method is plagued by insufficient pressure relief (rock burst still occurs after pressure relief) and excessive pressure relief (the strength of the original anchor layer will be damaged after pressure relief). Therefore, researchers put forward an integrated method of pressure relief and surrounding rock reinforcement, named ‘strong pressure relief and strong support. Tan et al. [27] examined the “relief and supporting” cooperative control mechanism of deep coal roadway and developed a “relief and supporting” cooperative control technology. Ensuring the stability of shallow surrounding rock and the associated support system while conducting proper pressure relief is key to safely controlling rock burst. For this reason, some scholars proposed a segmented enlarged-diameter borehole destressing technology and analyzed the effect of the diameter of the enlarged-diameter section on pressure relief effect and support system [28]. However, the above study only examined factors affecting the pressure relief due to the enlarged-diameter section using numerical simulation. The pressure relief mechanism of the segmented enlarged-diameter boreholes should be further analyzed.

This study first calculated and analyzed stress distribution characteristics around the segmented enlarged-diameter borehole as well as the position of enlarged-diameter section. Then, the effect of parameters, such as the diameter of enlarged-diameter section, length of enlarged-diameter section, and borehole space, on pressure relief and roadway deformation were further analyzed. Based on theoretical analysis and numerical simulation results, the key pressure relief parameters of the segmented enlarged-diameter borehole were determined and applied in the mining roadway of 6307 working face of the Tangkou Coal mine. The results provide reference for rock burst prevention and control as well as roadway stability control.

2. Destressing Mechanism of Segmented Enlarged-Diameter Borehole

2.1. Stress Distribution Characteristics around the Segmented Enlarged-Diameter Borehole

Numerous researchers have examined the stress distribution around circular holes. Kirsch [29] deduced the two-dimensional stress distribution solution around a circular hole in an elastic plate. Jaeger et al. [30] further developed the Kirsch equation. For a long time, the modified Fenner formula [31] or Castenet [32] solution was used to determine radius of the plastic zone around a circular hole, and plastic zone theory was widely used in surrounding rock with softening and dilatancy.

When solving the boundary equation for the pressure relief zone, the coal around the borehole is assumed to be in an elastic state, and the yield of the coal under the

stress condition is determined according to the plastic condition. The basic assumptions are listed as follows: (1) The coal body where the pressure relief borehole located is continuous, homogenous, isotropic, and linear elastic; (2) The axial direction of the pressure relief borehole is infinite; (3) The coal properties are consistent in the axial direction of the pressure relief borehole.

The segmented enlarged-diameter borehole is divided into an unenlarged-diameter section and enlarged-diameter section (Figure 1). The enlarged-diameter section is located in the elastic zone, which was outside the anchorage layer of the surrounding rock. The axial length of the pressure relief borehole is much greater than its radius, so the borehole can be analyzed as a plane strain problem. The vertical and horizontal stresses subjected to the pressure relief borehole are simplified as uniformly distributed stresses (Figure 1).

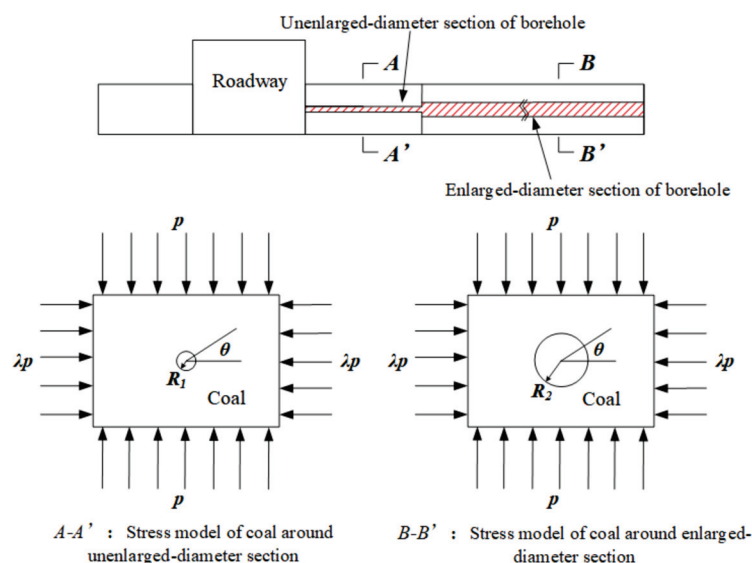


Figure 1. Stress model of coal around boreholes.

According to the superposition of elasticity, the load of an unenlarged-diameter section and enlarged-diameter section of borehole can be divided into two parts: the uniformly distributed compressive stress $(1 + \lambda)p/2$ (Figure 2a); the uniformly distributed tension stress $(1 - \lambda)p/2$ in the horizontal direction and the uniformly distributed compressive stress $(1 - \lambda)p/2$ in the vertical direction (Figure 2b).

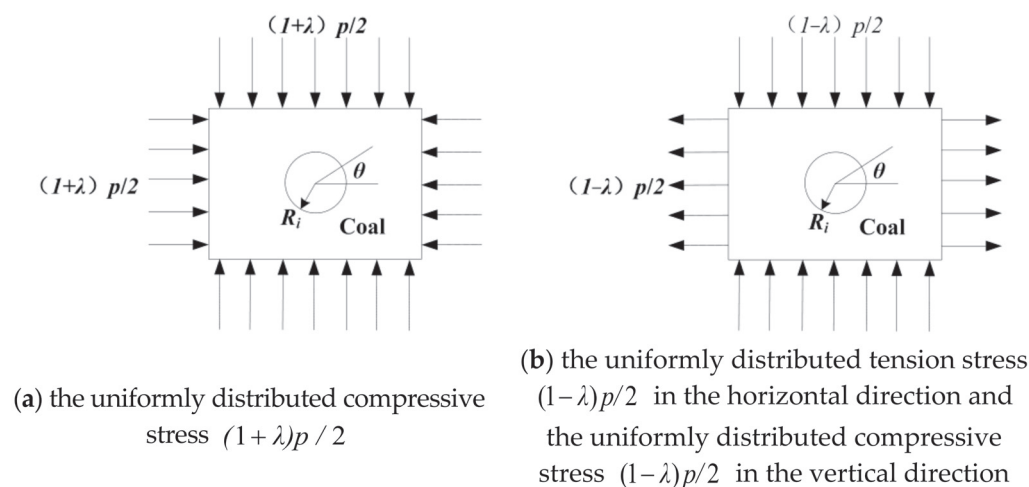


Figure 2. Borehole stress decomposition based on superposition.

According to the classical solution of elastic mechanics, the expression for the stress component of coal around the borehole in polar coordinates is:

$$\sigma_r = -\frac{(\lambda+1)p}{2} \left(1 - \frac{R_i^2}{r^2}\right), \quad \sigma_\theta = -\frac{(\lambda+1)p}{2} \left(1 + \frac{R_i^2}{r^2}\right), \quad \tau_{r\theta} = \tau_{\theta r} = 0 \quad (1)$$

where: p —Vertical stress on the coal body, MPa;

θ —The polar angle of the coal body at any point around the drill hole; σ_r —Radial stress at a point of coal around the borehole, MPa;

σ_θ —Tangential stress at a point of coal around the borehole, MPa;

$\tau_{r\theta}, \tau_{\theta r}$ —Shear stress at a point of coal around the borehole, MPa;

R_i —Radius (radius of unenlarged-diameter section is R_1 and radius of enlarged-diameter section is R_2), m;

λ —Side pressure coefficient;

r —The distance from a point of coal to the borehole center.

For the stress situation shown in Figure 2b, according to the classical solution of elastic mechanics, the expression for the stress component of coal around the borehole is:

$$\begin{cases} \sigma_r = \frac{(1-\lambda)p}{2} \left(1 - \frac{R_i^2}{r^2}\right) \left(1 - 3\frac{R_i^2}{r^2}\right) \cos 2\theta \\ \sigma_\theta = -\frac{(1-\lambda)p}{2} \left(1 + 3\frac{R_i^4}{r^4}\right) \cos 2\theta \\ \tau_{r\theta} = -\frac{(1-\lambda)p}{2} \left(1 - \frac{R_i^2}{r^2}\right) \left(1 + 3\frac{R_i^2}{r^2}\right) \cos 2\theta \end{cases} \quad (2)$$

The stress components in Figure 2 are superimposed to obtain the polar coordinate stress components around the borehole under the load in Figure 1:

$$\begin{cases} \sigma_r = -\frac{(\lambda+1)p}{2} \left(1 - \frac{R_i^2}{r^2}\right) + \frac{(1-\lambda)p}{2} \left(1 - \frac{R_i^2}{r^2}\right) \left(1 - 3\frac{R_i^2}{r^2}\right) \cos 2\theta \\ \sigma_\theta = -\frac{(\lambda+1)p}{2} \left(1 + \frac{R_i^2}{r^2}\right) - \frac{(1-\lambda)p}{2} \left(1 + 3\frac{R_i^4}{r^4}\right) \cos 2\theta \\ \tau_{r\theta} = -\frac{(1-\lambda)p}{2} \left(1 - \frac{R_i^2}{r^2}\right) \left(1 + 3\frac{R_i^2}{r^2}\right) \cos 2\theta \end{cases} \quad (3)$$

Due to stress concentration at the edge of the borehole, the coal around the borehole forms a broken zone, a plastic zone, and an elastic zone (Figure 3). The broken zone and the plastic zone around the borehole are collectively called the pressure relief zone, and the boundary line of the plastic zone is the boundary line of the borehole pressure relief zone.

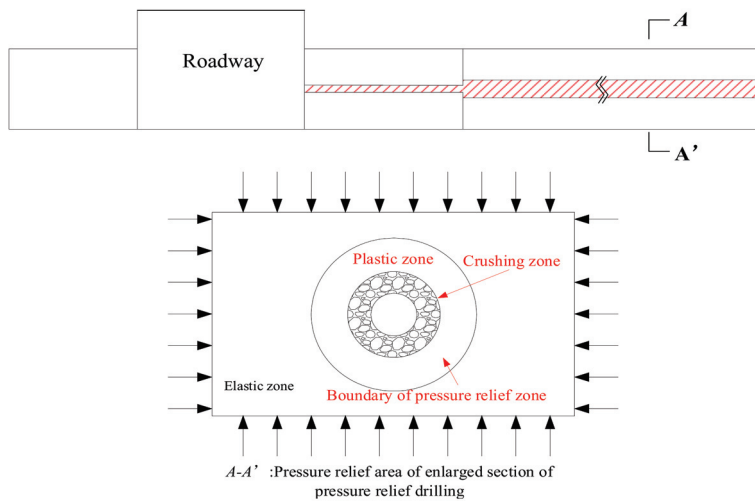


Figure 3. Borehole pressure relief zone diagram.

According to Equation (3), the expression for the principal stress at a point of coal around the borehole is:

$$\begin{cases} \sigma_1 = \frac{\sigma_r + \sigma_\theta}{2} + \frac{1}{2} \sqrt{(\sigma_r - \sigma_\theta)^2 + 4\tau_{r\theta}^2} \\ \sigma_3 = \frac{\sigma_r + \sigma_\theta}{2} - \frac{1}{2} \sqrt{(\sigma_r - \sigma_\theta)^2 + 4\tau_{r\theta}^2} \end{cases} \quad (4)$$

where: σ_1, σ_3 —the maximum and minimum principal stresses, MPa.

According to the Mohr-Coulomb criterion, the yield conditions at a point of coal around the borehole is:

$$\frac{1}{2} \sqrt{(\sigma_r - \sigma_\theta)^2 + 4\tau_{r\theta}^2} = c \cos \varphi - \frac{1}{2} (\sigma_r + \sigma_\theta) \sin \varphi \quad (5)$$

where: c —Coal cohesion, MPa;

φ —Internal friction angle.

Substituting Equation (3) into Equation (5) gives the boundary line equation of the coal plastic zone around the borehole:

$$b + h \frac{r^2}{R^2} + d \frac{r^4}{R^4} + e \frac{r^6}{R^6} + f \frac{r^8}{R^8} = 0 \quad (6)$$

where: $b = 9(1 - \gamma)^2$;

$h = 6(1 - \gamma)^2 \cos 2\varphi - 12(1 - \gamma)^2$;

$d = (1 + \gamma)^2 + 10(1 - \gamma)^2 \cos^2 2\varphi - 2(1 - \gamma)^2 \sin^2 2\varphi - 4(1 - \gamma^2) \cos 2\varphi - 4(1 - \gamma)^2 \cos^2 2\varphi \sin^2 \varphi$;

$e = 4(1 - \gamma)^2 (\sin^2 2\varphi - \cos^2 2\varphi) + 2(1 - \gamma^2) \cos 2\varphi - 4\frac{c}{p}(1 - \gamma) \sin 2\varphi \cos 2\varphi - 4(1 - \gamma^2) \cos 2\varphi \sin^2 \varphi$;

$f = (1 - \gamma)^2 - 4\frac{c^2}{p^2} \cos^2 \varphi - (1 + \gamma)^2 \sin^2 \varphi - 2\frac{c}{p}(1 + \gamma) \sin 2\varphi$;

r —Radius of pressure relief zone.

According to Equation (6), the distribution range of the pressure relief zone around the borehole is related to the vertical stress P , lateral pressure coefficient γ , borehole radius R_i , cohesion c of the coal, and internal friction angle φ .

2.2. Determination of Segmented Enlarged-Diameter Position of Large Diameter Pressure Relief Boreholes

Determining the segmented enlarged-diameter position of the pressure relief borehole is of great significance. Surrounding rock under bolt support is divided into elastic zone, non-anchoring plastic zone, anchoring plastic zone, and anchoring broken zone (Figure 4). In the figure, the roadway radius is r_a , the radius of crushing zone is R_b , the radius of plastic zone is R_p , the effective anchoring length of bolt is l_0 , and the annular distance of bolt is f_1 . There is assumed to be no slip between the bolt and the surrounding rock. The stress of the original rock is P_0 , and the uniform support resistance provided by the roadway support is P_i . Based on the Mohr-Coulomb criterion, the radius of the plastic zone was calculated to provide a basis for the calculation of the segmented enlarged-diameter position [33,34].

In the plastic zone, when the volume force is not considered, the stress component at a point should satisfy the polar coordinate balance equation:

$$\frac{d\sigma_r^p}{dr_p} + \frac{\sigma_r^p - \sigma_\theta^p}{r_p} = 0 \quad (7)$$

where: σ_θ^p —Tangential stress in plastic zone of surrounding rock, MPa;

σ_r^p —Radial stress in plastic zone of surrounding rock, MPa;

r_p —Radius of a plastic zone somewhere in the surrounding rock, m.

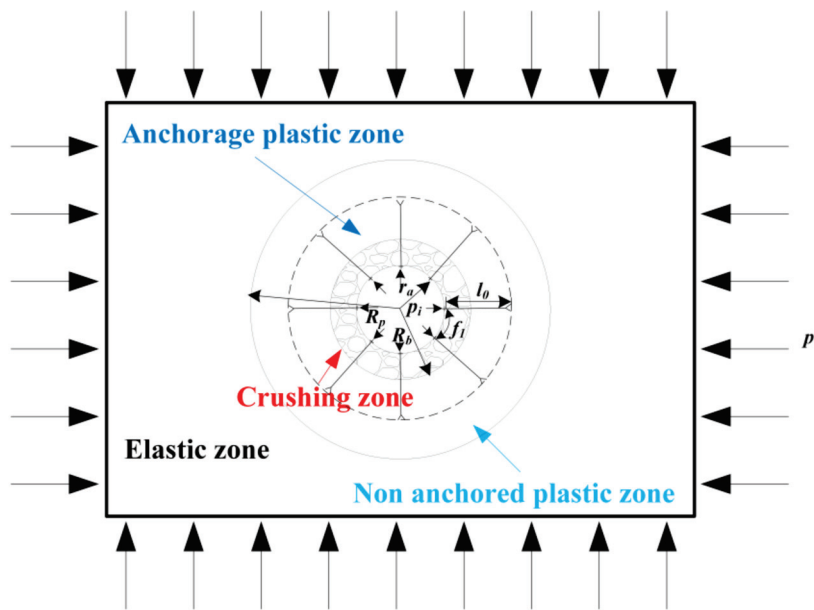


Figure 4. Diagram of plastic zone in surrounding rock under anchorage.

In addition to satisfying the equilibrium equation in the plastic zone, the internal stress of surrounding rock should meet the conditions of the plastic yield criterion, namely $\sigma_\theta^p = K_\phi \sigma_r^p + \sigma_c$, which can be substituted into Equation (7):

$$\frac{2 \sin \phi}{1 - \sin \phi} \ln r + C = \ln(\sigma_r^p + c \cot \phi) \quad (8)$$

Based on the boundary conditions, when the support is applied, the stress expression of plastic zone is:

$$\begin{cases} \sigma_r^p = (p_i + c \cot \phi)(r/r_a)^{\frac{2 \sin \phi}{1 - \sin \phi}} - c \cot \phi \\ \sigma_\theta^p = (p_i + c \cot \phi)^{\frac{1 + \sin \phi}{1 - \sin \phi}} (r/r_a)^{\frac{2 \sin \phi}{1 - \sin \phi}} - c \cot \phi \end{cases} \quad (9)$$

where: p_i —The roadway support body provides uniform support resistance, MPa.

The tangential stress σ_θ^p and radial stress σ_r^p both increase with increasing radius r within a certain range. Tangential stress σ_θ^p decreases with increasing radius r . The peak point is the stress junction of elastic-plastic stress, and the stress at this point satisfies both plastic stress and elastic stress conditions. When $r \rightarrow \infty$, both radial stress σ_r^p and tangential stress σ_θ^p tend to be stable and close to the original rock stress P_0 .

For the elastic zone, the radius of the plastic zone is R_p . When $r = R_p$:

$$\sigma_{R_p} = \sigma_r^p = \sigma_r^e, \sigma_\theta^p = \sigma_\theta^e \quad (10)$$

where: σ_{R_p} —Radial stress at elastoplastic interface of surrounding rock, MPa;

σ_r^e —Radial stress in elastic zone of surrounding rock, MPa;

σ_θ^e —Tangential stress in elastic zone of surrounding rock, MPa.

For the elastic region, $r \geq R_p$:

$$\begin{cases} \sigma_r^e = p_0 \left(1 - \frac{R_p}{r_e}\right) + \sigma_{R_p} \frac{R_p^2}{r_e^2} \\ \sigma_\theta^e = p_0 \left(1 + \frac{R_p}{r_e}\right) - \sigma_{R_p} \frac{R_p^2}{r_e^2} \end{cases} \quad (11)$$

The stress at the boundary between elastic and plastic zones ($r = R_p$) is:

$$\begin{cases} \sigma_r^e = p_0(1 - \sin \varphi) - c \cos \varphi = \sigma_{R_p} \\ \sigma_\theta^e = p_0(1 + \sin \varphi) + c \cos \varphi = 2p_0 - \sigma_{R_p} \end{cases} \quad (12)$$

Substituting $r = R_p$ into Equation (10) gives the relationship between the plastic zone radius R_p and p_i :

$$p_i = -c \cot \varphi + [p_0(1 - \sin \varphi) - c \cos \varphi + c \cot \varphi] \left(r_a / R_p \right)^{\frac{2 \sin \varphi}{1 - \sin \varphi}} \quad (13)$$

The above equation can also be written as:

$$R_p = r_a \left[(1 - \sin \varphi) \frac{c \cot \varphi + p_0}{c \cot \varphi + p_i} \right]^{\frac{2 \sin \varphi}{1 - \sin \varphi}} \quad (14)$$

It can be seen from Equation (14) that the range of plastic zone decreases with increasing supporting resistance P_i . In order to reduce the extent of roadway damage, the plastic zone radius (R_p) of surrounding rock under the influence of bolt support is the minimum length of the unenlarged-diameter section of borehole.

2.3. Mechanism of Segmented Enlarged-Diameter Borehole Destressing

If the pressure relief is excessive, the overall roadway stability may be affected. Therefore, a segmented enlarged-diameter borehole destressing mechanism is proposed based on conventional large-diameter pressure relief technology (Figure 5). For the unenlarged-diameter section, the small diameter of the borehole can reduce the damage to the roadway support system and shallow surrounding rock. While for the enlarged-diameter section, it can form a large broken zone around the borehole far from the anchorage area, transferring the stress concentration to the deep coal body.

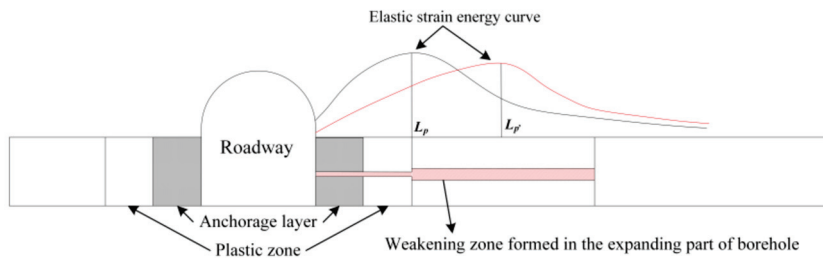


Figure 5. Pressure relief mechanism of segmented enlarged-diameter boreholes.

3. Simulation Schemes

3.1. Numerical Model and Simulation Schemes

Tangkou Coal Mine is located in Nanzhang Town, Jining City, Shandong Province, and the 6307 working face (Figure 6a) of Tangkou coal mine was analyzed in this study. The thickness of coal seam is 8.1 m–10.4 m, and the burial depth of the 6307 working face is approximately 980 m. Fully mechanized top coal caving with strike longwall retreating is used to mine the 6307 working face. The roof and floor strata of the coal seam are composed of relatively complete medium sandstone and siltstone (Figure 6b). The roadway is excavated along the coal seam floor with a section size of 5400×3800 mm, and a both anchor, net, and cable supports are used (Figure 6c).

A FLAC^{3D} numerical model was constructed based on the 6307 working face. The size of the model is $340 \text{ m} \times 200 \text{ m} \times 46 \text{ m}$ (Figure 7). Considering the burial depth of the working face, a uniform distribution load of 23 MPa was applied at the top to simulate the weight of overlying strata. Rock type and mechanical parameters were selected based on mine geology reports and laboratory test results (Table 1). The large-diameter borehole was

placed to the side of the coal side of the roadway. The mesh around the borehole is shown in Figure 8.

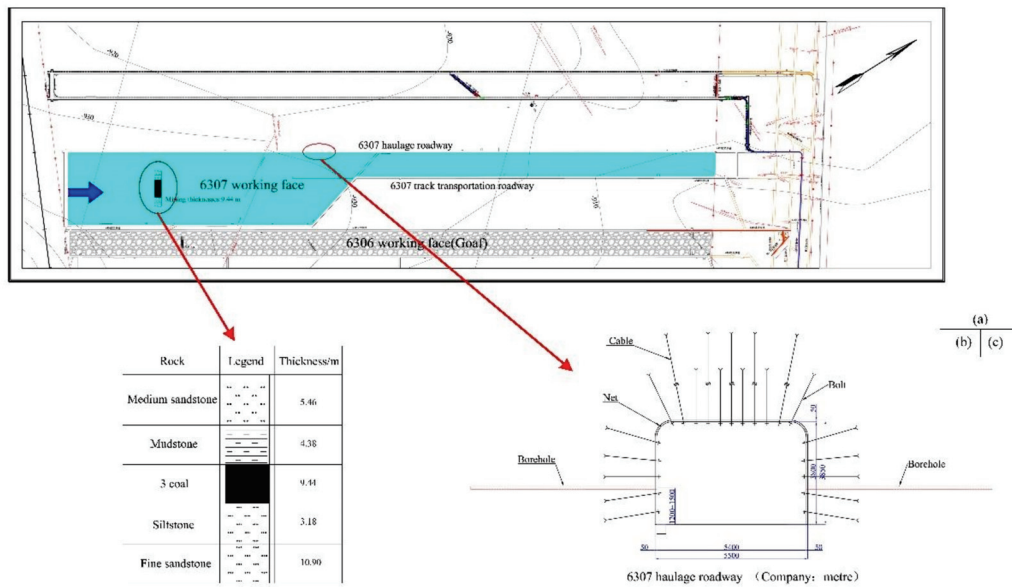


Figure 6. Diagram of the 6307 working face. ((a) 6307 working face layout plan; (b) 6307 working face top and bottom rock; (c) Diagram of roadway support at 6307 working face).

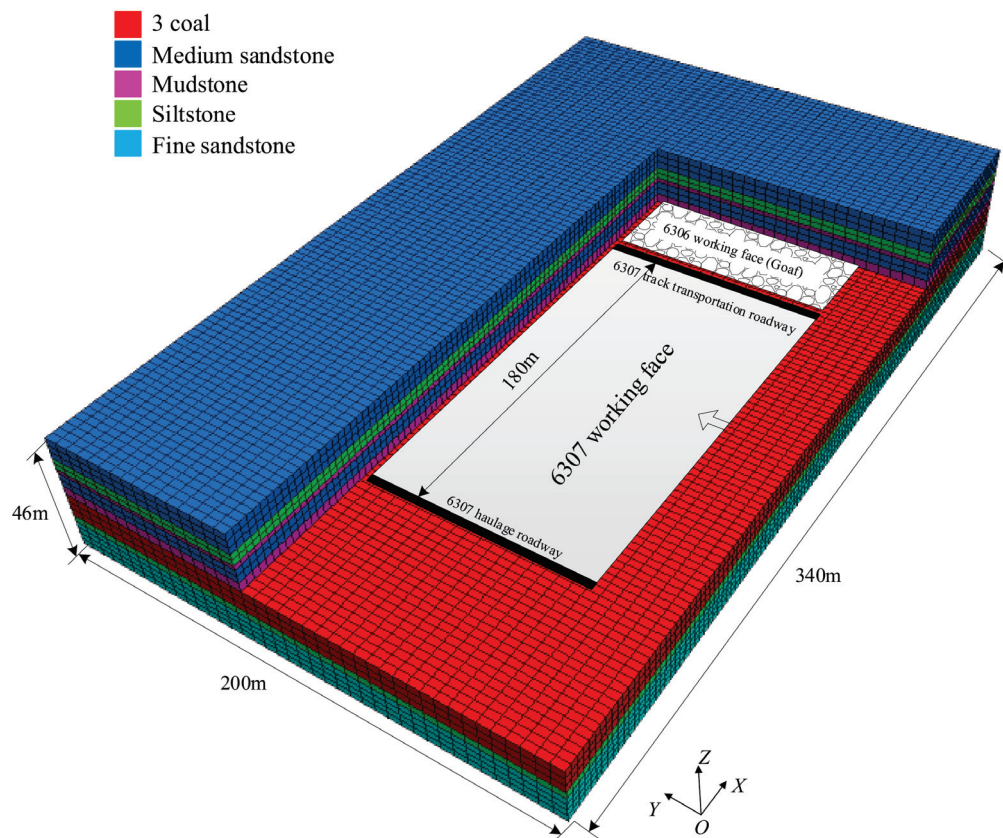
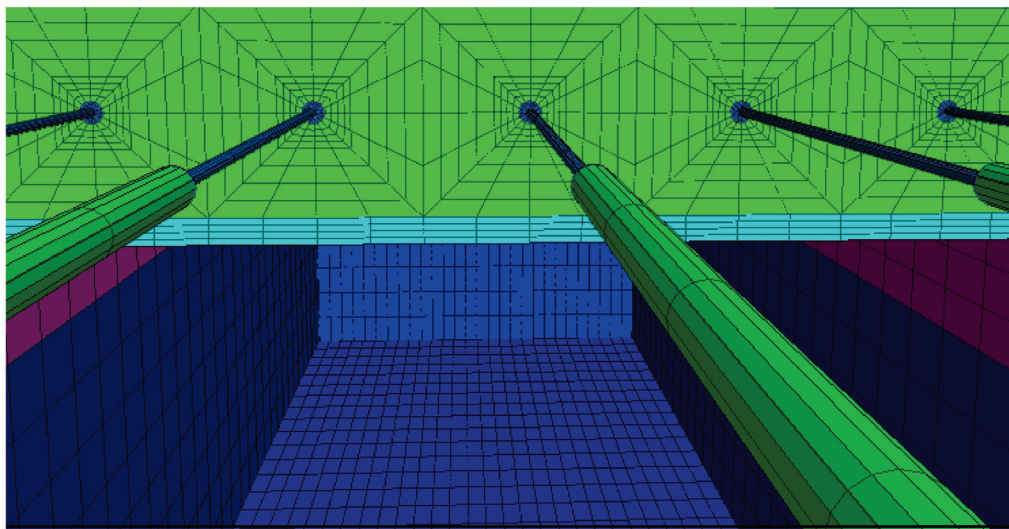


Figure 7. FLAC^{3D} numerical calculation model.

Table 1. Physical and mechanical parameters of coal and rock.

Rock Type	Thickness/m	Density kg/m ³	Bulk Modulus/GPa	Shear Modulus/GPa	Tension Strength/MPa	Cohesion /MPa	Friction Angle/(°)
Medium sandstone	4.9	2570	7.95	6.87	4.52	6.12	31
Siltstone	5.4	2700	16	10	2.3	2.4	32
Mudstone	2.4	2550	3	1.3	1.5	1.7	29
Medium sandstone	5.4	2570	7.95	6.87	4.52	6.12	31
Mudstone	4.4	2550	3	1.3	1.5	1.7	29
Coal	9.4	1400	1.5	0.7	1.1	1.2	23
Siltstone	3.2	2700	16	10	2.3	2.4	32
Fine sandstone	10.9	2620	8.5	5.6	4.7	5.52	35

**Figure 8.** Mesh generation around boreholes.

The effect of diameter of the enlarged-diameter section, length of the enlarged-diameter section, and borehole space on pressure relief effect and roadway deformation was simulated. The simulation scheme is as follows.

(1). Different diameters of enlarged-diameter section

Under the same total borehole length, length of the enlarged-diameter section, diameter of the unenlarged-diameter section, and borehole space, enlarged-diameter sections with diameters of 90 mm, 140 mm, 190 mm, and 240 mm were selected to investigate the pressure relief effect and roadway deformation (Figure 9) (Table 2).

Table 2. Simulation scheme of different diameters of enlarged-diameter section.

No.	Total Borehole Length l_1 /m	Length of the Enlarged-Diameter Section l_2 /m	Diameter of the Unenlarged-Diameter Section d_1 /mm	Borehole Space s_1 /m	Diameter of the Enlarged-Diameter Section d_2 /mm
1	20	15	90	1.6	90
2	20	15	90	1.6	140
3	20	15	90	1.6	190
4	20	15	90	1.6	240

(2). Different lengths of the enlarged-diameter section

Under the same total borehole length, diameter of the unenlarged-diameter section, diameter of the enlarged-diameter section, and borehole space, enlarged-diameter sections with lengths of 11 m, 13 m, 15 m, 17 m, and 19 m were selected to investigate the pressure relief effect and roadway deformation (Figure 10) (Table 3).

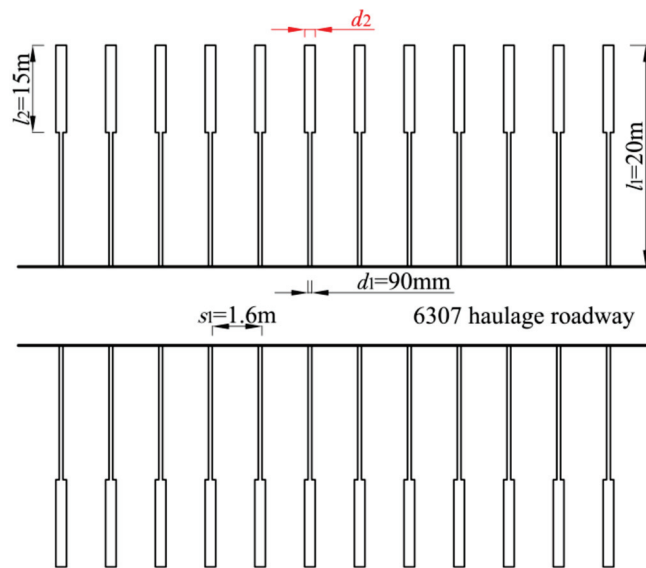


Figure 9. Diagram of the simulation scheme for enlarged-diameter sections with varying diameters.

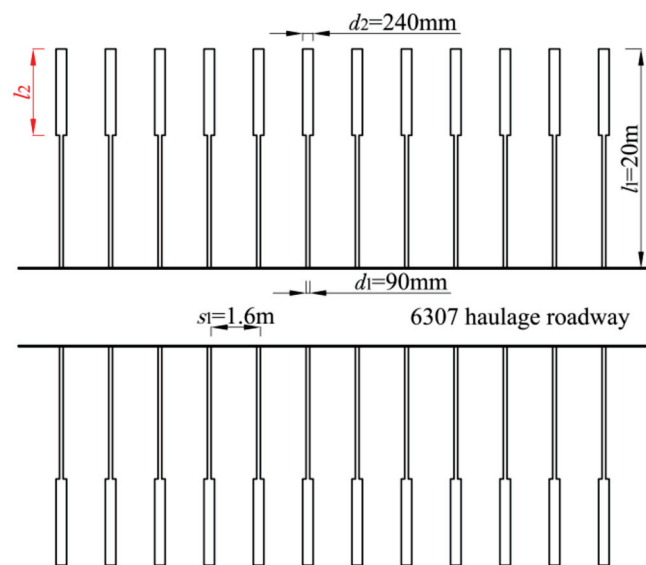


Figure 10. Diagram of the simulation scheme for enlarged-diameter sections with varying lengths.

Table 3. Simulation scheme for enlarged-diameter sections with varying lengths.

No.	Total Borehole Length l_1/m	Diameter of the Unenlarged-Diameter Section d_1/mm	Diameter of the Enlarged-Diameter Section d_2/mm	Borehole Space s_1/m	Length of the Enlarged-Diameter Section l_2/m
1	20	90	240	1.6	11
2	20	90	240	1.6	13
3	20	90	240	1.6	15
4	20	90	240	1.6	17
5	20	90	240	1.6	19

(3). Different borehole spaces

Under the same total borehole length, the length of the enlarged-diameter section, the diameter of unenlarged-diameter section, and the diameter of enlarged-diameter section, borehole spaces of 0.8 m, 1.6 m, 2.4 m, and 3.2 m were selected to investigate the pressure relief effect and roadway deformation (Figure 11) (Table 4).

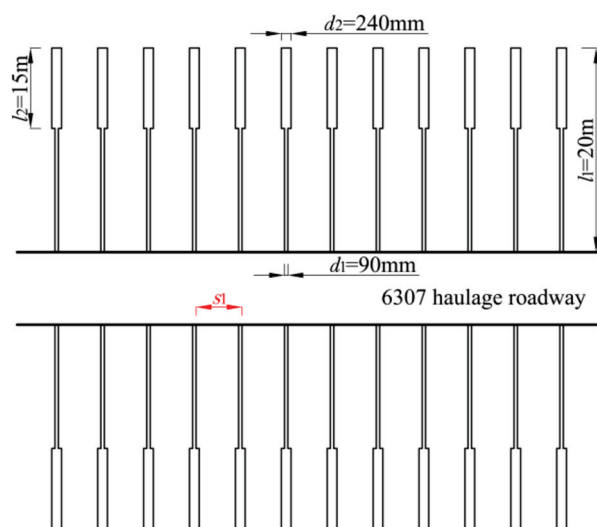


Figure 11. Diagram of the simulation scheme for different borehole spaces.

Table 4. Simulation scheme for different boreholes spaces.

No.	Total Borehole Length l_1/m	Length of the Enlarged-Diameter Section l_2/m	Diameter of the Unenlarged-Diameter Section d_1/mm	Diameter of the Enlarged-Diameter Section d_2/mm	Borehole Space s_1/m
1	20	15	90	240	0.8
2	20	15	90	240	1.6
3	20	15	90	240	2.4
4	20	15	90	240	3.2

3.2. Effect of Diameter of the Enlarged-Diameter Section on Pressure Relief Effect and Roadway Deformation

Figures 12 and 13 respectively show the distribution maps and curves of elastic strain energy density around boreholes with different diameters of the enlarged-diameter section. With increasing diameter of the enlarged-diameter section, the elastic strain energy density around the borehole gradually decreased (Figure 12). For conventional large-diameter pressure relief boreholes (the diameter of borehole was 90 mm, as shown in Figure 12a), a small low elastic strain energy density zone was observed around a single borehole. When the diameter of the enlarged-diameter section was 190 mm, the low energy zones around borehole expand and overlap (Figure 12c), and the energy peak was 15.3 m away from the coal side (Figure 13). With increasing diameter of the enlarged-diameter section, the energy peak gradually transferred into the deep surrounding rock. When the diameter of the enlarged-diameter section increased to 240 mm, the energy accumulation area was transferred to the end of the borehole (Figure 12d), and the energy peak was about 18.5 m away from the roadway side (Figure 13).

Borehole pressure relief reduces the peak elastic strain energy density of surrounding rock (Figure 13). With increasing diameter of the enlarged-diameter section, the peak elastic strain energy density of the coal body gradually decreases, from 5.3×10^5 J (diameter of enlarged-diameter section was 90 mm) to 4.2×10^5 J (diameter of enlarged-diameter section was 240 mm).

Figure 14 shows the displacement curves of surrounding rocks under the different diameters of the enlarged-diameter section. With increasing the distance from the coal wall, the displacement of the measured points in the surrounding rock gradually decreased (Figure 14). With increasing diameter of enlarged-diameter section, the displacement on both sides of the roadway changed slightly.

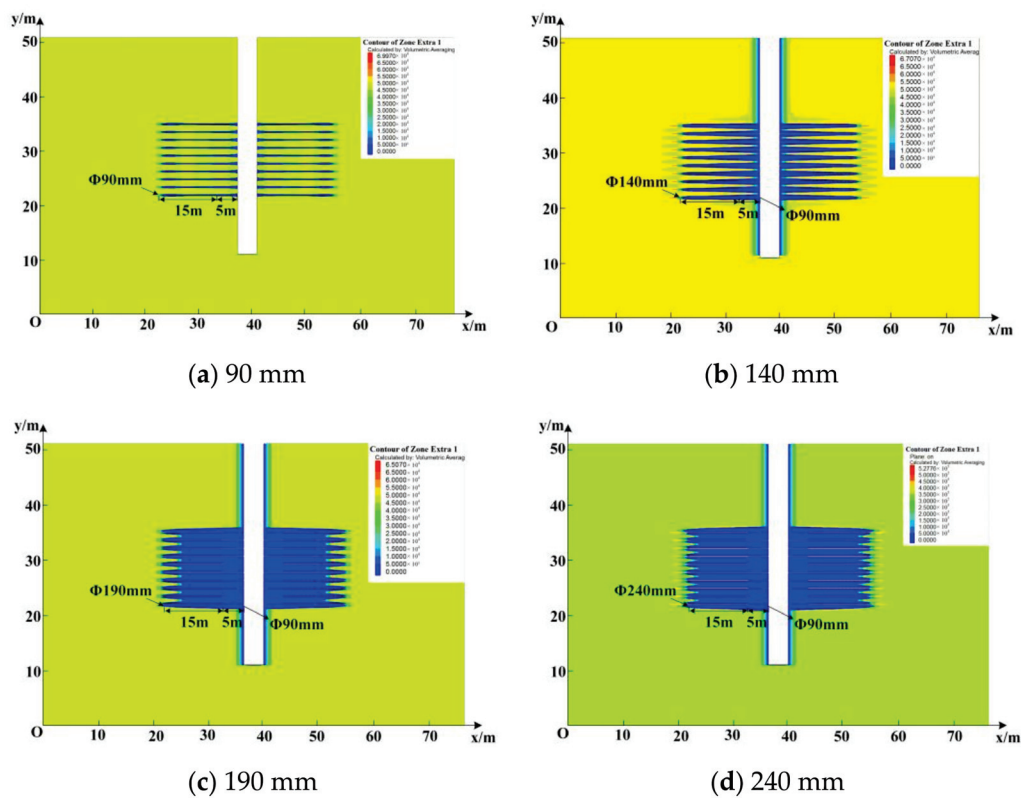


Figure 12. Distribution maps of elastic strain energy density around boreholes under different diameters of the enlarged-diameter section.

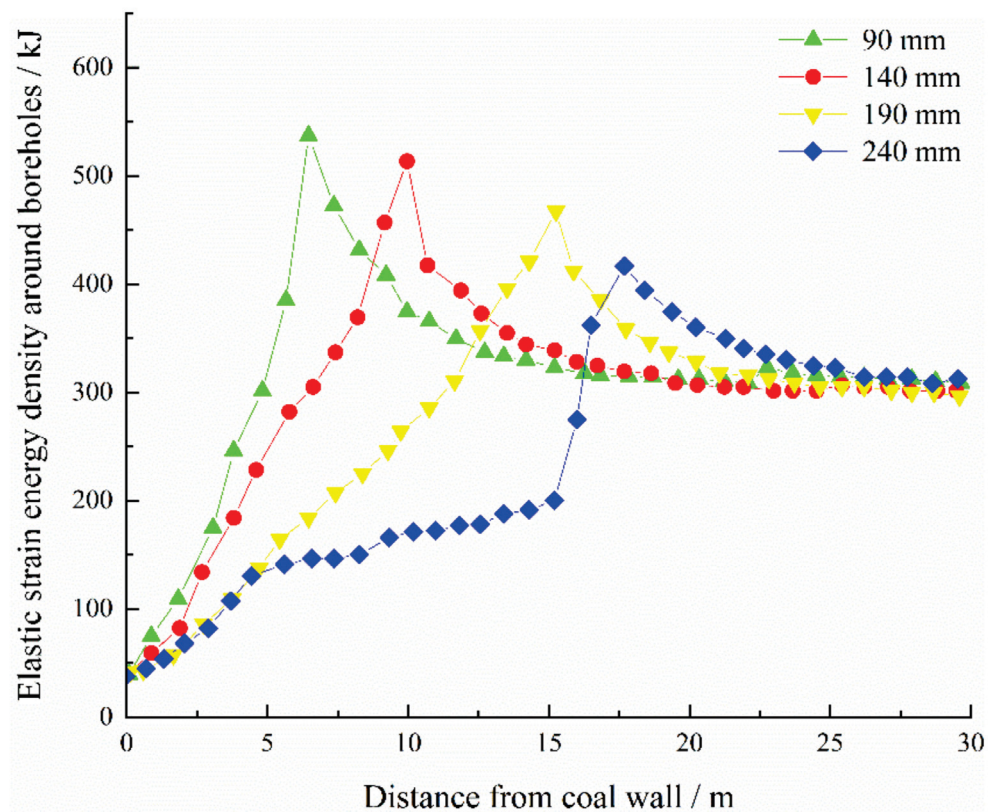


Figure 13. Distribution curves of elastic strain energy density around boreholes under different diameters of the enlarged-diameter section.

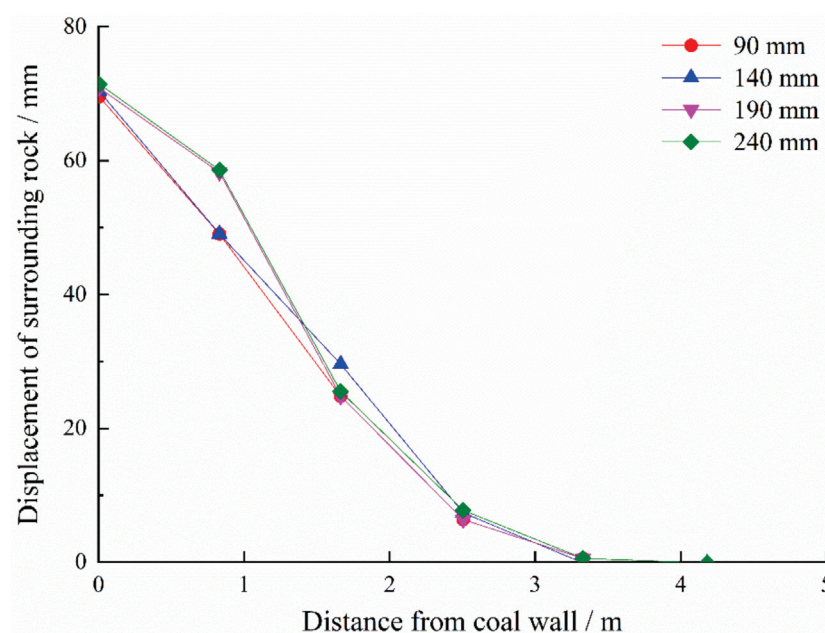


Figure 14. Displacement curves of surrounding rock under different diameters of the enlarged-diameter section.

3.3. Effect of Lengths of the Enlarged-Diameter Section on Pressure Relief and Roadway Deformation

Figures 15 and 16 respectively show the distribution maps and curves of elastic strain energy density around boreholes for different lengths of the enlarged-diameter section. The elastic strain energy density around the borehole decreased significantly with increasing the length of the enlarged-diameter section (Figure 15). When the length of the enlarged-diameter section was 15 m, the low elastic energy density zones around the borehole began to overlap (Figure 15c), and the peak strain energy density around borehole decreases from 5.3×10^5 J (the length of the enlarged-diameter section was 11 m) to 4.2×10^5 J (Figure 16). It can be concluded that increasing the length of the enlarged-diameter section can effectively reduce the accumulated energy.

Figure 17 shows the displacement curves of surrounding rocks under different lengths of the enlarged-diameter section. The displacement variation of the two sides of the roadway under the conditions of enlarged-diameter borehole lengths of 11 m, 13 m, and 15 m was similar. When the length of enlarged-diameter section was longer than 15 m (17 m and 19 m, Figure 17), the displacement of the two sides of the roadway (0 m away from the roadway side, Figure 17) increases by 42.9%.

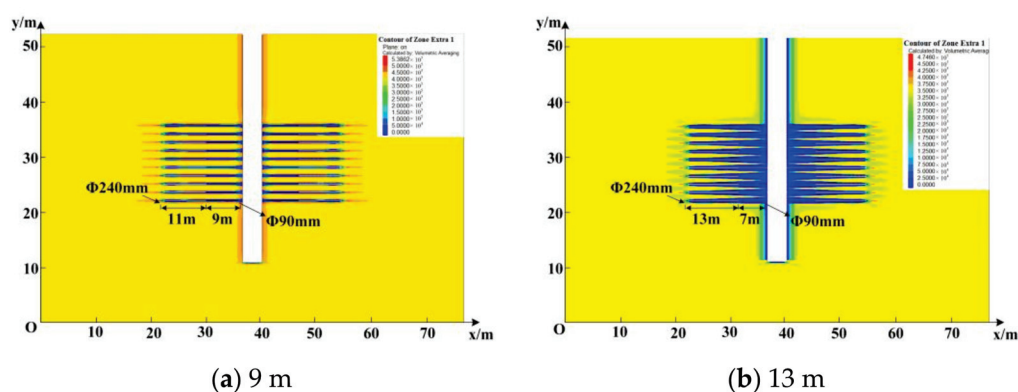


Figure 15. Cont.

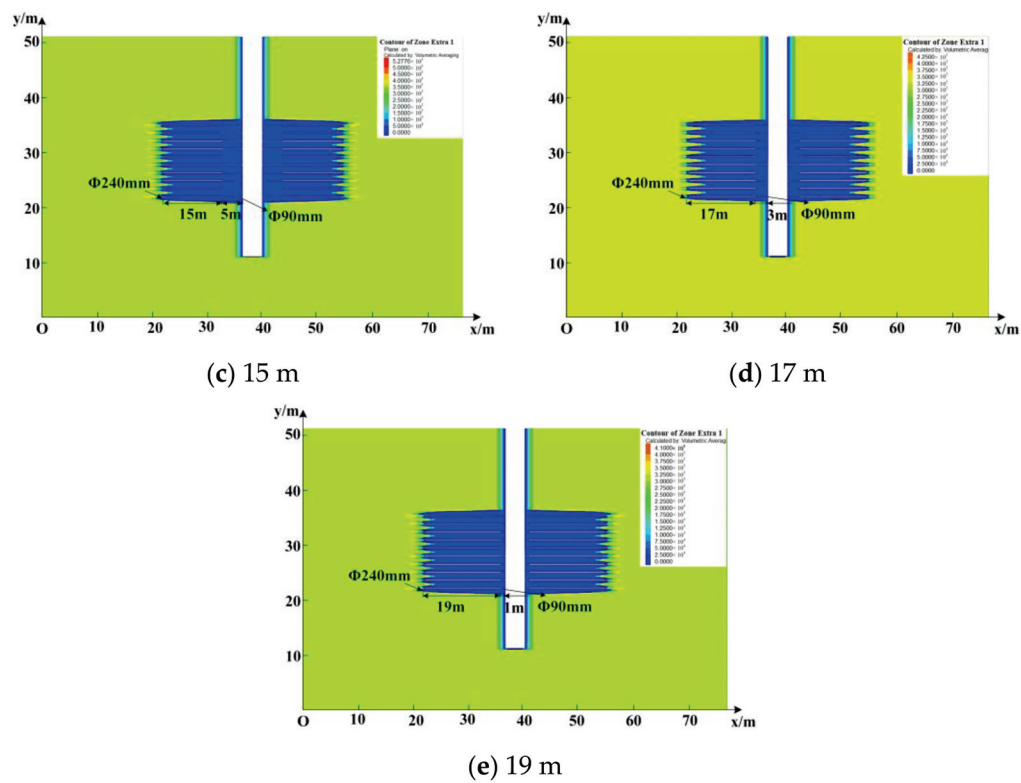


Figure 15. Distribution maps of elastic strain energy density around boreholes under different lengths of the enlarged-diameter section.

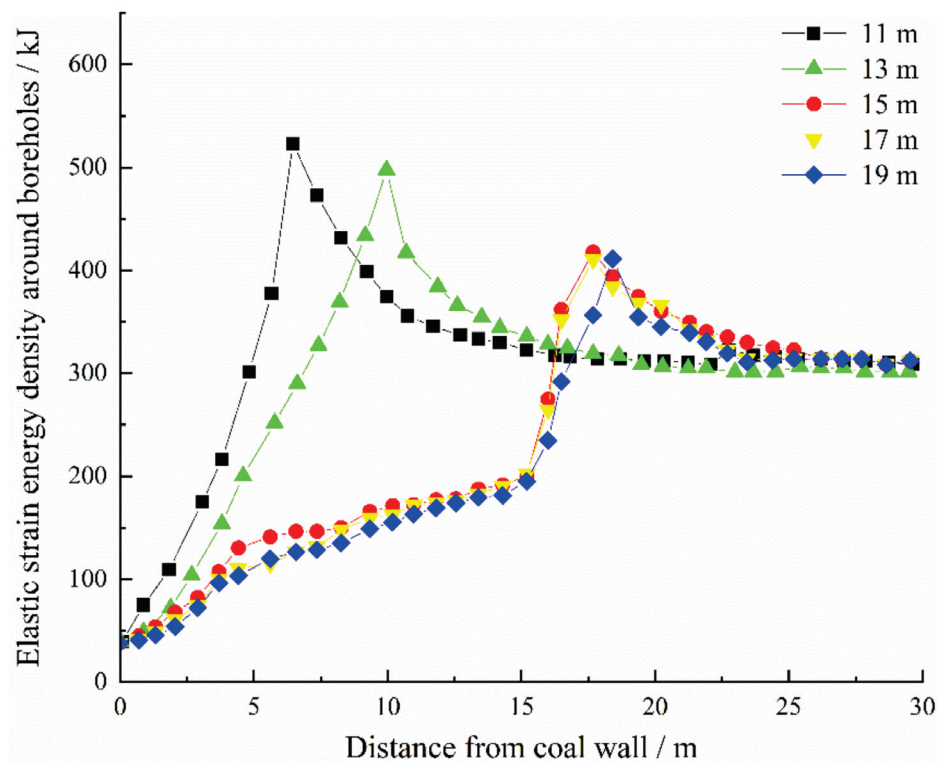


Figure 16. Distribution curves of elastic strain energy density around boreholes under different lengths of the enlarged-diameter section.

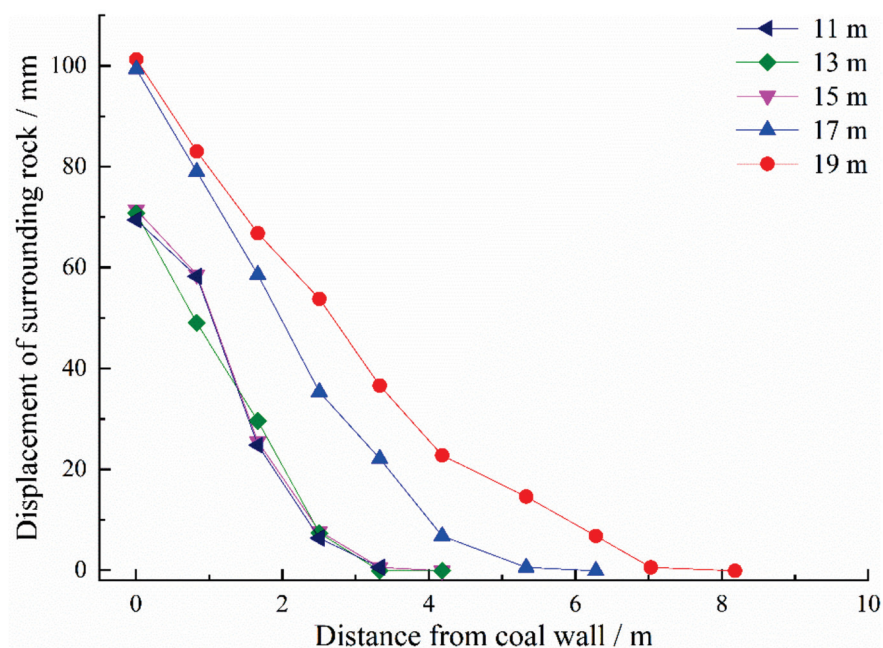


Figure 17. Displacement curves of surrounding rock under different lengths of the enlarged-diameter section.

3.4. Effect of Borehole Space on Pressure Relief Effect and Roadway Deformation

Figure 18 shows the distribution curves of elastic strain energy density around boreholes under different borehole spaces. With decreasing borehole space, the peak strain energy density decreases significantly. When the borehole space was changed from 3.2 m to 0.8 m, the peak elastic strain energy density decreased by 19.6%. The pressure relief effect for a borehole space of 1.6 m and 0.8 m was the same.

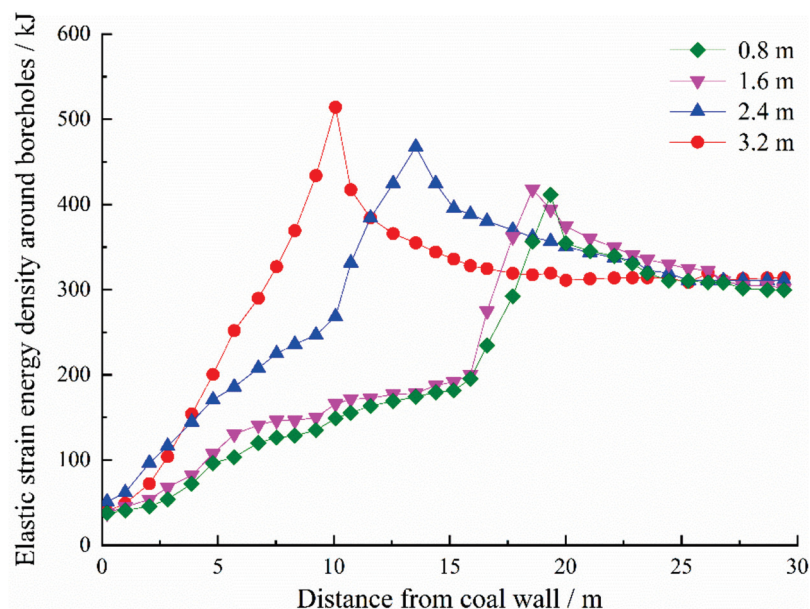


Figure 18. Distribution curves of elastic strain energy density around boreholes under different borehole space.

Figure 19 shows the roadway displacement curves for different borehole spaces. With decreasing borehole space, displacement on both sides of the roadway increases. When the borehole space was 0.8 m, the displacement was 103.3 mm. Compared with a borehole

space of 0.8 m, the displacements of surrounding rocks (0 m away from roadway, Figure 19) under a borehole space of 1.6 m, 2.4 m, and 3.2 m decreased by 23.3%, 25.6%, and 28.3%, respectively.

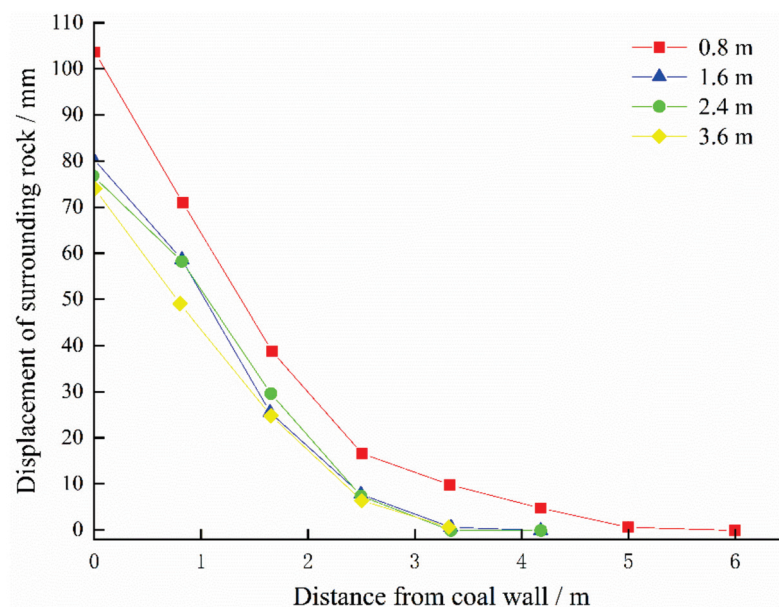


Figure 19. Displacement curves of roadway under different borehole spaces.

4. Field Test

The burial depth of the 6307 working face in Tangkou coal mine is approximately 980 m. After roadway excavation, a conventional large diameter borehole is constructed on the roadway side. According to the initial design scheme, the pressure relief borehole is 1.2–1.5 m away from the coal seam floor, with a 150 mm diameter, 20 m depth, and 0.8 m space, (Figure 20).

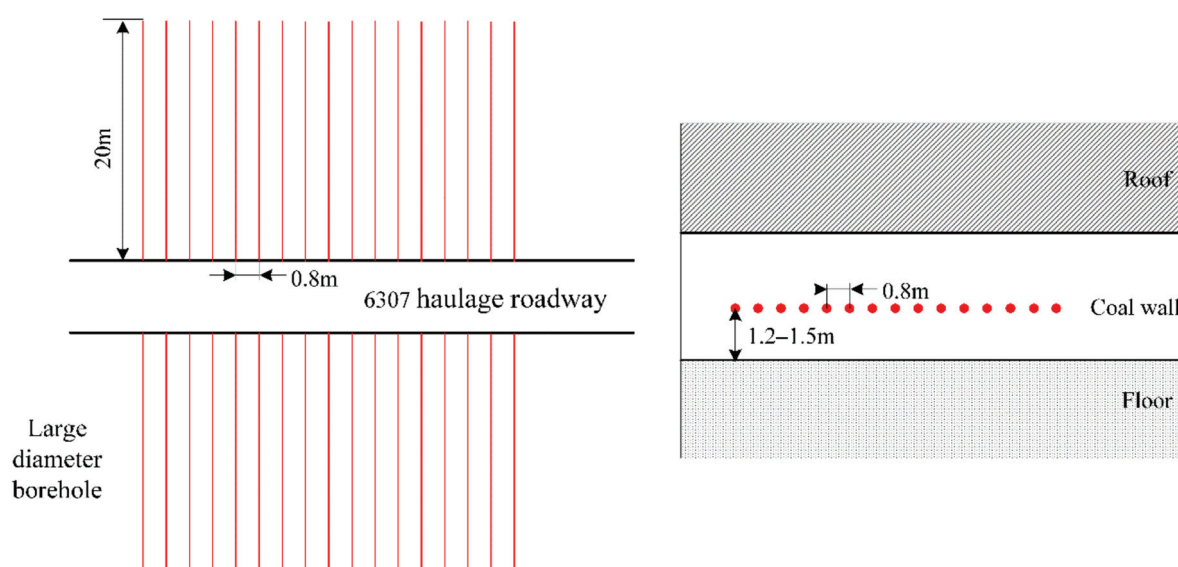


Figure 20. Diagram of large diameter pressure relief boreholes.

Figure 21 shows the displacement monitoring results of the roadway before and after the borehole pressure relief. At the initial monitoring stage, the deformation of surrounding rock increased significantly and entered the stable stage after 15 days (Figure 21). Before and after pressure relief, the displacement between roof and floor is the same, while the

displacement between the two sides of the roadway after pressure relief is 1.8 times that before pressure relief.

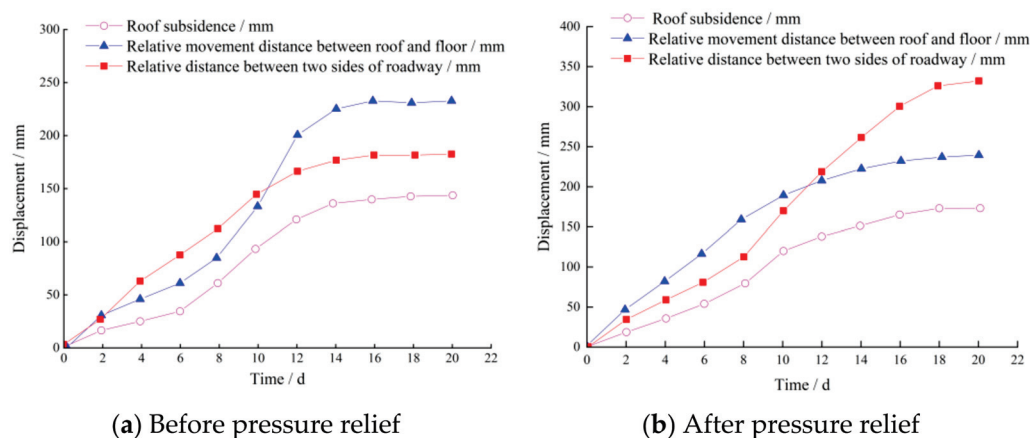


Figure 21. Roadway displacement curves.

Figure 22 shows the stress monitoring results of bolts before and after borehole pressure relief. At the initial stage of monitoring, the bolt stress increased significantly, then remained at a relatively stable value after 15 days (Figure 22). Where no pressure relief borehole were constructed, the stress on the two sides of the roadway stabilized at 70 kN (Figure 22). For the borehole pressure relief zone, the final stress of bolt ②, bolt ①, bolt ④, and bolt ③ is 49 kN, 94 kN, 60 kN, and 105 kN, respectively (Figure 22b), and the stress of the bolt near the borehole pressure relief zone is far less than that far from the pressure relief zone.

According to the field monitoring results, the concentrated high stress zone on the roadway side was transferred to the deep area due to borehole pressure relief, which reduces the possibility of rock burst. However, the construction of the pressure relief borehole also damaged the integrity of the surrounding rock and the support system. In some of the pressure relief zones, the bolt was broken. It is expected that the two sides of the roadway experience serious deformation after mining due to the influence of advanced abutment pressure.

4.1. Key Parameters for Segmented Enlarged-Diameter Borehole

Based on the theoretical and numerical simulation results, the key parameters of segmented enlarged-diameter borehole were designed in a 100 m length roadway in the 6307 working face.

(1). Total borehole length

Referring to the borehole pressure relief parameters of the adjacent working face of the 6307 working face, the length of the pressure relief borehole of the 6307 working face was determined to be 20 m.

(2). Diameter of the unenlarged-diameter section

According to the simulation results in Section 3.2, when the diameter of the hole in the unenlarged-diameter section was 90 mm, the roadway deformation was relatively small. Thus, the diameter of the borehole in unenlarged-diameter section was determined to be 90 mm.

(3). Diameter of enlarged-diameter section

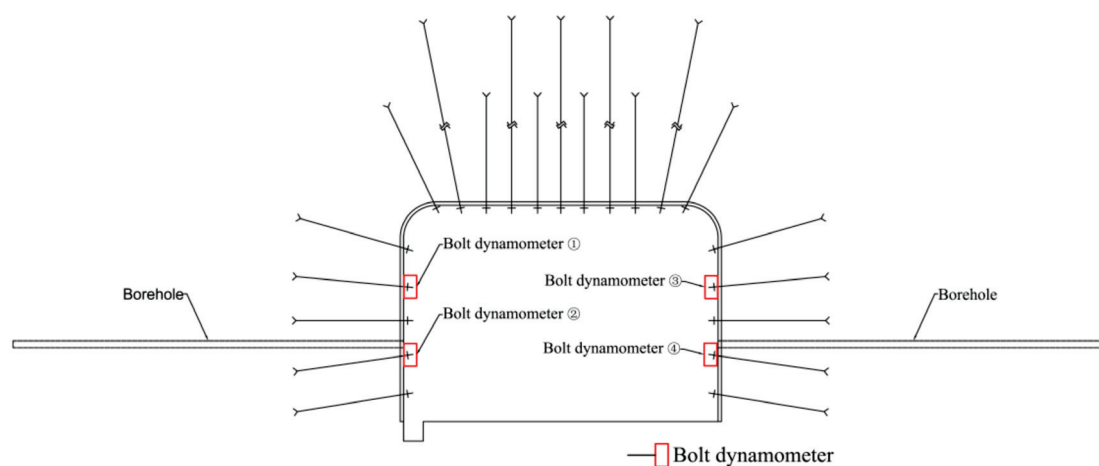
According to the simulation results in Section 3.2, with increasing diameter of the enlarged-diameter section, the peak elastic strain energy density gradually transferred to the deep surrounding rock of the roadway. The displacement on both sides of the roadway was less affected by the enlarged diameter section. Therefore, the diameter of the enlarged-diameter section was determined to be 240 mm.

(4). Length of enlarged-diameter section

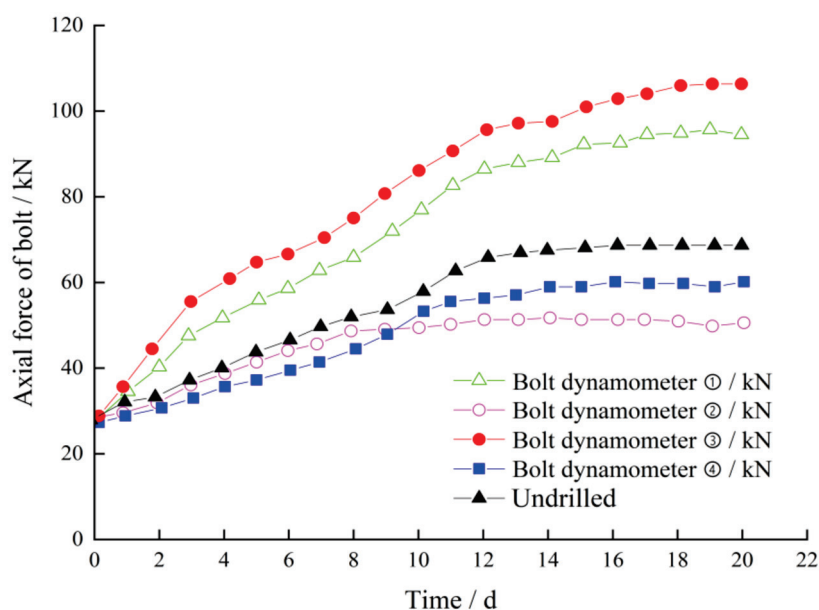
Based on the roadway support design in the 6307 working face, the length of unenlarged-diameter was calculated to be no less than 3.7 m (Equation (14)). According to the simulation results (Section 3.3), it has little influence on the support system when the position of enlarged-diameter section was 5 m away from the roadway side. The length of enlarged-diameter section in roadway of 6307 working face was determined to be 15 m.

(5). Borehole space

According to the simulation results in Section 3.4 (Figure 18), when the space between pressure relief boreholes was less than 1.6 m, the decrease in space will not improve the pressure relief effect. However, it will significantly increase roadway deformation. Considering the construction and safety factors, the space of pressure relief borehole was determined to be 1.6 m. Table 5 and Figure 23 present the key parameters of the segmented enlarged-diameter borehole in the 6307 working face roadway.



(a) Arrangement of stress measuring point

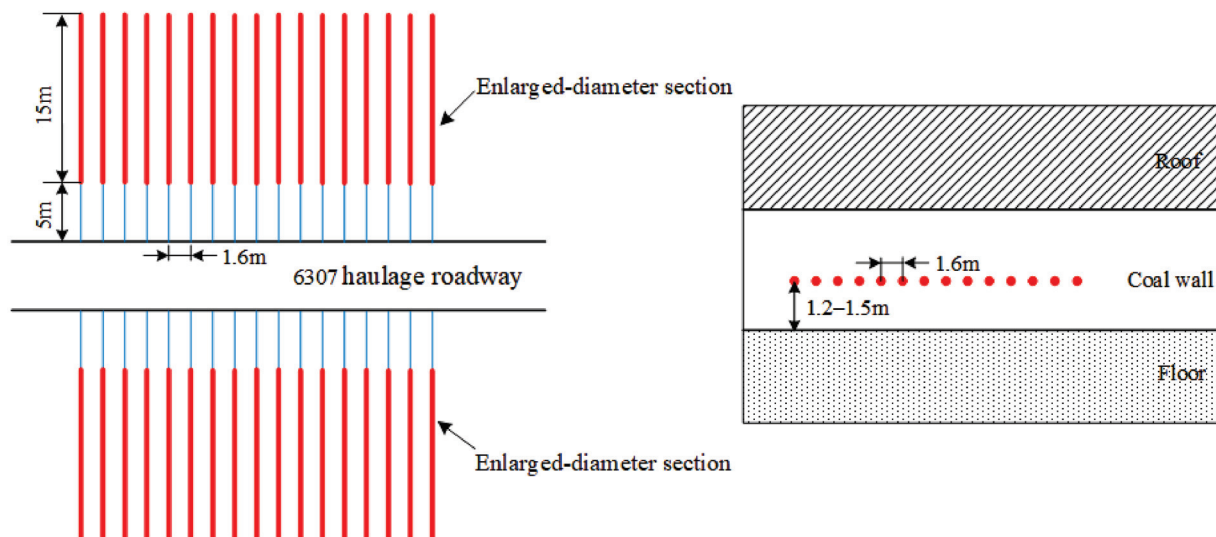


(b) Monitoring results

Figure 22. Stress monitoring of bolt.

Table 5. Key parameters of segmented enlarged-diameter borehole destressing technology.

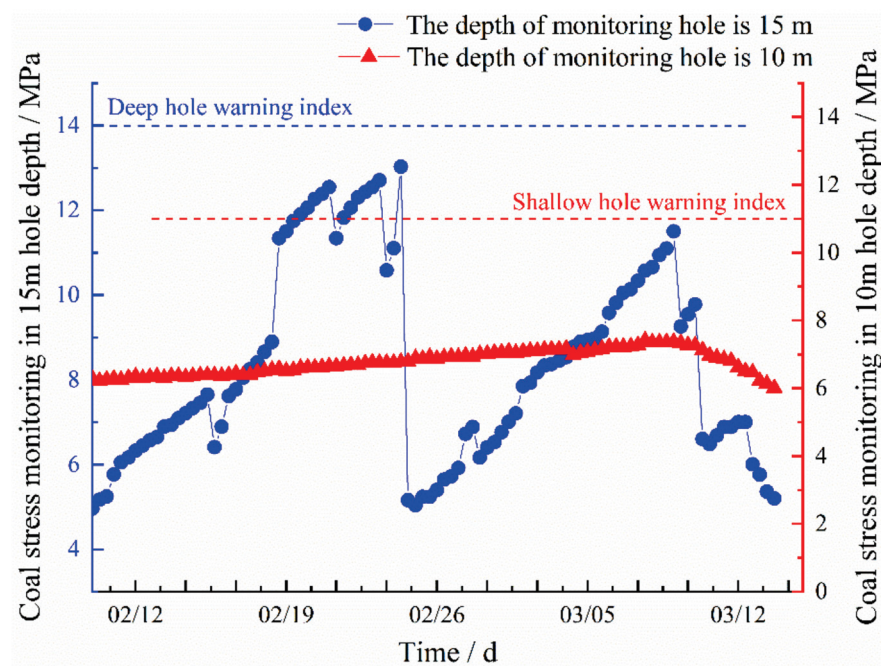
Total Length of Borehole/m	Diameter of Enlarged-Diameter Section/mm	Diameter of Unenlarged-Diameter Section/mm	Length of Enlarged-Diameter Section/m	Borehole Space/m
20	240	90	15	1.6

**Figure 23.** Diagram of segmented enlarged-diameter borehole.

4.2. Pressure Relief Effect Monitoring

4.2.1. Coal Stress Monitoring

Figure 24 shows the coal stress monitoring results obtained using the stress online monitoring system (the stress gauges for the shallow hole and deep hole were arranged between the two segmented enlarged-diameter pressure relief boreholes). The early warning indices are also presented in Figure 24.

**Figure 24.** Coal stress monitoring results.

In the initial stage, the stress value of the shallow hole (10 m) was relatively small (6.5 MPa). As the working face continued to advance, the stress gradually increased to 7.4 MPa, which is less than the early warning index (11.0 MPa). For the measuring point in the deep hole (15 m), as the working face advanced, the stress peak first reached 13.0 MPa (the early warning index was 14.0 MPa) and then dropped to 5.3 MPa, indicating failure of the coal body. The energy accumulated in the coal body was effectively reduced by the segmented enlarged-diameter pressure relief.

4.2.2. Roadway Deformation Monitoring

After the segmented enlarged-diameter borehole pressure relief was implemented, 10 monitoring sections were arranged to monitor the displacement of the roof, floor, and roadway side. The monitoring data were compared with the roadway deformation data in the zone where the conventional large-diameter boreholes pressure relief were implemented (Figure 25).

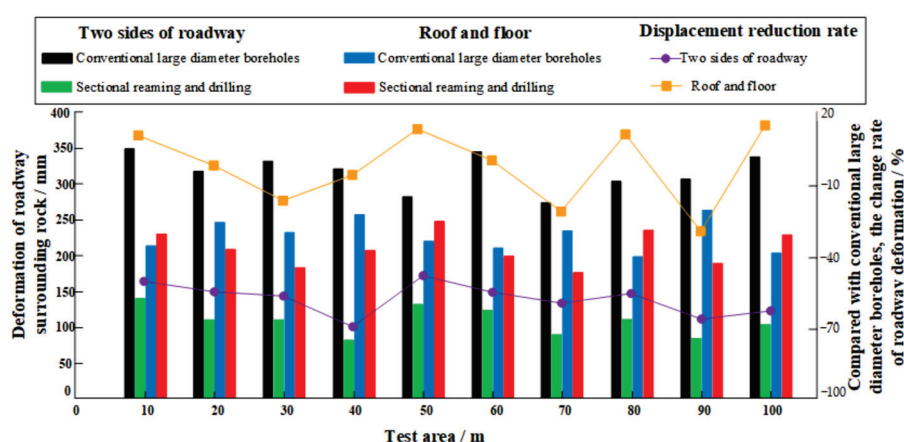


Figure 25. Monitoring data of maximum roadway deformation.

Compared with the zone of conventional large-diameter pressure relief, the displacement of the roadway roof, floor, and decreased in the zone of segmented enlarged-diameter borehole pressure relief. The displacement of the roadway side decreased more obviously. The maximum displacement of the roof and floor in the zone of segmented enlarged-diameter borehole pressure relief was about 250 mm, which was 7.5% lower than the zone of conventional large-diameter pressure relief.

For the displacement of the roadway side, the maximum displacement in the zone of conventional large-diameter pressure relief was about 350 mm, while it was only 150 mm in the zone of segmented enlarged-diameter borehole pressure relief, indicating that the segmented enlarged-diameter borehole destressing technology can effectively reduce roadway deformation.

5. Conclusions

In this study, segmented enlarged-diameter borehole destressing technology was investigated and applied to the 6307 working face of Tangkou Coal mine. The main conclusions are:

- (1). According to the theory of elastic-plastic mechanics, the distribution range and influencing factors of the coal pressure relief zone around the borehole are obtained. Furthermore, the minimum length of the unenlarged-diameter section of the borehole was determined.
- (2). The effects of diameter of the enlarged-diameter section, length of the enlarged-diameter section, and borehole space on pressure relief and roadway deformation were investigated. The larger the diameter of the enlarged-diameter section, the better the pressure relief effect. With increasing diameter of the enlarged-diameter

section, the displacement on both sides of the roadway changed slightly. Increasing the length of the enlarged-diameter section can effectively reduce the energy accumulated around the boreholes and transfer the energy peak to the deep surrounding rock. However, the longer the length of the enlarged-diameter section, the larger the deformation of the surrounding rock. With decreasing borehole space, the peak energy around the boreholes decreased, and the deformation of both sides of the roadway increased significantly.

- (3). The key parameters of segmented enlarged-diameter pressure relief borehole were determined for 6307 working face. Field monitoring results showed that the accumulated energy can be effectively reduced using segmented enlarged-diameter pressure relief boreholes, effectively controlling the roadway deformation. Segmented enlarged-diameter pressure relief technology can effectively mitigate the problems of excessive pressure relief (the strength of the surrounding rock was reduced after pressure relief) and insufficient pressure relief (rock burst still occurred after pressure relief) as well as provide a reference for the rock burst prevention and roadway stability control.

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Article

Investigation on Characteristics and Prevention of Rockburst in a Deep Hard and Soft Compound Stratum Tunnel Excavated Using TBM

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Abstract: Rockburst disasters frequently occur in deep tunnels excavated by TBM (tunnel boring machine) under complex geological conditions in western China. Using FLAC3D, the characterization of a three-dimensional numerical model of a compound stratum tunnel excavated by TBM is conducted, based on a water transport project in Shanxi Province. Then, the characteristics of rockburst in deep hard and soft compound stratum tunnels excavated by TBM are revealed, and the energy criteria of the rockburst considering the rock brittleness are proposed. In addition, the prevention and control method of drilling pressure relief for rockburst has been investigated. Results show that: (i) the rockburst risk of compound stratum tunnel excavated using TBM is mainly in the upper-hard rock part, while there is no rockburst risk in the soft rock part; (ii) after the excavation of the tunnel, slight rockburst risk occurs first in the hard rock area of the tunnel vault, and then the rockburst risk gradually rises to the strong level at 7 m behind the working face, indicating the hysteresis of strong rockburst; (iii) the rockburst in the vault of the rock surrounding the compound stratum tunnel has the effect of forming the deepest area, gradually narrowing to both sides, showing a “V” shape, and the occurrence of rockburst may not be completed at one time; (iiii) borehole pressure relief can significantly reduce the rockburst risk of surrounding rock in a tunnel. The larger the borehole diameter and depth, the better the effect of rockburst prevention. In addition, the effect of borehole diameter is more significant than depth. The research results provide guiding references for the prevention and control of similar rockburst disasters in underground engineering.

Keywords: hard and soft compound stratum; TBM; rockburst; regional effect; numerical simulation; borehole pressure relief

1. Introduction

With the in-depth implementation of China’s “Western Development” and “One Belt and One Road” strategies, resource mines, water conservancy structures, and traffic tunnels are gradually being transferred to western China [1], which has a topography of high mountains and deep valleys. In this region, the stratum structure is variable due to the significant burial depth and the effect of regional tectonic activities over multiple periods; thus, the soft and hard strata appear alternately. Compared with the central and eastern regions, the sharp in situ stress environment and composite stratum structure are essential characteristics [2]. Rockburst occasionally occurs in TBM construction of deep tunnels in China, resulting in severe equipment damage, construction delay, and significant economic and property losses [3–6]. The diversion tunnel of Jinping II Hydropower Station, Qinling

tunnel of Han-Ji-Wei River Diversion Project, and Zhongtianshan Tunnel are some places where this type of incident has occurred. During TBM construction, rockburst disasters with various characteristics occur several times, causing severe equipment damage, construction delays, and considerable economic and property losses.

Rockbursts in TBM construction tunnels have several characteristics, such as uncertainty, hysteresis, and massive damage, which creates obstacles to predicting and controlling the rockburst mechanism [7,8]. To investigate the related characteristics in depth, domestic and foreign scholars have carried out research work on rockburst in tunnel TBM construction. Xie et al. [9] stated that TBM tunnelling generates a lesser disturbance to the surrounding rock at the excavation boundary, ensures better integrity, and leads to the smoother releasing of surrounding rock strain energy; however, it should be noted that despite all this, due to its higher storage capacity and accumulation of surface surrounding rock strain energy, it easily induces rockburst damage. Wu et al. [10] investigated the diversion tunnel of Jinping II Hydroelectric Power Plant through microseismic monitoring and numerical simulation and found that it has a lithology of thick, dense, fine crystal marble noise, and the local joint development is in the good surrounding rock. For related tunnels, the initiation of rockburst under tunnel TBM excavation has mainly occurred at the tunnel vault and arch shoulder and gradually propagated to the deeper part of the surrounding rock. In another study performed by Chen et al. [11], an acoustic emission test was performed on the selected sites of the #1 test tunnel (referred to as 2-1 test tunnel) of #2 Hengtong Tunnel of Jinping II Hydropower Station. The overlying rock is about 1850 m thick, and the lithology is t-gray layered medium-coarse marble with relatively developed joints and cracks. Results obtained from a field acoustic emission test showed that the fracture damage of the surrounding rock under TBM construction was found in the most critical situation at 3 m after the palm face, and the damage of the surrounding rock has a certain hysteresis. Jiang et al. [12] proposed using the radial gradient of the elastic strain energy density of surrounding rock to predict the location of rockburst, according to the energy criterion of rockburst in TBM construction established by the authors based on the energy principle of rockburst. To reduce the risk of rock explosion in TBM construction of deeply buried tunnels, some scholars [13] have suggested that a combined excavation process consisting of the drill-and-blast method for tunnel guidance and TBM boring under high-stress conditions could be effective; hence, a series of measures were proposed to prevent and control the rock explosion. At present, there are several rock burst prevention and control methods, including drilling and pressure relief, as well as blasting [14–16]. However, the step-by-step excavation method of pre-guided cavern undoubtedly leads to an increment in the number of perturbations of the surrounding rock, which easily induces the generation of shear damage due to the high stress surrounding the rock and thus raises the risk of rock explosion [17].

The majority of the research outlined above concentrates on rockbursts in TBM construction for single hard rock strata; however, few studies have been performed on the rockburst characteristics and preventative measures for TBM construction in composite strata. Due to composite strata's particular stratigraphic structure and stress environment, the characteristics of rock explosion differ; thus, prevention and control approaches are reasonably different than those in single hard rock strata. Thus, the destructive nature and frequency of rock explosions during deep-tunnel TBM construction would be aggravated. Consequently, in the current paper, theoretical analysis and numerical simulation methods were applied to reveal the characteristics of rockbursts in TBM construction tunnels in deep soft and hard composite strata by taking as background a water transmission project in Shanxi. Moreover, to provide a reference for the prevention and control of rockburst disasters in tunnel TBM construction under similar conditions, methods of unloading and preventing rockbursts in boreholes were developed.

2. Engineering Geology of the Research Area

2.1. Engineering Background

Located in the research area, Shanxi Province, the water transport project includes the trunk line, the east trunk line, the west trunk line, and each water supply branch line; all of the #3 tunnel entrances are located approximately 450 m below the Tianguya Reservoir in Xingxian County on the left bank of Lanyi River, and the exit is located on the left bank of Beichuan River, about 3.6 km south of Dawu Town in Fangshan County. The tunnel is divided into three TBM sections. The main tunnel of the TBM-2 section was obtained via TBM excavation, with a total length of 20.7 km, a maximum buried depth of 590 m, a diameter of 4.9 m, and an inner diameter of 4.3 m after the completion of segment support. The main hole is formed using Robbins dual-shield TBM, consisting of front and middle telescopic shields and tail shields.

2.2. Formation Profile

During the process of TBM, the strata through which the main hole of the TBM-2 section of the total dry #3 tunnel passed are Cambrian, Ordovician tuff, dolomite, marl and mudstone, Swire boundary striped diorite hornblende, and unequal grain black cloud plagioclase gneiss. Among these, the rocks surrounding the tuff, dolomite, quartzite, hornblende, and gneiss section are hard or medium-hard rocks, and the geological classification of the surrounding rocks is type III, while the surrounding rocks of the marl and mudstone section are soft rocks, and the engineering geological classification of the surrounding rocks is type IV or type V. Due to the influence of orogenic movement and fold structure, the lithologic characteristics of the main tunnel in the TBM-2 section of the #3 tunnel are characterized by a typical TBM construction of soft and hard composite stratum.

2.3. Distribution of In Situ Stress

The results of the on-site ground stress test and reversal calculation [18] about the research area showed the following: horizontal tectonic stress was dominant along the main stem #3 tunnel, and the maximum horizontal principal stress direction was N82°E–N86°E. This ground stress direction was perpendicular to the tunnel axis direction and was in the most unfavorable direction for the tunnel's surrounding rock stability. The direction of ground stress on the tunnel cross section is shown in Figure 1. As a result of the regression analysis to obtain the principal stresses, the maximum principal stress σ_1 was calculated as 21.16 MPa, the intermediate principal stress σ_2 as 17.72 MPa, and the minimum principal stress σ_3 as 15.6 MPa at the buried depth of the diversion tunnel, about 600 m.

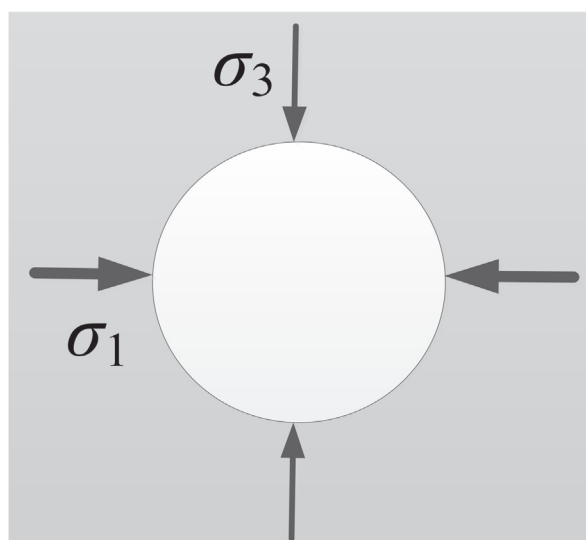


Figure 1. The in situ stress direction of tunnel cross-section.

3. Tunnel Numerical Model Creation

3.1. Model Establishment

Based on the engineering background and geological conditions mentioned above, the three-dimensional numerical calculation model was created with the help of FLAC3D software. To facilitate the comparative analysis of the computational results, the model was simplified by considering the typical upper-hard and lower-soft composite stratigraphic structure, as shown in Figure 2. The tunnel section was circular, with a diameter of 5 m. To reduce the model boundary effect, the lengths of the model's horizontal direction (X) and vertical direction (Z) were taken as 12 times the diameter (i.e., 60 m). In addition, the longitudinal (Y) direction is taken as 60 m. The boundaries of the model in all directions (left, right, front, back, and bottom) were constraints, which fixed their displacements. In addition, the upper boundary of the model was free. Considering the results of ground stress regression analysis, the horizontal x-direction of the model was the maximum principal stress direction. The axial direction of the tunnel (i.e., the y-direction of the model) was the intermediate principal stress direction, and the vertical z-direction of the model was the minimum principal stress direction. The loads applied in the x-, y-, z-directions were 21.16 MPa, 17.72 MPa, and 15.60 MPa, respectively. The hard surrounding rock of the simulated tunnel section was gneiss, and the weak surrounding rock was marl. Physical and mechanical parameters of rocks obtained from laboratory tests are shown in Table 1.

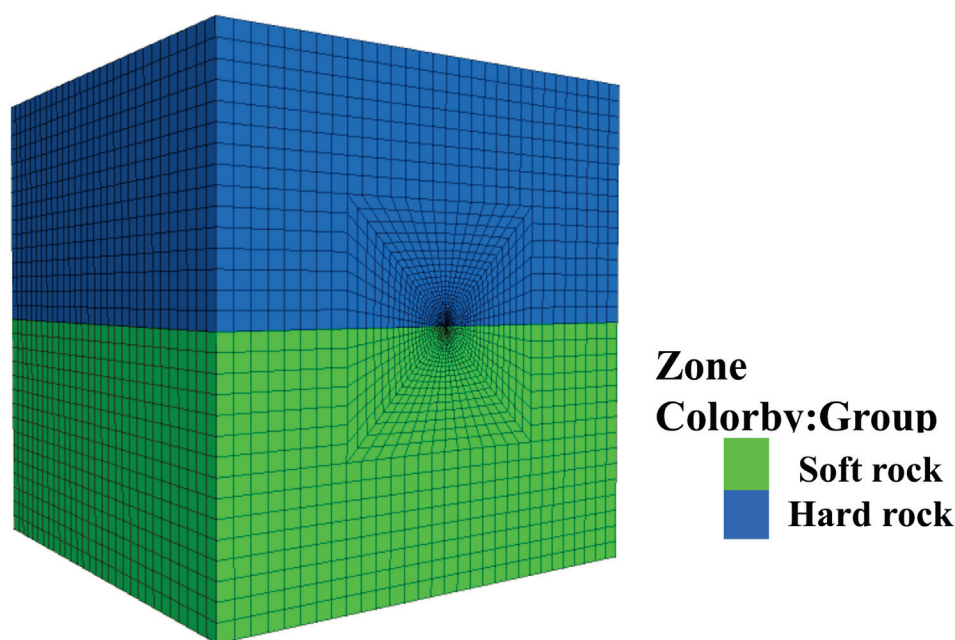


Figure 2. Numerical model of upper-hard and lower-soft stratum.

Table 1. Physical and mechanical parameters of rock.

Lithology	Elastic Modulus (GPa)	Poisson Ratio	The Angle of Internal Friction (°)	Cohesive Forces (MPa)	Uniaxial Compressive Strength (MPa)	Tensile Strength (MPa)
Gneiss	7.5	0.27	38	5.5	78	4.7
Marl	1.5	0.35	24	1.0	12	1.2

3.2. TBM Construction and Simulation Process

In order to simulate the construction process with TBM, as shown in Figure 3, simulation of the TBM body self-weight, rolling blade, and shield were implemented in the model [12,19].

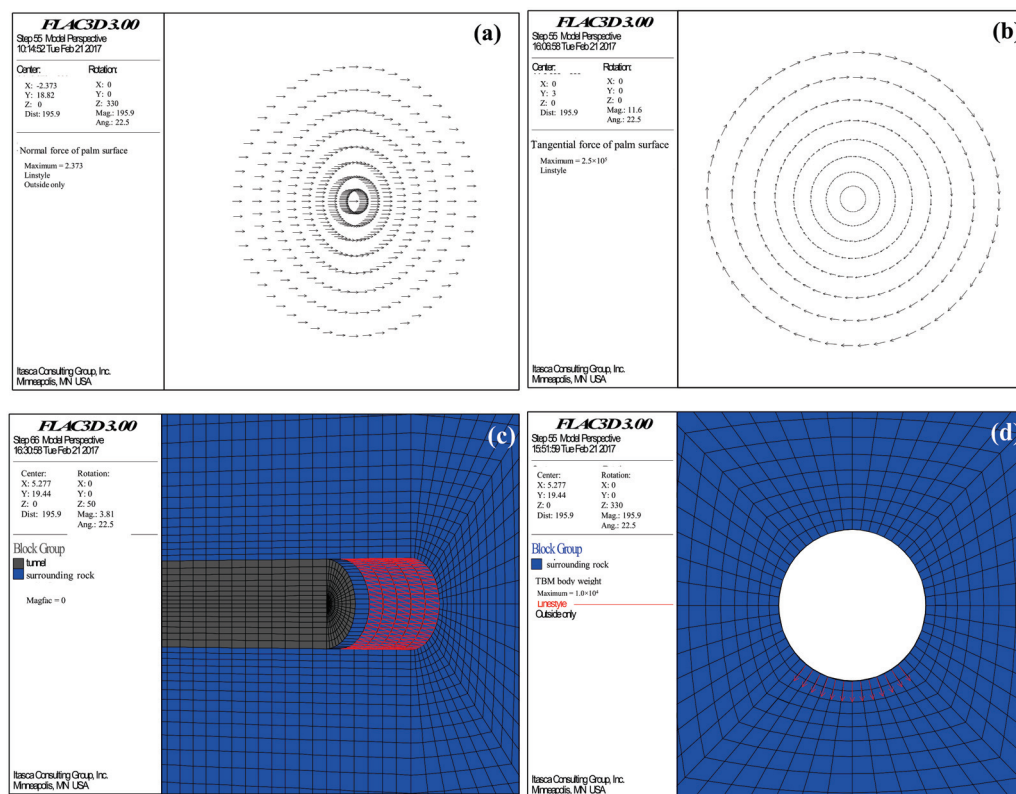


Figure 3. Numerical simulation of TBM excavation: (a) normal force; (b) tangential force; (c) shield; (d) bodyweight.

In terms of construction principle, TBM is a continuous mechanical excavation, which causes less disturbance to the surrounding rock. With the gentle excavation initiation, ground stress adjustment process, and slow stress release on the TBM boring excavation boundary, the surrounding rock stress-strain curve has a good continuity [20,21]. In terms of construction characteristics, the hob on the cutter during TBM excavation breaks the rock with a minimal infill for uninterrupted continuous excavation. There is no pause in TBM excavation except for particular circumstances, such as equipment maintenance, repair, and cutter replacement downtime. A common method used in current numerical modeling studies is to divide the continuous excavation of TBM into small excavation steps. However, the calculation time-step between excavation steps is often challenging to determine. This method's model forms the time-step continuity with high accuracy. In addition, the overall continuity and the degree of overall continuity depend on the size of the network partition.

In the numerical simulation, both excavation steps are calculated to be balanced, and there is a large pause in the simulation of the TBM forward movement, which does not reflect the continuity of the TBM excavation process well [22]. R. Hasanpour [23] pointed out that although the explicit time-step solution method in FLAC3D does not reflect the time-related stress-strain relationship with high accuracy, the method of gradually releasing the unbalanced force can effectively simulate the influence of time factors on the excavation process. To ensure that we were simulating the actual process of TBM excavation with maximum accuracy with the model used in the study, two major principles were proposed to be followed in the simulation calculation [24]: (1) the model grid size along the TBM excavation direction should be as small as possible but optimized so that it does not consume too many time steps; (2) the stress release of surrounding rock caused by excavation should be controlled, and the balance cannot be calculated between every two excavation steps.

Based on this, the surrounding rock stress-release rate control method employed in the numerical calculation process of the current study is as follows: (i) one step is calculated

after each excavation cycle is completed; (ii) after that, at the current excavation boundary, the maximum unbalanced force on the surface node of the surrounding rock is extracted and multiplied by the difference between the stress release coefficient and 1.0; (iii) the resultant value is then applied as a reverse load to the surface nodes of the surrounding rock of the excavation boundary and calculated to balance. The key benefit of this approach is that there is no need to set the number of calculation steps between two excavation steps; only the stress release coefficient of the surrounding rock is adjusted between two excavation steps. Furthermore, a stress release coefficient of 0.875 can be used to describe the ongoing process of TBM excavation accurately [23,25].

3.3. Constitutive Model Selection

The selection of the appropriate structural model has a significant impact on the simulation results. In this context, the constructive model reflects rock materials' strength and deformation properties in the most realistic way. In order to obtain the most realistic results, the elastic-plastic damage generation model [26] based on the Mogi–Coulomb criterion was adopted, and the yield function of the model was used as follows:

$$f(\sigma_{m,2}, J_2, D) = \frac{\sqrt{6}}{3} \sqrt{J_2} - b\sigma_{m,2} - (1 - D)a \quad (1)$$

where J_2 is the second invariant of deviatoric stress; $\sigma_{m,2}$ is the average effective normal stress on the shear plane; D is the damage variable; and a and b are material parameters related to rock mechanical properties, respectively.

Jiang et al. [26] obtained the relationship between the damage variation and the equivalent strain index through true triaxial testing, dividing the rock compression process into two stages, non-destructive and damaged. Additionally, they introduced the damage threshold, and set up the evolution equation of the damage variable, as shown below in Equations (2) and (3):

$$D = \begin{cases} 0 & 0 < \varepsilon_{eq} \leq \varepsilon_{eq}^0 \\ \beta \left\{ 1 - \exp \left[-\eta (\varepsilon_{eq} - \varepsilon_{eq}^0) \right] \right\} & \varepsilon_{eq}^0 < \varepsilon_{eq} \end{cases} \quad (2)$$

$$\varepsilon_{eq} = \sqrt{\frac{1}{2} [(\varepsilon_1 - \varepsilon_2)^2 + (\varepsilon_2 - \varepsilon_3)^2 + (\varepsilon_3 - \varepsilon_1)^2]} \quad (3)$$

where β is the residual strength coefficient; η is the positive proportional coefficient, which can be obtained from the test; ε_{eq} is the von Mises equivalent strain; and ε_{eq}^0 is the damage threshold represented by the equivalent strain.

The model was constructed based on the Mogi–Coulomb criterion, as the joint effect of plasticity and damage of rock materials was taken into account. In this way, it can more accurately describe the failure conditions and deformation characteristics of rock under true triaxial stress and is more suitable for deep underground engineering analysis under a three-dimensional stress state. The model parameter values are shown in Table 2.

Table 2. Model parameters [18].

Lithology	ε_{eq}^0	η	β
Gneiss	0.014	5	0.8
Marl	0.004	1	0.6

In order to verify the rationality of the model in the current study, the failure characteristics of surrounding rock under TBM construction in composite strata were simulated. The results obtained with the Mohr–Coulomb model were compared with the surrounding rock section 2 m behind the face of the tunnel face, as shown in Figure 4.

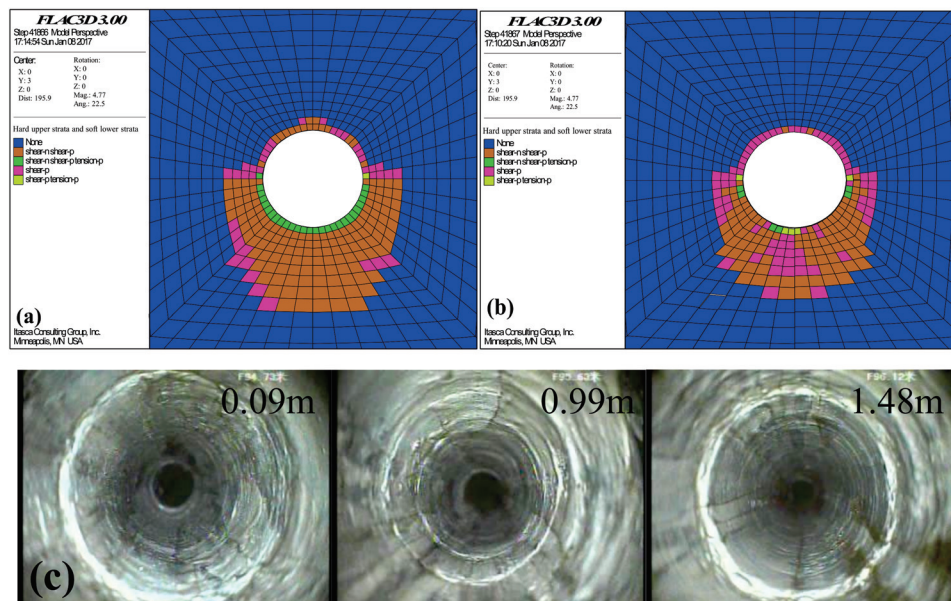


Figure 4. Distribution of plastic zone of TBM tunnel surrounding rock in the compound stratum. (a) Mohr–Coulomb model; (b) elastoplastic damage constitutive model; (c) the detection result of the distribution range of the cracks in the left side wall of the tunnel.

As can be seen from Figure 4, the Mohr–Coulomb model does not consider the effect of intermediate principal stress, so the calculated plastic zone range was wide. The extent of the plastic zone on the tunnel sidewall was about 2.02 m. Based on the Mogi–Coulomb criterion, the plastic zone of the surrounding rock was about 1.50 m. However, the actual field measurement of the tunnel left side wall fissure distribution depth was about 1.48 m. Therefore, it can be said that the calculation results of the elastoplastic damage constitutive model used in the current study reflected the real failure of surrounding rock more accurately.

4. Rockburst Energy Criterion Based on Rock Brittleness Index

Rockburst is of great harmfulness. A slight rockburst will result in the rock falling off in a flaky shape, and there will be no ejection. When a major rock burst occurs, it will feel like an earthquake with a magnitude of 4.6, and felt intensity is as high as 8 degrees. It will pose a serious threat of damage to the buildings in the immediate vicinity, due to the felt ground motion and loud noise, and will threaten the safety of the major projects still in progress and the lives of the relevant construction personnel. To discriminate the conditions under which rockburst will occur in the surrounding rock of underground engineering projects and the degree of damage it may cause, scholars at home and abroad have carried out a large number of relevant studies and investigations and have put forward many rockburst criteria and intensity classification standards from different angles, as shown in Table 3.

Except for the elastic energy index criterion and the Tao Zhenyu criterion, all of the above criteria are expressed in terms of tangential stress or maximum tangential stress, which requires coordinate transformation when using numerical calculation software to simulate rockburst risk; therefore, they are inconvenient to apply. Obtaining the elastic energy index criterion requires laboratory tests on rock, and it is measured using the integral area of the stress-strain curve under loading and unloading conditions. The rockburst discrimination result is greatly affected by test conditions and human factors. In point of fact, the occurrence of rockburst is closely related to the three-dimensional stress state of rock. The Tao Zhenyu rockburst criterion has a certain one-sidedness based on maximum principal stress, which is also proved by the rockburst engineering examples in China. With the development of rockburst energy theory, many scholars have introduced some energy

criteria of rockburst based on analysis of the energy transformation mechanism in rock deformation and failure, resulting in a good application effect.

Table 3. Rockburst criterion and intensity classification [17,27–29].

Rockburst Criterion	Country/ Regional Engineering	Rockburst Discriminant	Rockburst Intensity Classification			
			No Rockburst	Minor Rockburst	Medium Rockburst	Strong Rockburst
E. Hoek method	South Africa	$\sigma_{\theta\max}/\sigma_c$	<0.34	0.34~0.42	0.42~0.56	>0.70
Russenes criterion	Norway	$\sigma_{\theta\max}/\sigma_c$	<0.2	0.2~0.3	0.3~0.55	>0.55
Turchaninov criterion	Xibin Block	$(\sigma_{\theta} + \sigma_L)/\sigma_c$	<0.3	0.3~0.5	0.5~0.8	>0.8
Elastic energy index criterion	Mine, Kola Peninsula	$(\sigma_{\theta} + \sigma_L)/\sigma_c$	<0.3	0.3~0.5	0.5~0.8	>0.8
Elastic energy index criterion	Polish coal mine	$W_{et} = \Phi_{sp}/\Phi_{st}$	<2	2.0~4.9		≥ 5.0
Erlang Mountain highway tunnel criterion	China	σ_{θ}/σ_c	0.3	0.3~0.5	0.5~0.7	>0.7
Zhenyu Tao rockburst criterion	China	σ_c/σ_1	>14.5	14.5~5.5	5.5~2.5	<2.5
Rockburst potential criterion	China	$P_{rb} = (\sigma_{\theta}/\sigma_t)K_V$	<1.7	1.7~3.3	3.3~9.7	>9.7

Based upon the analysis of the energy accumulation mode of surrounding rock during TBM construction, Jiang et al. [26] proposed a rockburst energy criterion for TBM construction, expressed as follows in Equation (4):

$$\frac{U_e}{U^0} > 1 \quad (4)$$

where U_e is the stored elastic strain energy density of surrounding rock, and U^0 is the ultimate elastic strain energy density of surrounding rock; these can be expressed as the following Equations (5) and (6), respectively:

$$U_e = \frac{\sigma_1^2 + \sigma_2^2 + \sigma_3^2 - 2\nu(\sigma_1\sigma_2 + \sigma_1\sigma_3 + \sigma_2\sigma_3)}{2E} \quad (5)$$

$$U^0 = \frac{\sigma_c^3}{2E(\sigma_1 - \sigma_3)} \quad (6)$$

The above energy criterion is satisfied with the occurrence rockburst when the internal accumulation of the surrounding rock reaches the surface energy required to destroy the surrounding rock, at the critical value U^0 —that is, when the elastic strain energy U_e can be released. However, the above rockburst discriminatory index does not consider the critical influence of rock brittleness characteristics on the rockburst occurring in TBM construction in composite strata tunnels. For this reason, the above criterion is modified to propose a rockburst energy criterion C_r , taking into account these indicators, as follows below in Equation (7):

$$C_r = \frac{\sigma_c}{\sigma_t} \frac{U_e}{U_0} \quad (7)$$

where σ_c/σ_t indicates the brittleness coefficient of the rock formation, a characteristic characterization distinction for hard, brittle rocks. Some scholars [28] have used the brittleness coefficient for discrimination of rock explosion and classification of intensity class as follows: <15.0, no rock explosion; 15.0 to 18.0, minor rock explosion; 18.0 to 22.0, moderate rock explosion; >22.0, strong rock explosion.

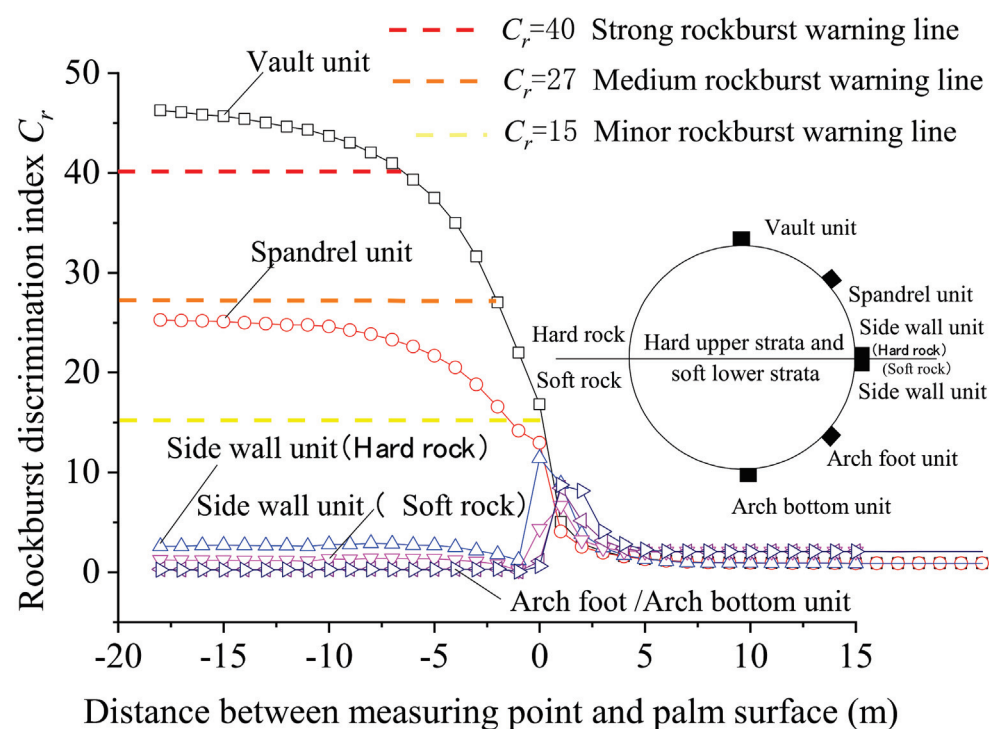
Combined with the existing rockburst criterion, the probability of boundary indexes of different factors reaching the maximum value was simultaneously small. For convenience of application, the boundary indexes and intensity classification of rockburst criterion C_r are shown in Table 4.

Table 4. Intensity classification of rockburst criterion C_r .

C_r Indicator Value	<15.0	15.0~27.0	27.0~40.0	>40.0
Rockburst intensity	No rockburst	Minor rockburst	Medium rockburst	Strong rockburst

5. Simulation Results of Rockburst Feature

The program used in the study was modified via FISH language with consideration of C_r (i.e., rockburst energy criterion) as the brittleness characteristics of rock strata, according to Equation (7), to obtain the distribution and evolution law of the C_r value of a tunnel's surrounding rock under TBM construction conditions, as shown in Figure 5. The positive value of the distance between the measuring point and the palm surface indicates the measuring point is positioned in front of the palm surface; the negative value indicates the measuring point is positioned behind the palm surface (Figure 5).

**Figure 5.** Evolution law of rockburst criterion C_r for a tunnel's surrounding rock during TBM excavation.

The conclusions that can be obtained from Figure 5 are as follows:

- (1) The hard rock part of the tunnel's upper part with composite strata had rockburst risk under TBM excavation; however, the soft rock part had no rockburst risk due to large deformation and release of most of the energy.
- (2) During excavation of the palm face up to a region situated before the measurement point, the C_r value of the surrounding rock changed according to the same law. In this situation, the palm face TBM excavation was maintained up to the monitoring section, and the C_r value of the surrounding rock at different locations in the hard rock part showed various change patterns, which were as follows: For surrounding rock at the sidewall of the tunnel, in the case of the face being excavated up to the monitoring section (the distance between the measuring point and the face was 0 m), the C_r value reached a maximum of 11.4, and then began to decrease and remained at a lower level. Throughout the entire TBM excavation process, the C_r value of the surrounding rock of the sidewall was always less than 15; thus, rockburst risk did not occur. Towards the surrounding rock at the vault and arch shoulder of the tunnel, in

the case of excavation of the arch face up to the monitoring section, the C_r value of the surrounding rock at the vault increased to 16.8, causing a slight risk of rockburst. For a related case, the C_r value of the surrounding rock at the arch shoulder was 12.9; therefore, the possibility of rockburst did not occur. When the palm face was pushed through the monitoring section of 2 m, the C_r value of the surrounding rock at the arch shoulder exhibited an increment to 16.6, with a slight risk of rock explosion. When the C_r value of the surrounding rock of the vault increased to 27, a medium risk of rockburst occurred. Afterward, the palm face was pushed through the monitoring section of 7 m; the C_r value of the surrounding rock's arch part exceeded 40 and gradually stabilized, and the risk of rockburst became strong. Similarly, the C_r value of the surrounding rock at the arch shoulder exhibited an increment to 24, and then gradually stabilized; the slight rockburst risk was continued.

According to Equation (7), the area with rockburst criterion C_r value exceeding 15 was regarded as the rockburst zone. The distribution characteristics of the rockburst zone during tunnel TBM construction were obtained, as shown in Figure 6.

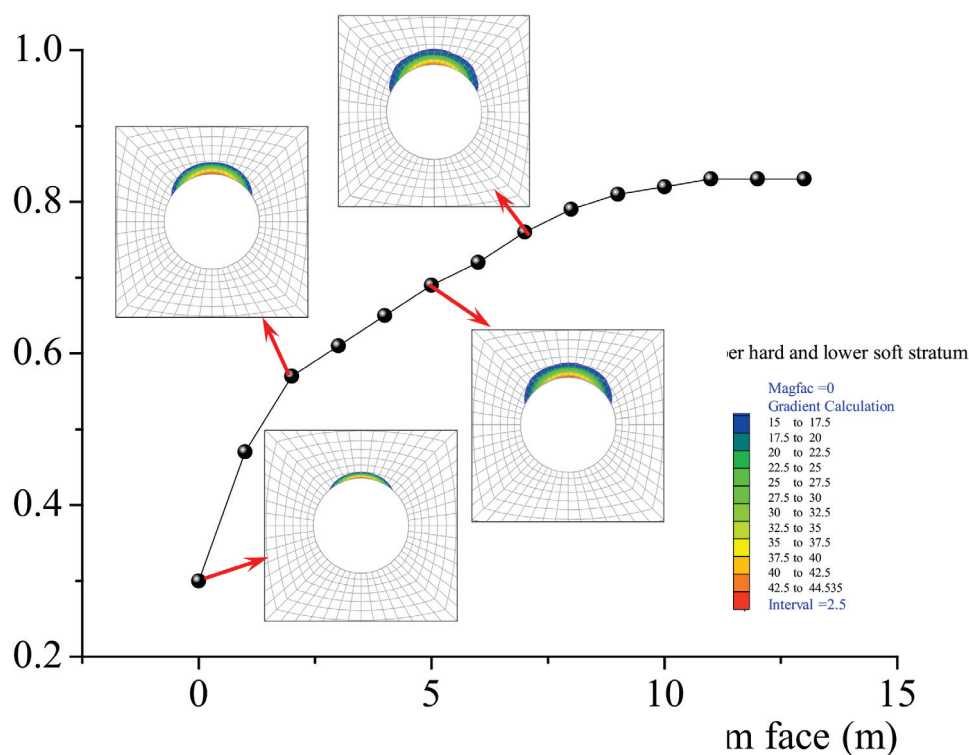


Figure 6. Evolution law of rockburst pits of surrounding rock during TBM construction.

As can be clearly seen from Figure 6, rockburst areas were mainly concentrated in hard rock; as TBM progressed, the rockburst area gradually propagated from the shallow part of the initial vault of the deep and arch shoulder position. In the section where the distance between surrounding rock and the face of the palm was about 10 m, the depth of the surrounding rockburst pits did not increase; the maximum development depth of the rockburst area was located at the vault, at about 0.88 m. The depth of the rockburst pits at both sides of the arch shoulders gradually decreased, and the burst pits generally had a “V” shape, which was consistent with a rockburst situation that occurred in actual TBM tunnel engineering [30–32]. Moreover, the occurrence of a rockburst may not end at once; the risk of another rockburst exists, unless corresponding measures are taken in the area where the rockburst has occurred.

6. Borehole Pressure Relief Prevention of Rockburst

The prevention and control of rockburst are essential in avoiding the brittle dynamic instability of surrounding rock, which can reduce the strength and brittleness of surrounding rock, improve the stress state of surrounding rock, and promote the favorable transformation of the failure mode of surrounding rock [33]. In this context, borehole pressure relief is a widely used active prevention and control measure for rockbursts in coal mines around the globe. Through manual drilling in the surrounding rock, the integrity of the surrounding rock is destroyed, and the mechanical properties of the surrounding rock are weakened. Thereby, the surrounding rock within a certain range around the borehole is plastically damaged, and elastic strain energy is released. Consequently, the high stress of surrounding rock is transferred; thus, rockburst risk is eliminated.

To analyze the treatment effect of borehole pressure relief on rockburst of a tunnel constructed by TBM in a composite stratum, a drilling pressure relief model of surrounding rock was established based on the abovementioned model. The variation rules of surrounding rock stress and rockburst criterion C_r values were simulated and analyzed before and after borehole pressure relief.

6.1. Pressure Relief Model of Borehole

The evaluations were performed using the automatic generation method of complex FLAC3D model based on the ANSYS platform [34]. In the first stage, the model was established, and meshes were divided in ANSYS software; afterward, the established model was imported into FLAC3D software by using an FLAC3D-ANSYS interface program. The drilling pressure relief model is shown in Figure 7. The size and boundary conditions of the model were the same as those of the model given in Figure 2. For the model's realization of the TBM excavation process, refer to Section 2.2 above, and refer to Table 2 for model material parameters.

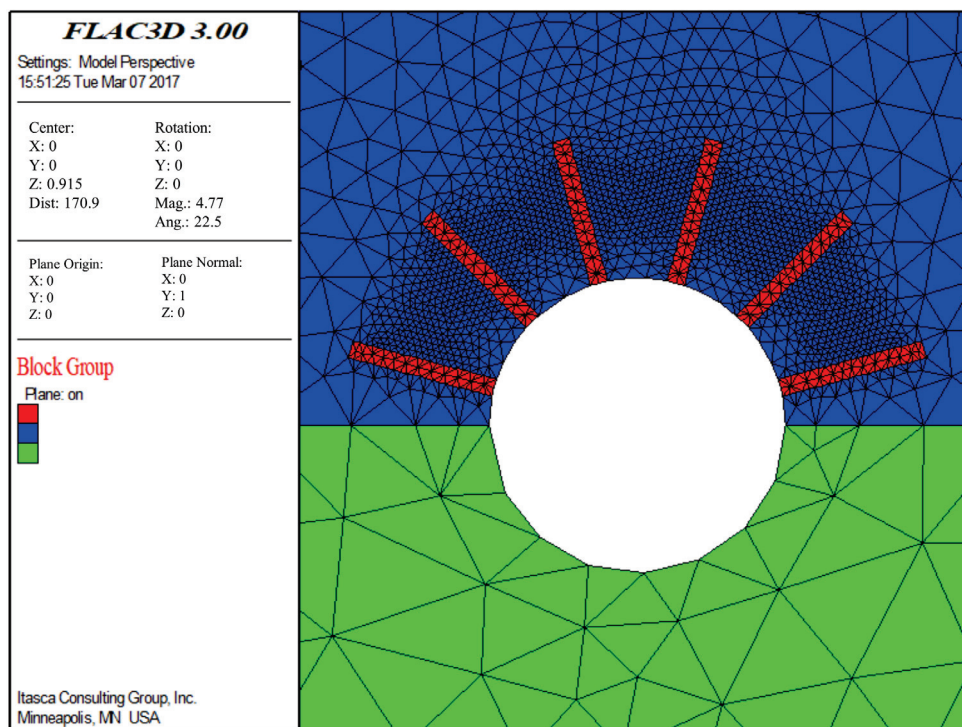


Figure 7. Numerical model of borehole pressure relief.

6.2. Rockburst Treatment Effect of Borehole Pressure Relief

The C_r value distribution of the rockburst criterion before and after the drilling pressure relief was obtained by simulation, according to Equation (7). The section 2 m behind

the palm face was analyzed, as shown in Figure 8. The variation rule of surrounding rock C_r values at the vault and arch shoulder of hard rock in composite strata was monitored before and after drilling pressure relief (Figure 9).

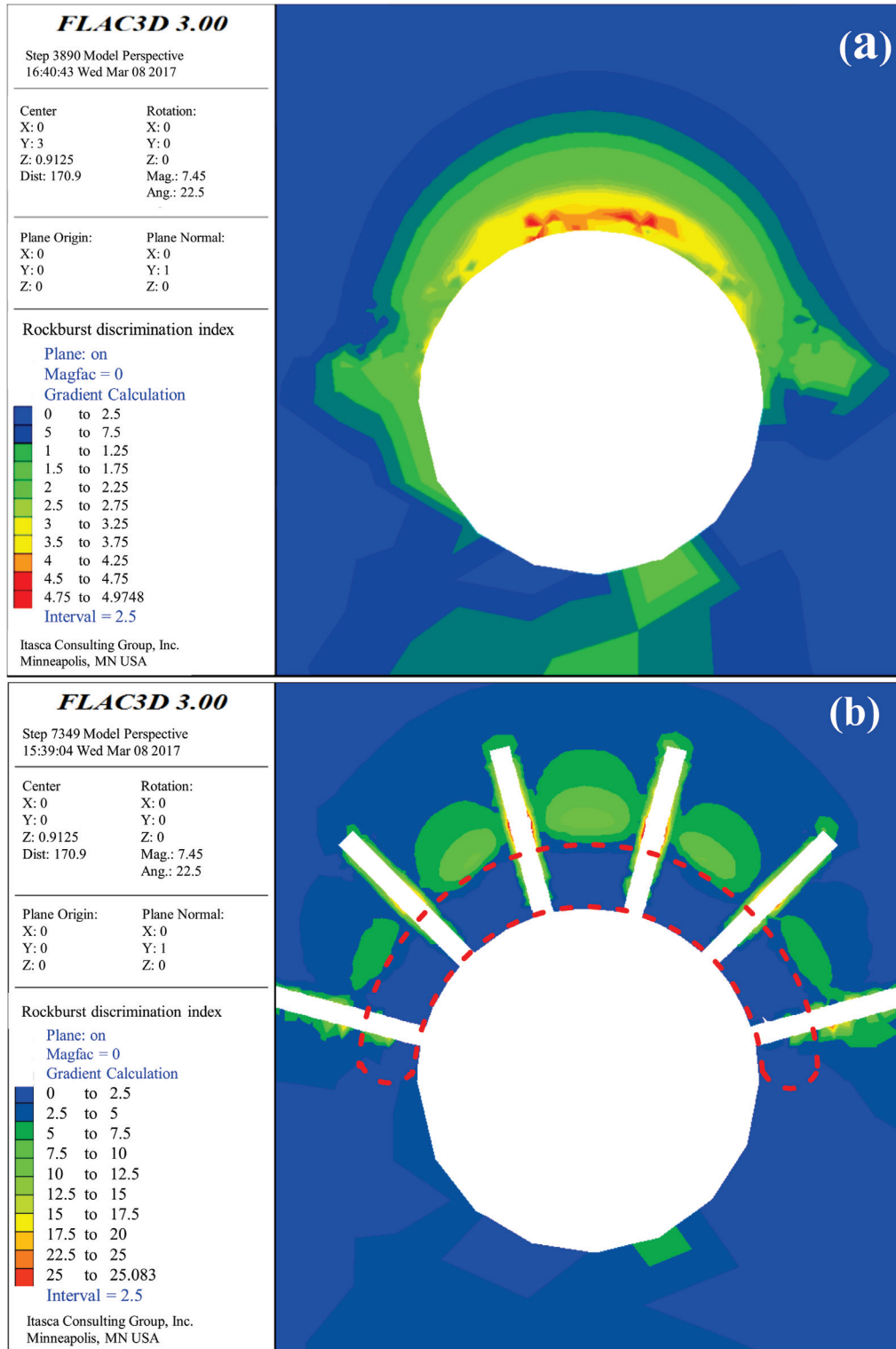


Figure 8. Distribution of rockburst criterion C_r of tunnel surrounding rock after borehole: (a) not drilled; (b) drilling pressure relief with hole diameter of 300 mm and hole depth of 2.5 m.

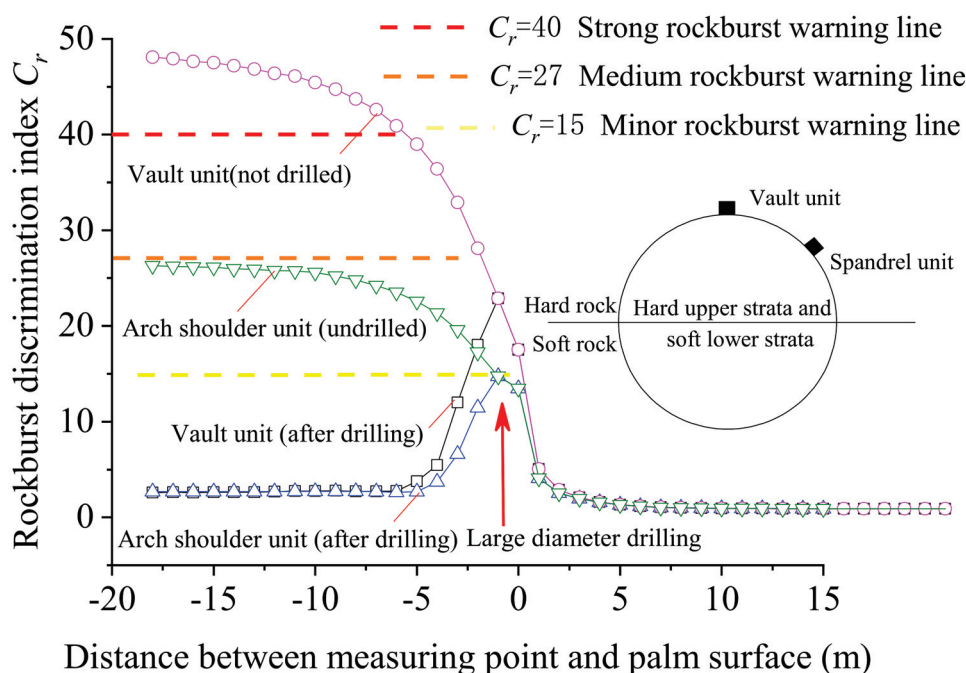


Figure 9. Variation law of rockburst criterion C_r of tunnel surrounding rock during TBM excavation after borehole.

The conclusions that can be obtained from Figures 8 and 9 are as follows:

- (1) The C_r value of the tunnel surrounding the rock vault and arch shoulder immediately dropped below 3.0 after the borehole pressure relief; furthermore, the C_r value no longer increased as the TBM progressed compared with the non-implemented borehole pressure relief.
- (2) After pressure relief, the maximum value of C_r in surrounding rock exhibited a decrement to about 25 and was mainly located around the hole wall at the section of 1.5 m deep in the borehole. The main reason for this situation was the elastic state of the rock mass around the deeper section of the hole wall with a certain degree of stress concentration.
- (3) After drilling and depressurizing the hard rock at the upper part of the tunnel that was 2 m behind the tunnel face, a certain depth of weakened zone was formed in the shallow region of surrounding rock; hence, the C_r value of the surrounding rock within the weakened zone decreased significantly. As shown in Figure 8b, in the area that the red dashed circle encloses, the C_r value of the surrounding rock in the weakened zone was considerably less than the rockburst threshold value of 15, and the rockburst risk was significantly reduced.

Figure 10 shows the distribution of the rockburst zone before and after pressure relief by drilling in the surrounding rock of the tunnel. Without drilling treatment, the rockburst zone of the tunnel vault and arch shoulder was more extensive, and the C_r value of surrounding rock at the vault could reach 49, indicating serious rockburst risk. After pressure relief, the shallow rockburst zone of the tunnel's surrounding rock disappeared completely. The deep rockburst zone of the tunnel became quite small, mainly distributed around the deeper region of the hole wall, as shown in Figure 10b. According to the numerical simulation results, for the TBM construction tunnel in composite strata, the pressure relief treatment of some hard rock in the surrounding rock can effectively reduce and eliminate rockburst danger.

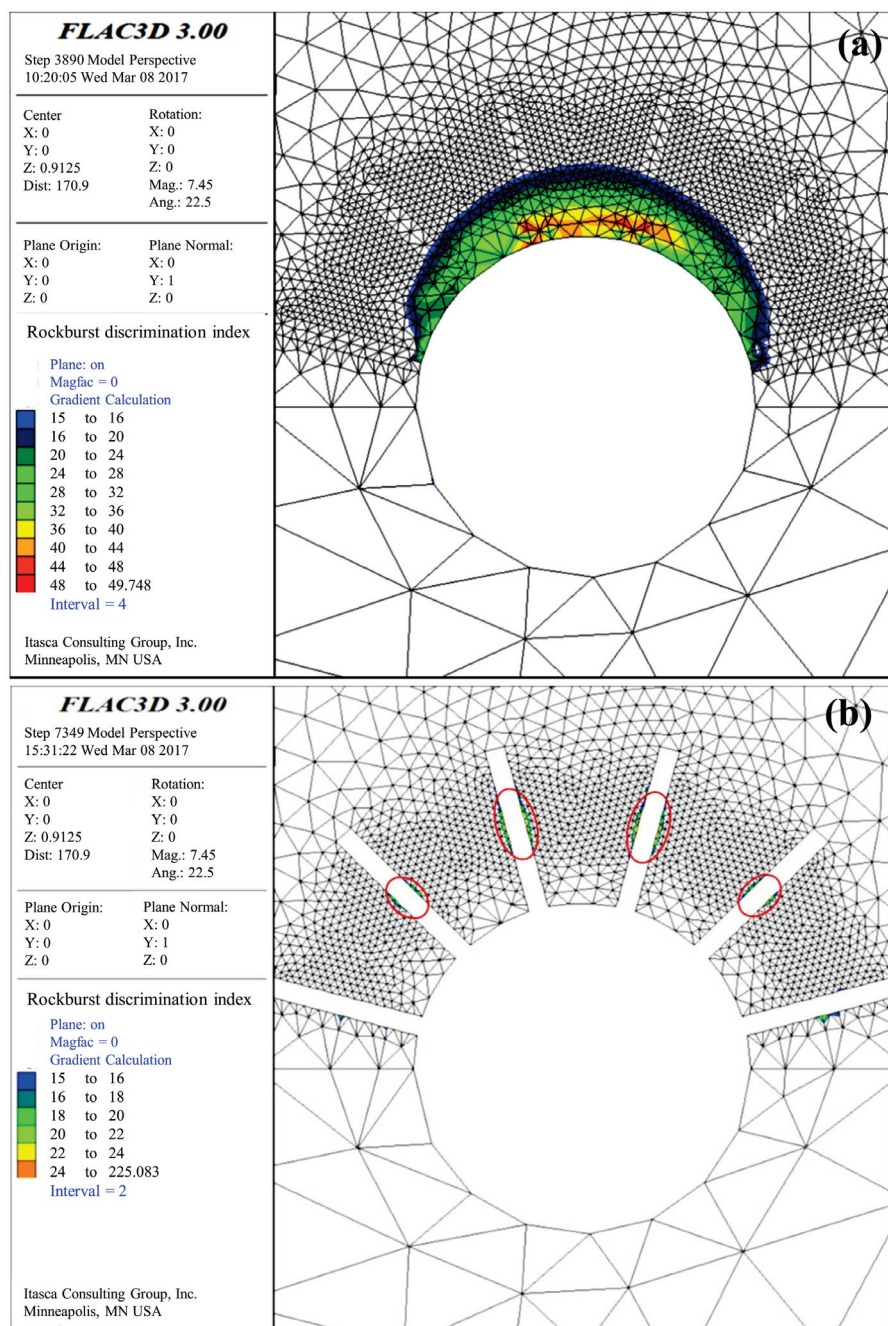


Figure 10. Distribution of rockburst area of tunnel surrounding rock after borehole: (a) not drilled; (b) drilling pressure relief (hole diameter 300 mm, hole depth 2.5 m).

6.3. Influence of Borehole Parameters on Rockburst Prevention and Control Effect

By changing the diameter and depth of the borehole pressure relief, the variation rule of the depth of the pressure-relief-weakening zone of the surrounding rock was simulated and analyzed, as shown in Figure 11.

Figure 11 shows that by increasing the borehole diameter from 200 mm to 400 mm, the depth of the weakened zone increased by nearly 200%; as the borehole depth increased from 2.5 m to 5 m, the depth of the weakened zone increased by 40%. The intersection point of the curve is the optimal parameter, that is, the drilling depth is 4 m, and the drilling diameter is 300 mm. Consequently, as borehole diameter and depth increased, the depth of the shallow weakening zone in the surrounding rock gradually increased; thus, the rockburst prevention and control mechanism were improved. Compared with the borehole

depth, the impact of the borehole diameter is more dominant on prevention and control of the rockburst.

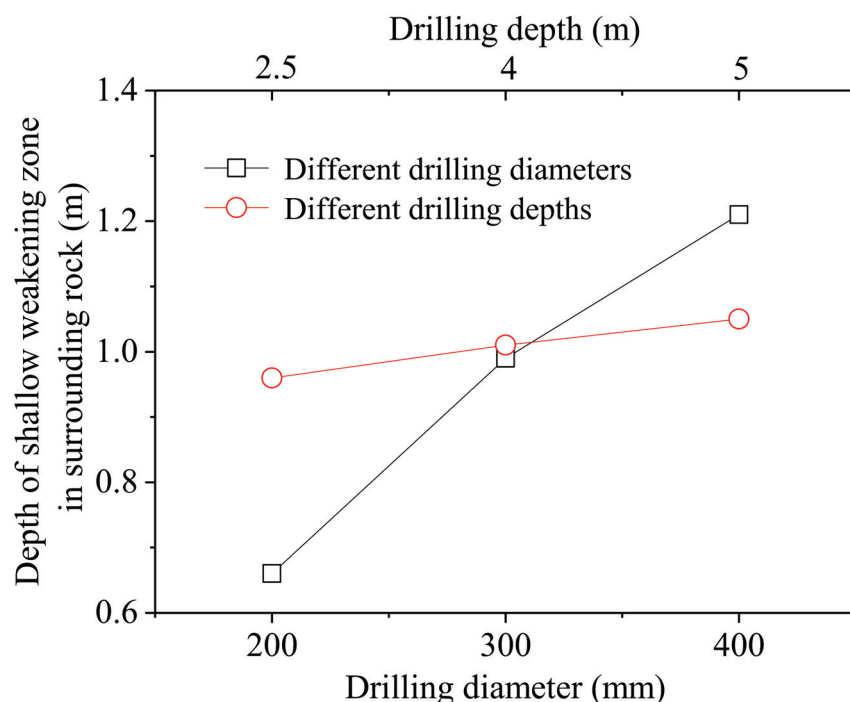


Figure 11. Depth variation of tunnel surrounding rock under different borehole parameters.

7. Conclusions

In this study, the characteristics and prevention of rockburst in a deep hard and soft compound stratum tunnel excavated using TBM were investigated. The specific conclusions of the research are as follows:

- (1) The rockburst risk of the compound stratum tunnel excavated by TBM mainly occurred in the upper-hard rock part, while in the soft rock part, rockburst risk did not occur. After the excavation of the tunnel, the first slight rockburst risk occurred in the hard rock area of the tunnel vault. Subsequently, the rockburst intensity at the vault and arch shoulder positions increased progressively, resulting in the occurrence of a severe rockburst risk in the vault position at 7 m behind the working face, which indicated the hysteresis of strong rockburst.
- (2) The rockburst area at the vault of the surrounding rock was the deepest, with a depth of about 0.88 m. The blast crater's depth progressively decreased on both sides of the arch shoulder, forming a "V" shape.
- (3) After the hard rock part of the composite strata, the tunnel was relieved by drilling. A certain depth of weakening zone was formed in the shallow part of the surrounding rock, which significantly reduced the rockburst risk of the surrounding rock. Compared with the depth of the borehole, the diameter of the borehole has a more significant impact on the rockburst prevention effect.

These research results provide guiding references for the prevention and control of similar rockburst disasters in underground engineering.

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Article

Energy Accumulation Characteristics and Induced Rockburst Mechanism of Roadway Surrounding Rock under Multiple Mining Disturbances: A Case Study

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Abstract: The source of energy release when rockburst occurs must be determined to understand the mechanisms underlying disaster formation and achieve accurate prevention and control. Although previous research has systematically investigated the energy source underlying rockburst from different perspectives, issues such as an unclear understanding of the energy accumulation state and inaccurate positioning of the energy release source remain to be resolved. In this study, the “1·17” major roof accident in the Danshuigou Mine was used as the background to evaluate and analyze the stress environment and energy accumulation characteristics of roadway surrounding rock under multiple mining disturbances, and the results showed that a super energy package occurs in the surrounding rock of the mining roadway. Subsequently, the evolution process of energy in this region and the mechanism of induced rockburst were elaborated. The results showed that the degree of stress concentration in the surrounding rock of the roadway will increase several times as the number of mining disturbances increases. Under the influence of multiple mining disturbances, the maximum principal stress peak of the surrounding rock of the roadway can reach 5–10 times the maximum principal stress value outside the mining-affected area. A large amount of elastic energy was accumulated in the rock surrounding the roadway, and super-high-density energy packages were formed locally. The maximum energy density value reached 50–185 times the value observed in areas outside the mining-affected zone. Thus, rockburst may be induced when the large amount of energy accumulated in the super energy package is suddenly and violently released; moreover, the degree of energy accumulation in the super energy package is likely closely related to the magnitude of rockburst. These results have important theoretical significance and application value for clarifying the mechanism of rockburst and improving the effectiveness of rockburst prediction and prevention.

Keywords: multiple mining disturbances; stress field; energy accumulation; super energy package; rockburst

1. Introduction

Rockburst is one of the main dynamic disasters threatening the safety and efficient production of coal mines [1,2]. Over the past five years, eight rockburst accidents have occurred in China, resulting in 48 deaths and more than 100 injuries. With the increase in mining scale, mining depth, and mining intensity, a more serious rockburst disaster development trend is expected to occur [3–5].

Rockburst is a dynamic failure phenomenon caused by the sudden release of elastic energy accumulated in coal and rock mass when the mechanical system of coal and rock reaches the strength limit [6–8]. Coal mining under different geological and mining technology conditions leads to diverse manifestations of rockburst, and although the specific mechanisms differ [9–11], a consensus has been reached in academic and engineering circles that rockburst is driven by energy [12–15].

The formation of rockburst is closely related to the elastic strain energy accumulated in the surrounding rock [16–18]. According to energy theory [19], rockburst occurs when the energy released by the coal-surrounding rock system is greater than the energy consumed when its mechanical equilibrium state is destroyed. This theory was previously used to explain the cause of rockburst from the perspective of energy conversion. Li et al. [20] first deduced the calculation formula of the energy distribution of an elastic foundation beam and analyzed the distribution law of energy accumulation and energy release before and after the fracture of the hard roof plate in front of the work face. Zhou et al. [21,22] studied the coupling effect of elliptical cavities and cracks on tunnel stability under static and dynamic loads. Guo et al. [23] applied elastoplastic mechanics to deduce the stress distribution formula and releasable strain energy concentration of roadway surrounding rock. Hao et al. [24] investigated a rockburst accident in the Chengshan Coal Mine by studying the influence of the upper goaf, roof strength, and excavation time on the energy distribution in front of the working face. Tan et al. [25] deduced the calculation method of kinetic energy generated in a coal body due to mining and proposed an evaluation index of the kinetic energy of rockburst risk. Chen et al. [26] explored the accumulation horizon of energy that causes rockburst in a coal–rock system, deduced the pre-peak energy distribution formula of coal–rock combinations, and studied the failure characteristics, mechanical properties, and energy accumulation law of the combination through tests. The integral volumetric measures of damage–dangerous volumes where stresses, strains, or energy surpass their limiting values are long and well known in the works on tribo-fatigue for systems simultaneously working in the conditions of fatigue, friction, wear, and thermal loads [27–29]. Sherbakov et al. [30] conducted computer modeling of stress–strain state and volumetric damageability in the neighborhood of a mine roadway and calculated volumetric damageability through the model of a deformed solid body with dangerous volume.

Previous research has systematically investigated the energy source underlying rockburst from different perspectives; however, the energy accumulation state is poorly understood, and a method of accurately locating the energy release source has not been developed. These issues remain to be resolved.

Based on the “1·17” major roof accident in the Danshuigou Mine, this study examined and analyzed the stress environment and energy accumulation characteristics of the roadway surrounding rock under the influence of multiple mining disturbances, and expounded the evolution process of the surrounding rock energy of the mining roadway and the mechanism of inducing rockburst. The results can be used as an important theoretical basis for determining the size and location of the energy source of rockburst. They also have certain engineering significance and reference value for further understanding the incubation and development process of rockburst and improving the monitoring and prevention efficiency of rockburst disasters.

2. Project Background

2.1. Project Overview

The Danshuigou Coal Mine is located in Xiaopingyi Township, north of Shuocheng District, Shuozhou City, Shanxi Province, China. The minable coal seams of the Danshuigou Coal Mine are coal seams 4⁻¹, 4⁻², 8, 9, and 11. The first-level coal seams 4⁻¹ and 4⁻² are jointly arranged, and the second-level coal seams 8, 9, and 11 are jointly arranged. The 4203 fully mechanized working face is located to the north of the main roadway in coal seam 4, the 4202 goaf is located to the east, and the lower part of the 4202 goaf corresponds to the 9202 goaf of coal seam 9. The strike length of working face 4203 is 1091 m, the dip length is 215 m, the average thickness of the coal seam is 5.68 m, and the dip angle of the coal seam is 2–14°, with an average of 5°. The working face adopts the long-wall coal mining method, the coal mining process is comprehensive mechanized mining, while the full-caving method is used to manage the roof. The direct roof and main roof of the working face are medium-coarse sandstone and mudstone, which present undeveloped

fissures. The thickness of the direct roof is 5.95 m, which belongs to the Class I stable roof category, and the thickness of the main roof is 3.97–9.04 m. The direct bottom and old bottom of the working face are mudstone and fine sandstone, respectively, which present undeveloped fissures. The thicknesses of the direct and old bottom are 3.91–9.35 m and 5.88 m, respectively.

The 4203 haulage roadway is excavated along the floor of coal seam 4 with a rectangular cross section. The roof support is a combination of support by bolt, anchor cable, steel belt, and metal diamond mesh. In the mining process of the 4203 working face, due to the coal roof of the haulage roadway, roof separation and roadway floor heave have occurred during mining. To ensure normal mining, some areas need to carry out roadway expansion operations, such as the expansion and repair of roadway sides, excavation and leveling of roadway floor. A single column with a steel cantilever beam is used to strengthen the support.

On 17 January 2017, a major roof accident occurred in the 4203 haulage roadway. The abnormal ground pressure phenomenon revealed by the accident was the first occurrence in this coal mine. The specific accident location and on-site excavation engineering layout are shown in Figure 1.

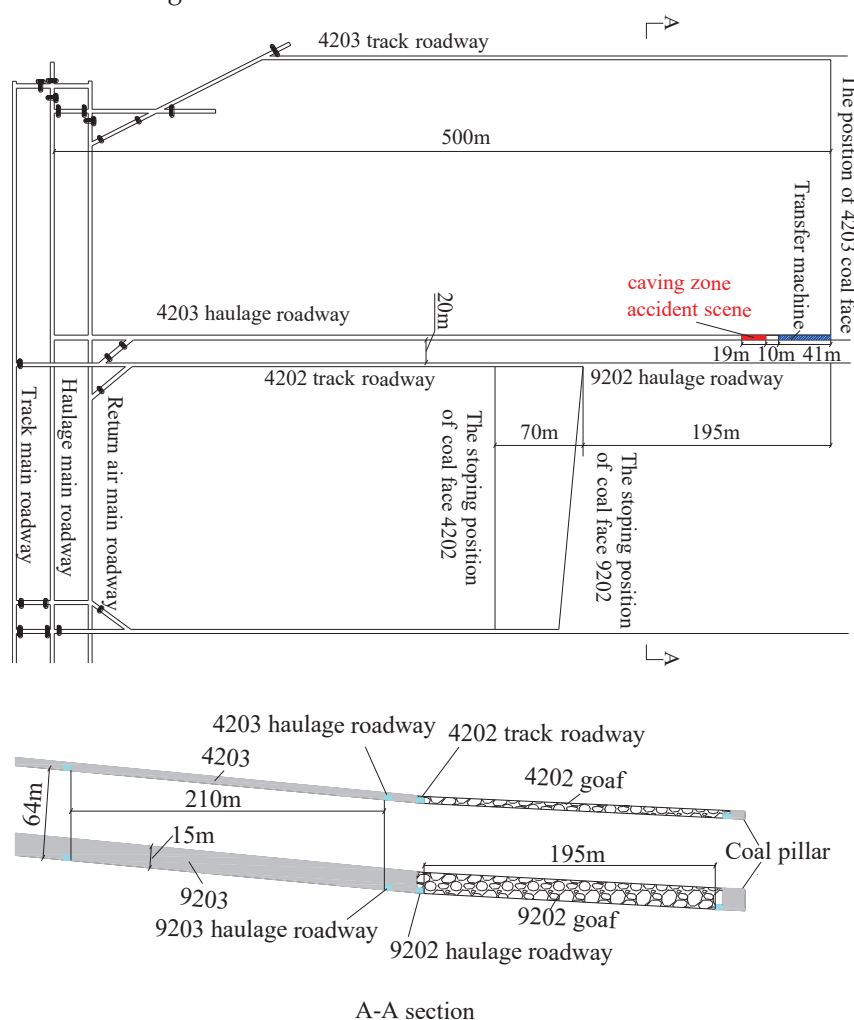


Figure 1. Schematic diagram of the accident location and surrounding mining layout of the 4203 working face haulage roadway.

2.2. Failure Characteristics of the Accident Roadway

After a major roof accident occurred in the 4203 haulage roadway, the roadway presented obvious sectional damage characteristics and typical dynamic damage characteristics as follows.

- (1) The accident roadway showed obvious characteristics of sectional damage. The 4203 haulage roadway is divided into five sections from the end of the working face outwards. As shown in Figure 1, the surrounding rock of the roadway within a range of 0–41 m in front of the working face was severely damaged (length 41 m). The transfer machine head was inclined, with one side close to the roof and the other side located 350 mm away from the roof, as shown in Figure 2. The roof with a range of 41–51 m in front of the working face was basically intact (length 10 m). The 51–70 m section in front of the working face presented evidence of the impact roof collapse (length 19 m), and its height of 2.0–2.5 m reached the direct roof. Most of the anchor rods on the side were exposed, the coal side was collapsed. The 70–200 m section in front of the working face was the damaged section outside the roof falling area, with 70–115 m being more severe. The roof and sides appeared to have a net pocket, and the floor bulge was severe. The normal rock pressure manifestation section was located 200 m away from the front of the working face, and the roadway surrounding rock was basically intact.
- (2) The accident roadway had typical dynamic damage characteristics. Some single hydraulic props were bent at 0–41 m in front of the working face. Within the 19-m-long roof fall and wall collapse section, more than 30 sets of roof bolts were broken, accounting for nearly 30% of the total number of bolts, most of them were broken without necking, and all 19 anchor cables were pulled apart, which is evidently different from the breaking and failure characteristics of bolts and anchor cables in ordinary roof fall accidents, as shown in Figure 3. According to the data provided by the Center of Seismological Network of Shanxi Province, the SHC Shenchu Station (~42 km away), PIG Pianguan Station (~83 km away), KEL Kelan Station (~80 km away), and L1410 Yuanping Station (~78 km away) all experienced obvious “earthquake anomalies” when the accident occurred.



Figure 2. Photo of the tilting of the end transfer machine on the working face.



Figure 3. Photo of the anchor bolt (cable) breaking.

3. Establishment of a Numerical Model

3.1. Numerical Calculation Model

Based on the actual data for the Danshuigou Mine, a numerical calculation model was developed (length $\times$ width $\times$ height = 1000 m $\times$ 530 m $\times$ 170 m), where the horizontal displacement of the x and y axis boundary was fixed, and the vertical displacement of the lower boundary of the z axis was fixed, as shown in Figure 4. A compensation load of 8 MPa was applied to the upper boundary to simulate the unit weight of the rock layer above the model, and the initial stress field determined from the field stress data was applied. The Mohr–Coulomb constitutive model and the Mohr–Coulomb failure criterion were applied for calculation. The physical and mechanical parameters of the rock mass are shown in Table 1.

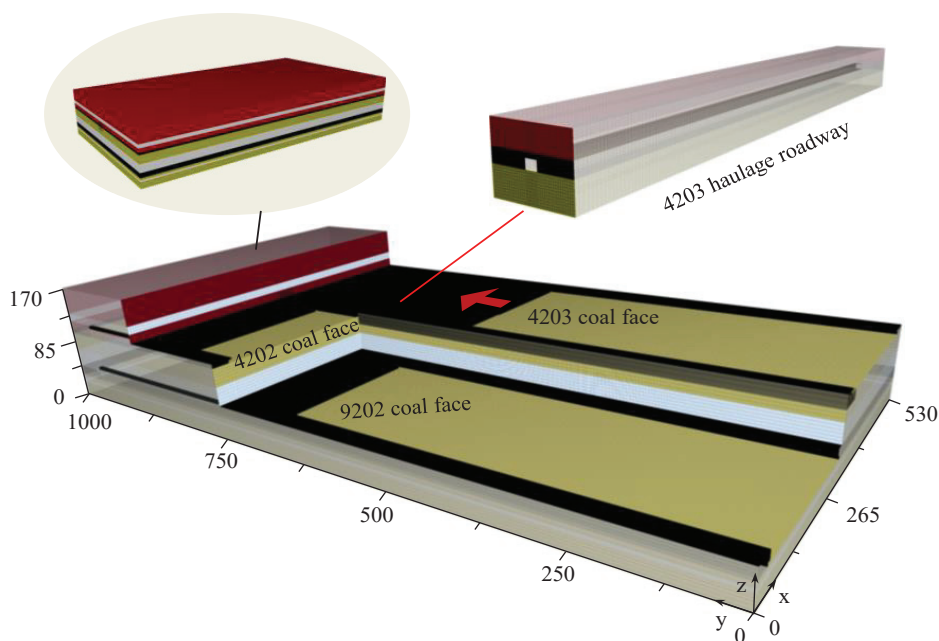


Figure 4. Numerical calculation model.

Table 1. Mechanical parameters of the stratified rock mass in the model.

Stratum	Density/(kg/m ³)	Bulk Modulus/GPa	Shear Modulus/GPa	Cohesion/MPa	Friction Angle/°
mudstone	2480	9.97	7.35	4.4	32
medium-coarse sandstone	2630	18.7	22.0	6.6	36
coal seam 4 ^{−2}	1380	4.91	2.01	3.7	32
fine sandstone	2540	2.7	1.6	4.0	35
mudstone	2480	9.97	7.35	4.6	32
coal seam 9	1380	4.91	2.01	3.8	32
mudstone	2480	9.97	7.35	4.2	32
fine sandstone	2540	2.7	1.6	4.0	35

For the research of the relationship among the stress environment and energy evolution observed in the surrounding rock of the 4203 haulage roadway and the impact roof fall accident, the width and height of the 4203 haulage roadway were set to 5 m $\times$ 4 m, respectively, in the numerical calculation model. Considering the influence of roadway excavation on the stress change of the surrounding rock, the section size of the surrounding rock of the 4203 haulage roadway was set to 35 m $\times$ 26 m (width $\times$ height); that is, the ratio of width direction to roadway width was 7, and the height direction was 6.5. This study assumed that the rock mass outside the mining surrounding rock was not affected by roadway excavation.

3.2. Calculation Schemes

Based on the mining layout of the working face when the 4203 haulage roadway roof fall accident occurred in the Danshuigou Coal Mine, four calculation schemes are proposed, as shown in Figure 5. Scheme I: considering the influence of single mining disturbance, only the 4203 working face will be mined to 490 m. Scheme II: considering the influence of double mining disturbances, the 4202 working face is first mined to the stopping position of 755 m, and then the 4203 working face is mined to 490 m. Scheme III: considering the influence of double mining disturbances, the 9202 working face is first mined to the stopping position of 685 m, and the 4203 working face is mined to 490 m. Scheme IV: considering the influence of triple mining disturbances, the 4202 and 9202 faces are mined to the stopping position of 755 m and 685 m, then the 4203 face is mined to 490 m.

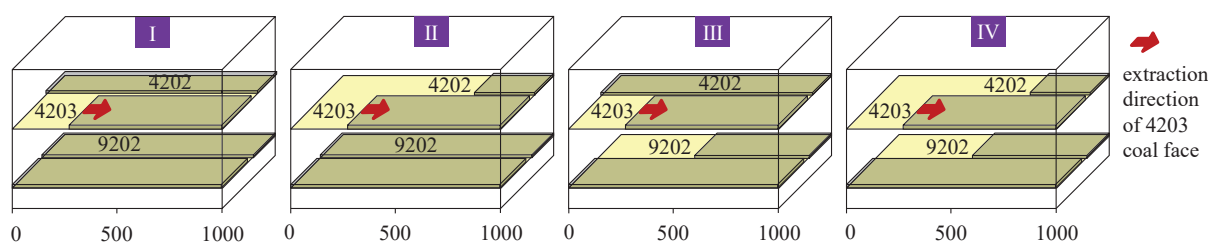


Figure 5. Mining layout schemes of the working face.

4. Stress Environment Characteristics of the Roadway Surrounding Rock

4.1. Vector Characteristics of the Mining Roadway Surrounding Rock Stress State

Figure 6 shows the calculation results of the vector distribution of the main stress field of the 4203 haulage roadway at 20 m in front of the working face under four different calculation schemes. According to the results, under the influence of double mining disturbances, the size of the main stress field significantly increased compared with that of a single working face; moreover, the direction of the main stress field changed significantly. However, under the influence of triple mining disturbances, the maximum principal stress field of the roadway surrounding rock increased several times compared with the influence of double mining disturbances, while that of the local area increased nearly 10 times. In addition, the uneven distribution of the principal stress field in the roadway section increased significantly, especially in the roof of the roadway, where the stress concentration was higher.

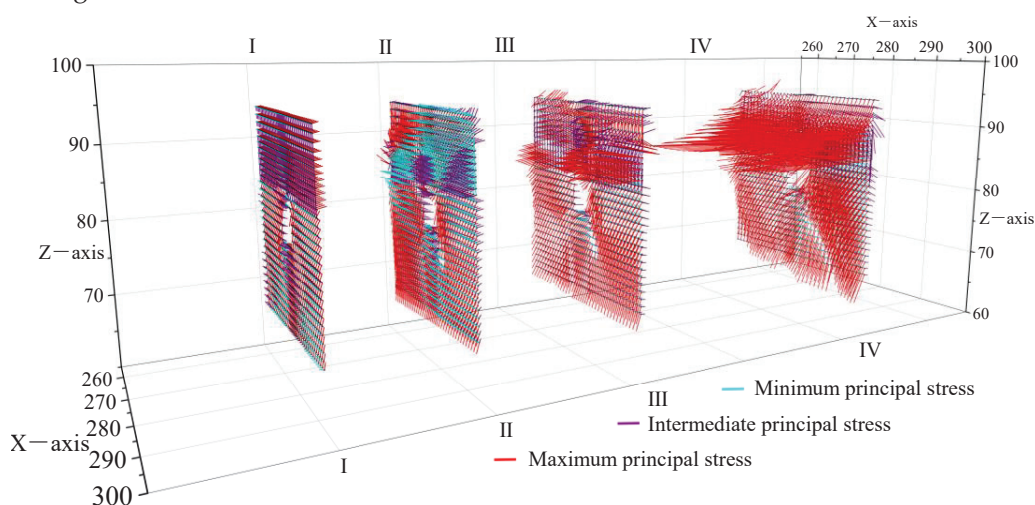


Figure 6. Vector distribution of the principal stress field of the roadway surrounding rock section under different mining schemes.

4.2. Distribution Characteristics of the Surrounding Rock Stress Field of Mining Roadway

According to the above calculation results, the maximum principal stress value in the area where the depth of the roof surrounding rock was 4 m was extracted along the axial direction of the roadway, and the center line of the roadway roof was used as a benchmark. The results are shown in Figure 7. Under the influence of mining disturbance at different working faces, the distribution of the maximum principal stress field was significantly different. Among them, the maximum principal stress under the influence of triple mining disturbances was far greater than the calculation results under the influence of single or double mining disturbances. From the peak value of the maximum principal stress field, the calculation result under the influence of triple mining disturbances was 3–5 times the peak value under the influence of double mining disturbances, 6.6 times the peak value under the influence of a single mining disturbance, and 7.3 times the peak value outside the area of a mining disturbance in front of the working face. Among them, a comparison of Schemes II and III showed that the 9202 working face had a greater impact on the peak value of the principal stress than the 4202 working face, and the stress increase range under the influence of triple mining disturbances could reach 300 m in front of the working face.

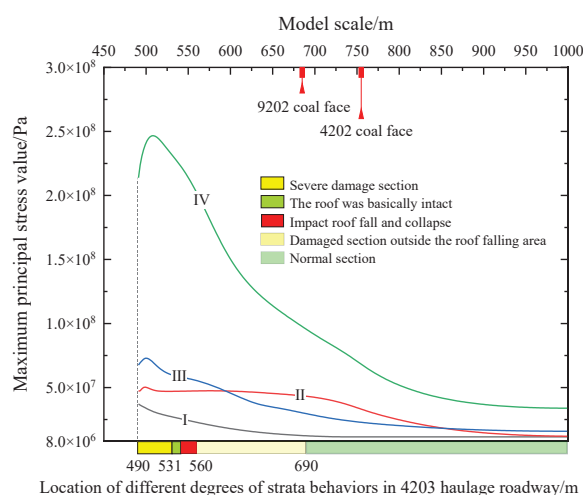


Figure 7. Stress distribution of the roadway surrounding rock in different mining schemes.

Figure 8 shows the calculation results based on Scheme IV when working face 4203 is located at different mining locations. According to the calculation results, in the mining process of working face 4203, the peak value of the maximum principal stress was 5–10 times that of outside the mining-affected area and centered on the stopping position of working face 9202. Before this position, the peak value of the maximum principal stress continuously increased, while after this position, the peak value gradually decreased, the distribution of the maximum principal stress no longer appeared in the stress rise area, and the peak value was directly located near the working face.

The results shown in Figures 7 and 8 were used to compare the damage area of the accident roadway with the distribution of the stress field. The mining stress field of the 4203 haulage roadway moved dynamically during the advance of the 4203 working face. With the approach to the stopping position of the 9202 working face, the stress concentration degree gradually increased. When the working face advanced to the accident location, the continuously migrating and increasing dynamic high stress acted on the 4203 haulage roadway, thereby inducing rockburst. The impact stress caused instantaneous destruction of the roof and floor of nearly 200 m of roadway and the two sides of the coal body, simultaneously broke a large number of bolts and cables in the local roadway, and led to local roof fall and wall collapse. Among them, the rockburst induced by multiple mining stresses was the direct cause of roof fall and wall collapse.

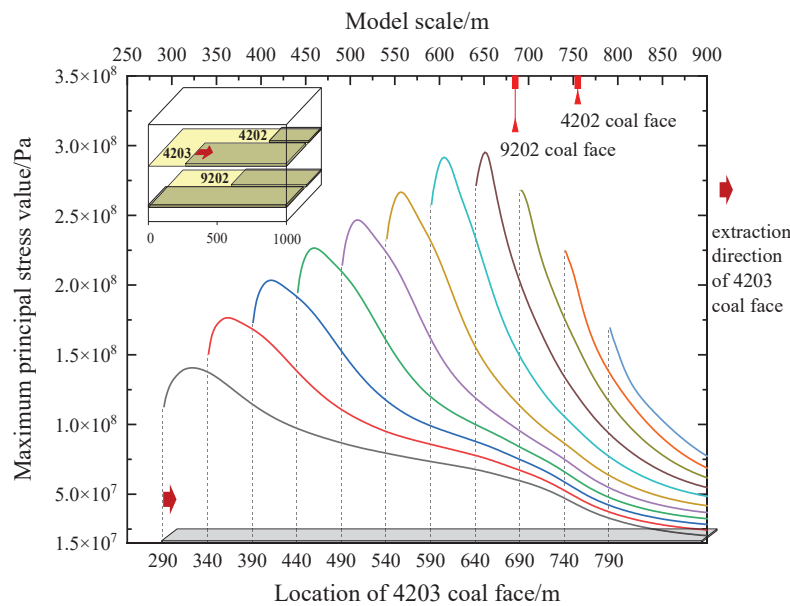


Figure 8. Stress distribution of the roadway surrounding rock in different mining positions.

5. Energy Accumulation Characteristics of the Roadway Surrounding Rock

When a rock mass undergoes elastic deformation due to external forces, energy accumulates inside the mass because of work by external forces. If the energy loss is not considered, then all work done by external forces acting slowly in the elastic range will be accumulated in the rock mass in the form of strain energy [31]. Therefore, when the surrounding rock of the roadway is affected by multiple mining disturbances, the rock mass under strong mining stress may accumulate huge amounts of elastic energy. In the elastic state, for the mining roadway surrounding rock in a certain area (expressed in Ω), the energy stored in any unit of its space can be expressed as follows [32,33]:

$$f(x, y, z) = \frac{1}{2E_i} \left[\sigma_{1i}^2 + \sigma_{2i}^2 + \sigma_{3i}^2 - 2\mu_i(\sigma_{1i}\sigma_{2i} + \sigma_{2i}\sigma_{3i} + \sigma_{1i}\sigma_{3i}) \right] * \Delta V_i \quad (1)$$

where E_i is the elastic modulus of the unit; σ_{1i} , σ_{2i} , and σ_{3i} are the maximum, intermediate, and minimum principal stresses of the unit, respectively; μ_i is the Poisson ratio of the unit; and ΔV_i is the unit volume.

As a certain plastic zone range usually forms around the roadway surrounding rock (denoted by Ω_p) and its micro unit is still considered to be in the elastic state, the elastic strain energy density of this area can be calculated by Equation (1) (at this time, the elastic modulus and Poisson ratio are the values under the plastic state). Under this state, the energy accumulated in the surrounding rock of the mining roadway can be expressed by Equation (2):

$$U = \iiint_{\Omega} f(x, y, z) dV = \iiint_{\Omega_e} f(x, y, z) dV_e + \iiint_{\Omega_p} f(x, y, z) dV_p \quad (2)$$

where Ω_e represents the rock mass area in an elastic state, i.e., $\Omega_e = \Omega - \Omega_p$.

5.1. Energy Accumulation Characteristics of the Roadway Surrounding Rock

According to the above, within the 4203 haulage roadway surrounding rock section along the axial direction of the roadway, the roadway surrounding rock with a length of 2 m was considered as the calculation unit to calculate the energy accumulated in the mining roadway surrounding rock within this range. Figure 9 shows the calculated energy accumulation distribution within the surrounding rock of the 4203 haulage roadway under the four different calculation schemes. According to the results, the energy accumulation

value of the roadway surrounding rock under the influence of triple mining disturbances was far greater than that under single or double mining disturbances. The peak value of energy accumulation of the roadway surrounding rock under the influence of triple mining disturbances can reach 1.53×10^9 J, which is equivalent to the energy of 364.3 kg TNT (4.2×10^6 J/kg), 4.6 and 3.1 times the peak value of accumulated energy under the influence of mining disturbances in the double working faces 4203 + 4202 and 4203 + 9202, respectively, 24 times the peak value of accumulated energy under the influence of mining disturbances in the single working face, and 72 times the value of accumulated energy outside the mining influence area in front of the working face. Under the influence of triple mining disturbances, the distribution characteristics of the accumulated energy in the roadway surrounding rock are noticeable and show a trend of initially increasing, then decreasing, and then tending to be flat. The range of the energy increase zone can reach 300 m away from the front of the working face. Under the influence of single or double mining disturbances, the zone of increasing energy distribution is small or disappears, and the range of the zone of increasing energy is relatively small.

The calculation results based on Scheme IV when working face 4203 is located at different mining locations are shown in Figure 10. According to the results, under the influence of triple mining disturbances, the accumulated energy of the surrounding rock of the 4203 haulage roadway was at a high level, and the peak value of the accumulated energy of the roadway surrounding rock was 15–73 times the accumulated energy value outside the mining influence area in front of the working face, which gradually increased while approaching the mining stopping position of the 9202 working face, the accumulated energy reached the maximum value at the 590 m position and then gradually decreased. Therefore, from the perspective of the evolution process of the accumulated energy of the roadway surrounding rock with the mining of the working face, the stopping position of the 9202 and 4202 working faces significantly affected the change in its peak value.

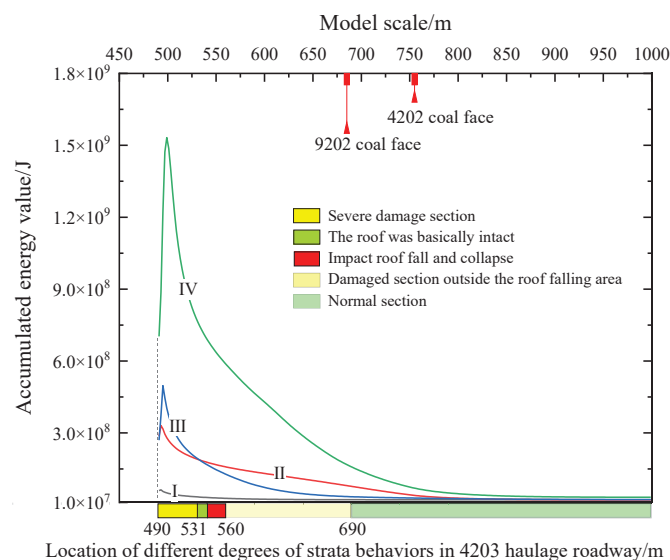


Figure 9. Distribution of accumulated energy of the roadway surrounding rock under different mining schemes.

According to the above analysis and the results shown in Figures 9 and 10, the energy value accumulated in the surrounding rock caused by mining in the 4203 haulage roadway moved and increased with the dynamic migration and gradual increase in the concentration of the mining stress field during the advance of the 4203 working face. When the working face advanced to the accident location, the large amount of energy accumulated in the roadway surrounding rock provided the energy basis for the occurrence of rockburst.

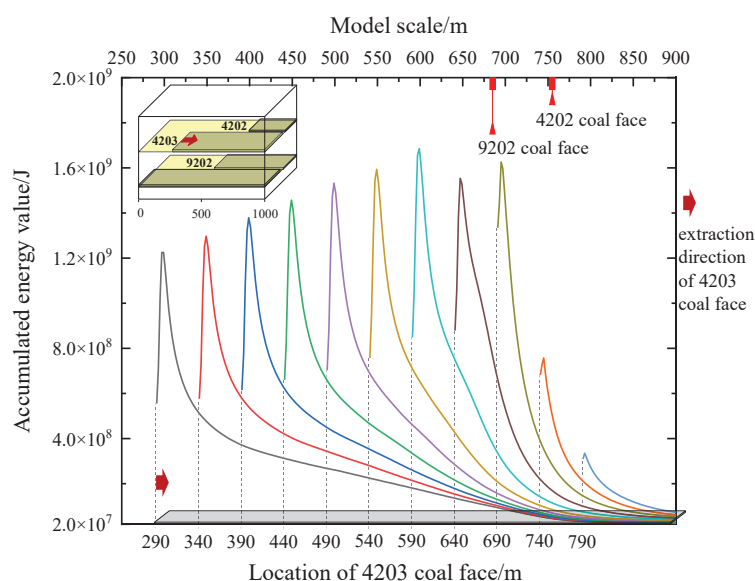


Figure 10. Distribution of accumulated energy of the roadway surrounding rock under different mining positions.

5.2. Super Energy Package of the Roadway Surrounding Rock and Its Evolution Characteristics

5.2.1. Super Energy Package and Its Connotation

The energy density distribution of the roadway section in Figure 6 was calculated, and the results are shown in Figure 11. Under strong triple mining stress, the stress concentration degree of the roadway surrounding rock increased several times. Due to the regional concentration of the mining stress distribution, a super-high-density energy accumulation area was locally formed in the roadway surrounding rock, and the maximum energy density value in the section can reach 35 times under the influence of the mining of a single working face.

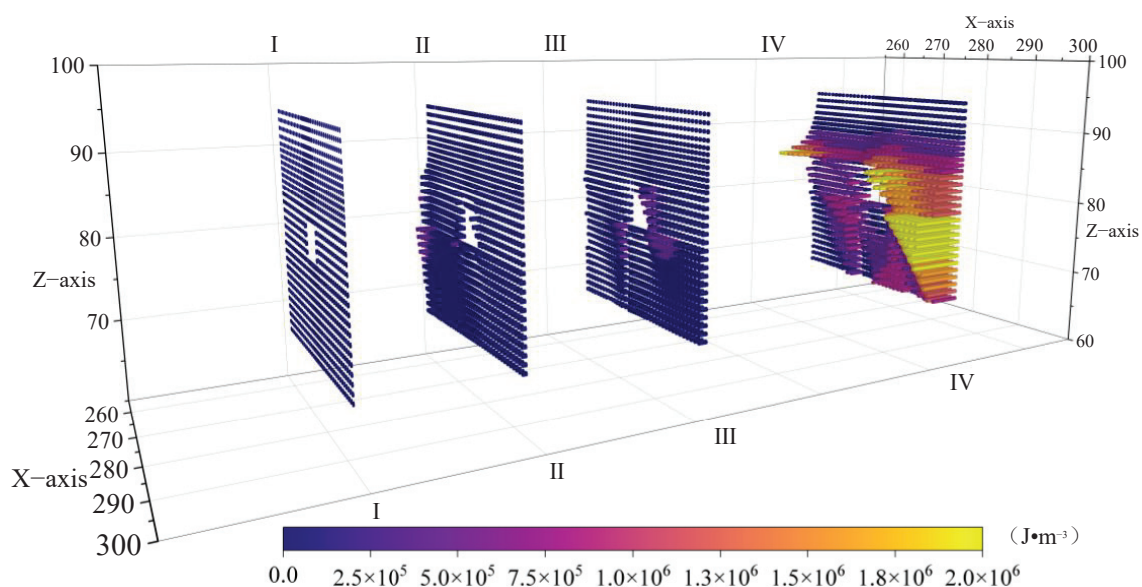


Figure 11. Comparison of the energy density distribution of the roadway surrounding rock section under different mining schemes.

When working face 4203 was located in different mining positions, according to the energy accumulation area of the roadway surrounding rock, along the axial direction of the roadway, i.e., the Y direction, take the section at $X = 288$ m, and the energy density distribu-

tion was calculated, as shown in Figure 12. In the mining process of the 4203 working face, under continuous increases of mining stress concentration, the energy accumulated in the 4203 haulage roadway increased, and the distribution of the energy field also underwent cumulative adjustment. Simultaneously, combined with the calculation results in Figure 11, in the mining process of working face 4203, a super-high-density energy package occurred in the roadway surrounding rock, which is termed as the super energy package in this study. When a certain energy density value is determined as the boundary of the super energy package, the energy density value of the rock mass within the boundary of the super energy package increases in multiples, and a region containing the maximum energy density is formed in the center, known as the energy nucleus, the center of the energy nucleus is the region where the maximum energy density is located in the super energy package. The scope, shape and internal stored energy of the super energy package changes significantly with the change of the position of the 4203 working face.

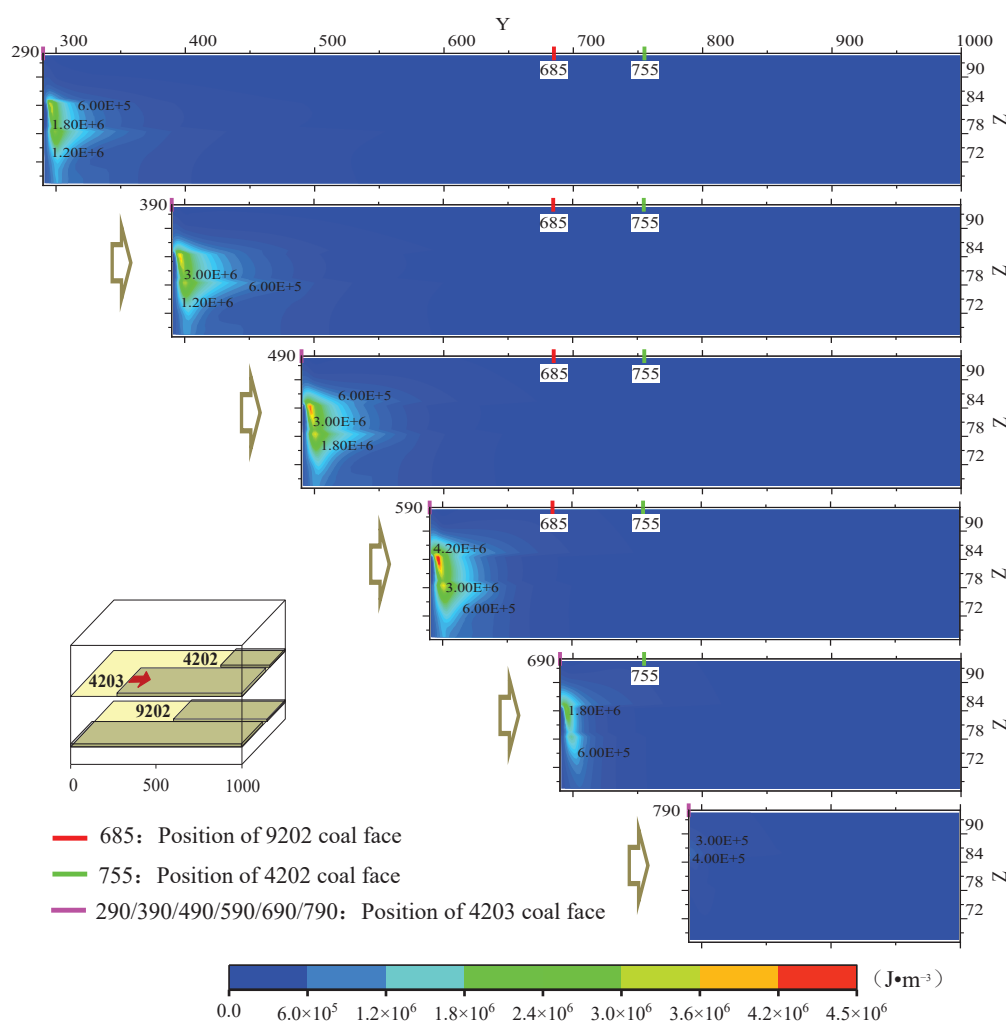


Figure 12. Energy density distribution of the 4203 transport roadway surrounding rock when the 4203 working face is located at different mining positions.

5.2.2. Evolution Characteristics of Super Energy Package

To characterize the evolution characteristics of the super energy package with the change of the mining position of the 4203 working face, the maximum energy density value was extracted in the surrounding rock section of the 4203 haulage roadway, that is, the maximum energy density within a certain section of the surrounding rock of the roadway, and the maximum energy density distribution curve of the surrounding rock in different sections was formed along the axial direction of the roadway, as shown in Figure 13. Before the stopping

position of the 9202 working face, the distribution of the maximum energy density presented a “double peak” feature of “increase → decrease → increase → decrease → flat.” At this time, the maximum energy density in the super energy package was the maximum value of the maximum energy density distribution curve. After the stopping position of the 9202 working face, it transformed into a “single peak” that followed the form “increase → decrease → flat”.

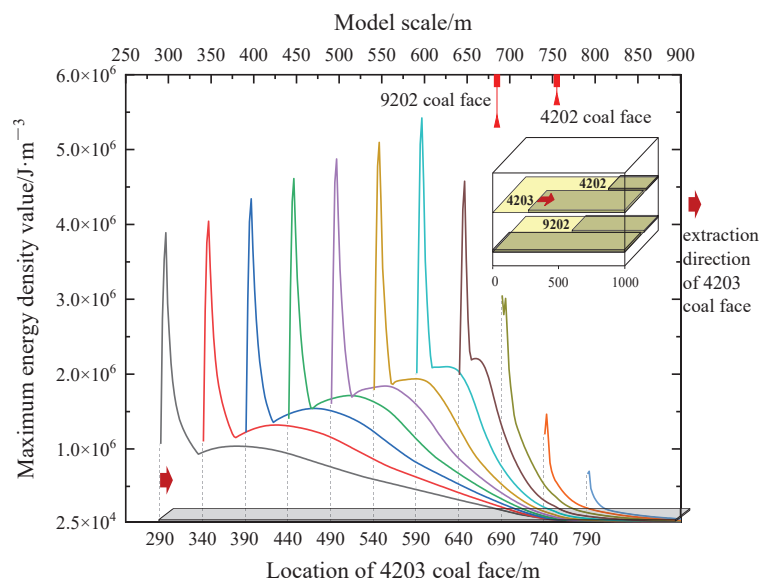


Figure 13. Distribution curve of the maximum energy density.

Figure 14 shows the calculation results of the maximum energy density distribution eigenvalue. Here, nuclear is the maximum energy density in the super energy package, and the center position of nuclear is the distance between the maximum energy density in the super energy package and the working face. With the continuous approaching of the working face 4203 and the stopping position of working face 9202, the maximum energy density in the super energy package gradually increased and reached the maximum values at 590 m. In this process, the central part of nuclear stabilizes at 7 m in front of the working face. Afterwards, as the 4203 working face continued to advance, the maximum energy density in the super energy package began to gradually decrease, and the distance between the center of the energy nuclear and the working face was also gradually approaching.

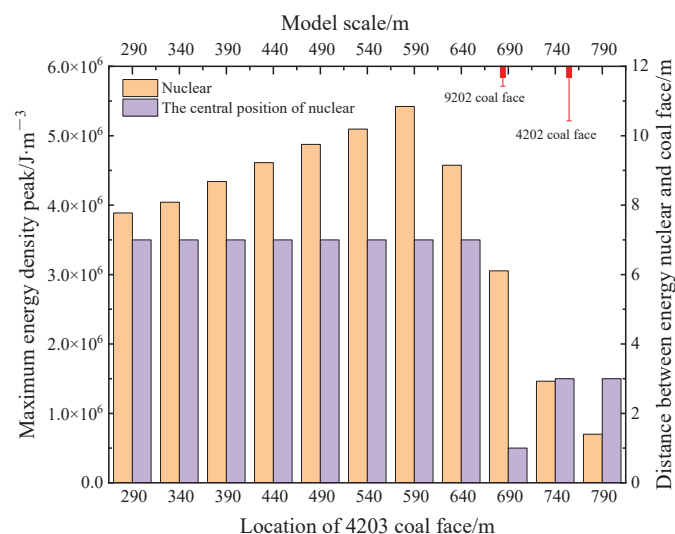


Figure 14. Distribution characteristic values of the maximum energy density.

Under the influence of triple mining disturbances, the maximum energy density value of the super energy package in the surrounding rock of the 4203 haulage roadway was at a high level. The accumulation degree of energy in the super energy package changes significantly with the position of the working face, and the maximum energy density value inside the super energy package can reach values 50–185 times that outside the mining-affected area. Considering the mining of the working face to be 490 m (i.e., the position of the 4203 working face when the accident occurred) as an example, the maximum value of the energy density in the energy package was $4.87 \times 10^6 \text{ J/m}^3$, which was 166 times that of the maximum energy density outside the mining-affected area in front of the working face. When the huge energy accumulated in the super energy package reached a certain limit, it may become the main energy source underlying rockburst catastrophe accidents.

6. Discussion

In the process of the deformation and destruction of rocks, energy input, accumulation, dissipation, and release occurs [34,35]. Consider the mining roadway within a certain range in front of the coal face as a relatively closed system (4203 haulage roadway with a certain range, as mentioned earlier), that is, the surrounding rock system of a mining roadway. The surrounding rock system of the mining roadway likely also experiences energy input, energy accumulation, energy dissipation, and energy release during working face mining. The spatiotemporal evolution of these four aspects of energy is closely related to rockburst disasters induced by energy.

6.1. Energy Evolution Process of the Surrounding Rock of Mining Roadway

(1) Energy input

External energy inputs mainly refer to the work done by external forces. For the surrounding rock system of the mining roadway, the external force can be considered mining stress that is constantly moving with the advance of the working face during the mining process. Increased mining stress acts on the surrounding rock system of the mining roadway, and the external force impacts the system and also inputs external energy. In addition, the external force includes the dynamic load caused by rock mass fracture of the outer roof of the surrounding rock system of the mining roadway, fault activation, earthquakes, and other mining engineering disturbances.

(2) Energy accumulation

For the surrounding rock system of a mining roadway, part of the external input energy is accumulated in the surrounding rock of the roadway in the form of elastic deformation energy, which is reversible and can be released when the increased mining stress is unloaded. When energy is input from an external source, the surrounding rock system of the mining roadway accumulates energy continuously, and the distribution of the energy field forms a super-high-density energy package due to the local adjustment of the stress state of the mining surrounding rock. Under different mining conditions, the degree of energy accumulation in the super energy package varies greatly. When the huge energy accumulated in the super energy package reaches a certain limit, it may become the main energy source of rockburst catastrophe accidents.

(3) Energy dissipation

When an external force acts to input energy into the surrounding rock system of the mining roadway, if the stress in the surrounding rock exceeds its elastic limit, the surrounding rock will produce plastic deformation. Due to the irreversibility of plastic deformation, this is a process of energy dissipation. Simultaneously, the energy input from the outside not only leads to the further development of initial damage (cracks and holes) in the roadway surrounding rock, but also produces new cracks and holes in the surrounding rock; that is, part of the energy is dissipated in the form of irreversible damage energy. As the energy input increases, the amount of energy transformed into plastic deformation energy and damage energy becomes larger. In the surrounding rock system of the mining

roadway, it is specifically manifested in the expansion of the plastic zone of the surrounding rock and the increase of the degree of fragmentation of surrounding rock.

(4) Energy release

Energy release mainly refers to the energy released by the surrounding rock system of the mining roadway to the outside world (mainly refers to the roadway space and the rock mass outside the system). The intensity of energy release in unit time can be graded according to the magnitude of the earthquake. When the energy accumulated in the surrounding rock system is released evenly and slowly, it is mainly manifested in the form of general ground pressure manifestations, such as roof subsidence, wall bulge, and floor heave. When the energy accumulated in the surrounding rock system is released violently in a short time, it is mainly manifested in the form of dynamic destruction of the surrounding rock, such as rock mass ejection, surrounding rock vibration, and roadway roof fall, simultaneously, accompanied by the sound of surrounding rock fracture, the intensity of the released energy is positively related to its destructiveness [36,37], which can severely destroy mechanical equipment and support components in the roadway space and cause casualties.

6.2. Mechanism of Rockburst Induced by Energy Accumulation in the Surrounding Rock of Mining Roadway

In the mining process of the working face, when dynamic changes of the surrounding rock stress field in the mining space occur and the stress field of the surrounding rock system of the mining roadway is in a specific stress state, the corresponding energy field has a specific energy state corresponding to the state [38]. Under certain mining conditions, the energy of the surrounding rock system of the mining roadway is continuously accumulated and forms a super energy package with a very high energy density value. Subsequently, the sudden and violent release of the huge energy accumulated in the super energy package can induce rockburst. In this process, the formation of the super energy package and the degree of its energy accumulation are important factors that induce rockburst. The huge energy accumulated in the super energy package will become the main energy source of the dynamic disaster accident. As shown in the previous research results, under the influence of triple mining disturbances, the mining surrounding rock system in a certain area of the 4203 haulage roadway experienced four processes of energy input, accumulation, dissipation, and release during the advancing process of the working face. With the continuous input of external energy, the energy in the mining surrounding rock system continues to accumulate, and the energy accumulation degree in the super energy package also continues to increase. When the working face is pushed to the accident location, under the action of a trigger event, the huge energy accumulated in the super energy package will be released violently in a short time, thus triggering an impact roof fall accident.

The results of this study indicate that further theoretical exploration is required to determine how to prevent energy in the rock mass from accumulating at a local height, release the accumulated energy smoothly, and accurately predict and monitor the location and degree of rockburst during coal mining operations.

7. Conclusions

- (1) The stress environment characteristics of the roadway surrounding rock under the influence of multiple mining disturbances were studied and analyzed. The degree of stress concentration in the surrounding rock of the roadway will increase several times as the number of mining disturbances increases. The stress concentration degree of the roadway roof surrounding rock and the uneven degree of the stress field distribution are relatively higher, and the peak maximum principal stress can reach 5–10 times that outside the mining-affected area. During the mining process of the working face, as the distance from the stopping position of the adjacent working face decreases, the stress concentration degree of the roadway surrounding rock continues to increase during its dynamic migration.

- (2) Under the influence of multiple mining disturbances, a large amount of elastic energy is accumulated in the roadway surrounding rock and a super-high-density energy package is formed in the roadway surrounding rock. The maximum energy density value in the super energy package can reach 50–185 times that of outside the mining-affected area. The degree of energy accumulation in the super energy package will change significantly with the change of the location of the working face. When the huge energy accumulated in the super energy package reaches a certain limit, it may become the main energy source of the dynamic disaster accident. This relationship can be used as an important theoretical basis for determining the size and location of the energy source of rockburst.
- (3) The evolution process of the surrounding rock energy of a mining roadway and the mechanism of induced rockburst are expounded. Under the influence of multiple mining disturbances, the energy of the surrounding rock system of the mining roadway is continuously accumulated and forms a super energy package with a very high energy density value. When the huge energy accumulated in the super energy package is suddenly and violently released, it can cause dynamic destruction of the surrounding rock, such as rock mass ejection, surrounding rock vibration, and roadway roof fall, which are accompanied by the sound of surrounding rock fracture. The degree of energy accumulation in the super energy package is closely related to the magnitude of rockburst.

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Article

Investigation into Occurrence Mechanism of Rock Burst Induced by Water Drainage in Deep Mines

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Abstract: Confined aquifers widely exist in the strata of the Ordos mining area. Water drainage before mining is an effective measure to avoid water inrush disasters caused by the connectivity between mining-induced roof fractures and aquifers. However, rock burst disasters occur frequently in the mining process of many water drainage working faces. The statistics show that the surrounding rock of nine water drainage working faces in five mines has suffered different degrees of rock burst, which seriously restricts the safe, efficient production and sustainable development of ten-million-ton modern mines in China. Based on this, taking the 22,106 longwall working face (LW22106) of the Shilawusu Mine, Ordos, as the engineering background, this paper investigates the occurrence mechanism of water drainage on rock burst in the roof water-rich area by using theoretical analysis, similar material simulation, microseismic measurement and other methods. The main conclusions are as follows: (1) After the drainage of the water-rich area, the pressure relief zone, pressurized zone and pressure stabilization zone are formed in sequence from the center to both sides. The width of the pressure relief zone is consistent with that of the water-rich area, and the width of the pressurized zone is about 35 m on one side. (2) When the mining is passing the pressurized zone, the coal rock mass is under the joint influence of gravity stress, mining disturbance stress and drainage transfer stress. The superimposed stress generally exceeds 2.5 times the uniaxial compressive strength (UCS) of the coal rock mass, and the maximum reaches 3.24 times, far exceeding the critical value of rock burst (1.5 times UCS), which is the main reason to induce rock burst. (3) The dynamic change in the rock burst risk areas before and after drainage in water-rich area no. 4 has been predicted, and the number of risk areas increases from 4 before drainage to 13 after drainage. Since the stress superposition effect differs due to different mining speeds, it is proposed that the dynamic regulation of mining speed in the front and at the back of the drainage area is an effective and efficient method for rock burst prevention and control. The research results will provide a theoretical basis and technical support for the prevention and control in the roof water-rich area of deep mines.

Keywords: deep mining; water drainage; rock burst; stress distribution; classification of risk areas

1. Introduction

With the gradual depletion of shallow coal resources in China, coal resource exploitation presents a deep mining tendency, and deep mining is increasingly becoming normal [1,2]. After coal mines step into the deep mining stage, geological conditions and stope structures become much more complex, the occurrence frequency and damage degree of dynamic disasters remarkably increase, and more and more mines will be faced with the threat of dynamic disasters such as rock burst [3–6]. Since 2010, 25 rock burst accidents

have occurred in China, resulting in 119 deaths and 108 injuries. Rock burst has become a key scientific problem and a worldwide problem that needs to be solved urgently [7–11].

Ordos coalfield, as an important coal base in China, stretches across Shaanxi, Gansu, Ningxia, Inner Mongolia and Shanxi provinces, as shown in Figure 1. Group Yan'an and Group Zhiluo artesian aquifers widely exist in the coal seam roof, and the large water-rich areas serve as the direct water source for coal seam mining [12]. In the mining process on the working face below the roof water-rich areas, roof disasters such as water inrush and sand bursting may be faced [13]. In order to avoid the roof water inrush and reduce the preparation time on the working face, the roof water drainage should be carried out during excavation on the working face. However, the drainage in the water-rich area causes the stress redistribution of the coal seam and roof [14], forming a stress concentration area and increasing the possibility of rock burst. In recent years, rock burst has appeared several times in the mining process of some roof water drainage working faces, which seriously restricts the safe and efficient production and coal supply. Therefore, the investigation into the influence mechanism of roof water drainage on the stress distribution of overlying rock is of great significance to reveal the occurrence mechanism of rock burst on the water drainage working face in the Ordos mining area, take measures for rock burst prevention and control and ensure the safety of the working face.

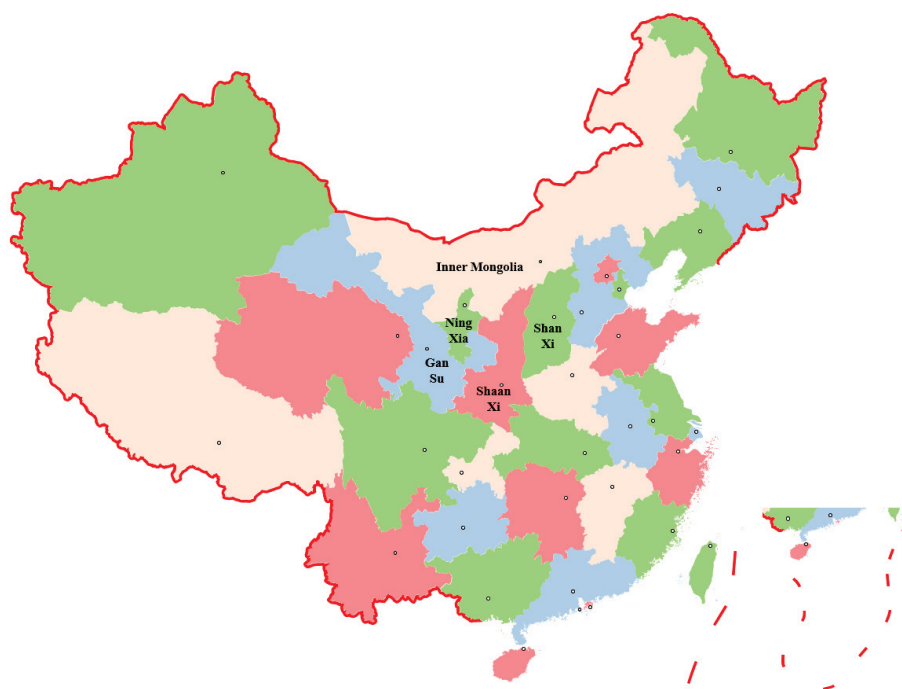


Figure 1. Location map of Ordos coal base in China.

Field practice shows that rock burst is affected by many factors, including geological and mining factors. Researchers have conducted extensive studies on the influencing factors of rock burst and made rich scientific research achievements. Jiang et al. [2,15] believed that rock burst, affected by various inducing factors, was a nonlinear dynamic process in the deformation of the coal rock mass system under different geological conditions and mining environments. During the process, energy stored in a stable state was transformed into energy released in an unstable state. They also pointed out that the essence of rock burst was the stress-induced sudden instability and failure of the coal rock mass. Dou et al. [16,17] and Cai et al. [18] held that rock burst was closely related to the physical and mechanical properties of coal rock, as well as the static and dynamic loads on coal rock. Based on this, they further investigated the energy and stress conditions of rock burst induced by dynamic and static loads and clarified the mechanism and two types of rock burst induced by static and dynamic loads: high static load type and strong dynamic

load type. Qi et al. [19] explained many determining factors for the occurrence of rock burst and put forward the idea of multi-scale and multi-source prevention and control of dynamic disasters in deep coals. Jiang et al. [20] and Zhu et al. [21,22] found that gravity and tectonic stress were the foundation stress of determining the rock burst in the stope, and the stress generated by the movement of overlying strata was the force source that induces the rock burst. They believed that the variable structure forms and movement laws of overlying strata were the main reasons for the complex mechanism and various forms of rock burst and finally raised the technical idea of classification evaluation on rock burst. Zhao et al. [23] studied the loading characteristics and instability mechanism of gob-side roadway in deep mining and revealed the “three-load” superimposition rock-burst-inducing mechanism of gob-side roadway under the influence of lateral static load, advanced static load and disturbance dynamic load in goaf. Cao et al. [24–26] conducted an investigation into the stress distribution law and multi-dimensional information of microseismic events near the residual coal pillar, holding that the residual coal pillar was the main influencing factor of rock burst.

In terms of the studies on the relationship between mining disturbance and rock burst, Gong et al. [27,28] found that the critical loading rate obviously existed in the coal–rock combined body, and the mutation rate of rock burst tendency was relatively obvious near the critical loading rate. Zhao et al. [29] illustrated the mechanism of roof energy accumulation and release under the influence of mining speed, believing that the increase in roof energy accumulation was essentially the increase in loading rate when the mining speed changed from slow to fast. Huang et al. [30] conducted uniaxial compression tests on the coal–rock combined body under different loading rates, explaining that the loading rate had a huge effect on failure forms and the rock burst tendency of coal rocks. Cui et al. [31] evaluated the rock burst risk of a coal body under different mining strengths and revealed the mining disturbance characteristics on the working face of deep coal seams under different mining strengths. Feng et al. [32] adopted theoretical analysis and microseismic monitoring to reveal how mining speed influenced the energy released by the movement of the hard roof, indicating that the energy from the hard roof breaking presented an obvious positive correlation with the mining speed. In the aspect of drainage-induced rock burst, scholars have conducted some research studies on the effect of water on the physical and mechanical properties of coal rock, but few studies focus on how drainage affects the stress field of coal and rock. Zhou et al. [33] explored the mechanical behavior of coal samples during water saturation, finding that when the coal around the mining space was subjected to water, the high degree of water saturation in the coal decreased the risks of coal bursts significantly. Zheng et al. [34] pointed out that there was a negative correlation between the compressive strength of coal and soaking time by studying the influence of soaking time on the macro and micro mechanical properties of coal samples. Zhang et al. [35–37] focused on the water weakening of rock strength and its influencing factors (water content, immersion time and wetting–drying cycles), and they found that the strength of the rock mass decreases to varying degrees with water content, immersion time and wetting–drying cycles depending on the rock mass type and mineral composition.

It can be found that the existing rock burst theories usually ignore water drainage as an inducing factor of rock burst, which leads to the lack of reliability and accuracy in the classification of rock burst risk areas on the working face with the roof water-rich areas and the absence of targeted prevention measures. For example, the serious rock burst accident in the drainage lane and no. 3 contact lane of LW1303 in Shandong Longyun Mining Company on 20 October 2018 caused 21 deaths [5] and damaged a 348 m roadway, including a 198 m severely damaged section, as shown in Figure 2. According to the accident investigation report, the influence of roof water drainage on the regional stress distribution was one of the indirect causes. The result showed that roof water drainage was one of the induced factors of rock burst in Ordos deep mining area.



Figure 2. Scene damage of 20 October rock burst accident in Longyun Mine: (a) Anchor bolt bent and suspended; (b) Two sides convergence, roof subsidence.

In view of this, taking LW22106 of the Shilawusu Mine in Ordos as the engineering background, this paper adopts theoretical analysis, similar material simulation and micro-seismic monitoring to investigate the occurrence mechanism of water drainage on rock burst in the roof water-rich area of deep mines and reclassified the risk areas on the working face, attempting to predict the rock burst risk more accurately. The research results can provide a theoretical basis and technical support for safe and efficient mining on the roof water-rich working face in the Ordos mining area.

2. Engineering Background

2.1. Field Conditions of LW22106

The Shilawusu Mine, located in the Ordos mining area, is mainly mining the 2-2 coal seam, whose coal seam, roof and floor have a weak rock burst tendency. LW22106, with a width of 300 m and a length of 1120 m, is the first mining face of the south wing in the 221 mining area, and a strike longwall full-mechanized coal caving method is adopted for mining on this working face. The average surface elevation is 1340 m, the average mining depth is 660 m, and the average coal thickness is 9.02 m. Like a flat seam, the 2-2 coal seam has a simple geological structure. The immediate roof of the 2-2 coal seam is 0.77 m thick mudstone, the basic roof is 31.26 m thick fine sandstone, the direct bottom is 6.10 m thick sandy mudstone, and the basic bottom is 9.25 m thick siltstone. According to the hydrogeological exploration results, there is a Group Zhiluo artesian aquifer with an average thickness of 110 m at 39.65~62.65 m and an average of 50.52 m above LW21106 of the 2-2 coal seam. The water-rich areas are unevenly distributed above the working face. The specific numbers and spatial distribution locations of the four water-rich areas are shown in Figure 3.

During the mining process on the working face, the water drainage has been carried out through drainage holes. Specifically speaking, the drilling fields and the drainage holes are intensively arranged in the vicinity of the water-rich area, with one drilling field for every 200 m. The diameter of drilling holes for roof water drainage is 91 mm, and the final pitch of the holes is less than 80 m. After the drilling is completed, the gate valve and pressure gauge are installed at the orifice. The gate valve is connected with the drainage pipe and flowmeter for drainage water until the water pressure in the water-rich area is reduced up to the safety mining standard, which is that roof water disasters such as water inrush and sand bursting will not occur in the mining process. The microseismic monitoring system is SOS, developed in Poland [38]. Eight microseismic sensors are arranged around LW22106, among which no. 5 and no. 6 sensors are constantly moved and optimized along with the mining on the working face. These sensors are used to monitor microseismic events in the mining process and capture the precursor information of rock burst.

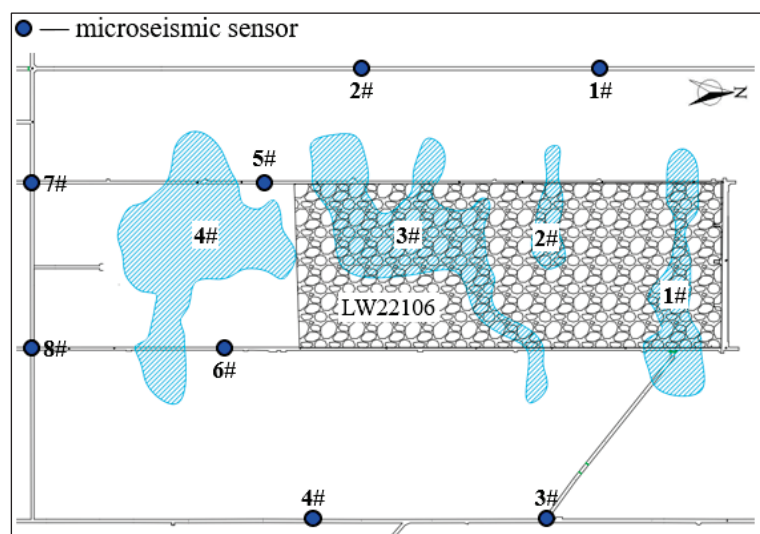


Figure 3. Distribution plan of sandstone water-rich area on roof of LW22106.

2.2. Description of Mine Seismicity on the Working Face

At 19:00 on 10 October 2019, the rock burst appeared in the normal mining process of LW21106, and the staff reported the big sound of coal blasting on the site. After further on-site investigation, obvious large rib splitting occurred on the coal wall of the working face, a large amount of slag fell off the roof in front of the working face, and part of the coal body in the roadway wall leaked out from the anchor net. Meanwhile, the SOS microseismic monitoring system monitored a large energy event of 4.0×10^5 J (source 1) and another large energy event of 1.13×10^5 J (source 2) at 20:06, with a time interval of 66 min. In addition to the sound of coal blasting, there was also a small amount of slag falling off the roof. By comparing the real-time online monitoring results of stress on the working face, it was found that the stress of the two gate roads before and after the incident did not change greatly, and the detection hole of pulverized coal presented a normal amount of drilling cuttings in the field construction. The plane projection and profile projection of the events were shown in Figure 4.

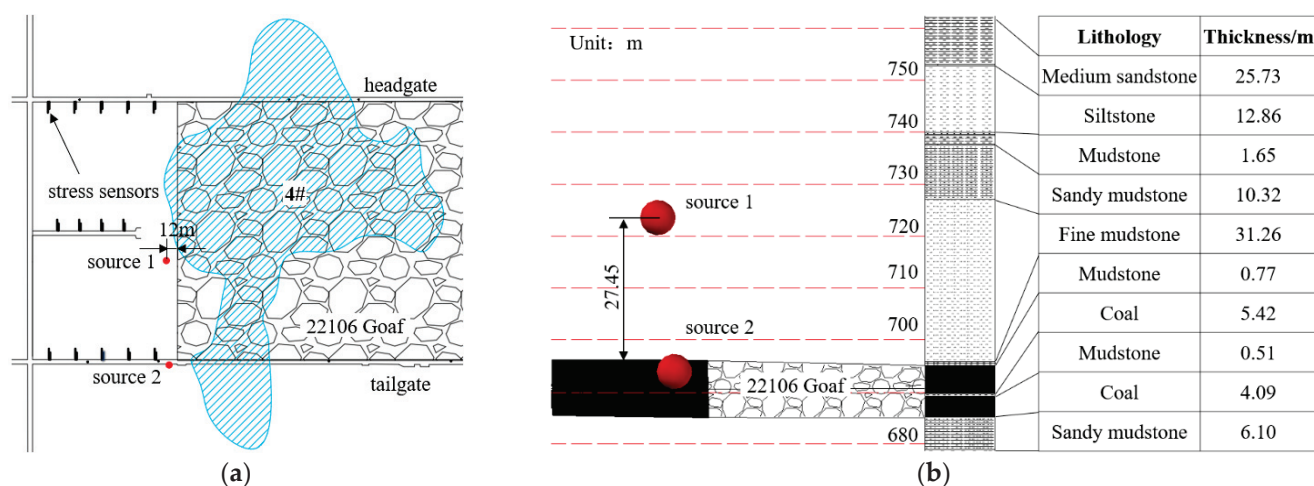


Figure 4. Projection of two microseismic events on 10 October: (a) planar projection; (b) section projection.

As can be seen from Figure 4, Source 1 was horizontally located 12 m in front of the working face and at the edge of the no. 4 water-rich area and vertically located in the basic roof (fine sandstone) above the coal seam with 27.45 m away from the 2-2 coal roof. Source 2 was located in the coal seam. According to preliminary analysis, the water pressure in the no. 4 rich water area declined from the initial 3 MPa to 0.5 MPa after drainage. The

pressure step interval of LW22106 was 20 m, and the mining speed reached 7.2 m/d on that day. Therefore, it was believed that the main reasons for this rock burst were as follows: the overlying basic roof failed to collapse in time due to the excessively fast mining speed on the working face, thus forming a suspended roof and causing energy accumulation; meanwhile, because of the no. 4 water-rich area on the overlying, a stress concentration appeared near the water-rich area after drainage, which increased the stress above the basic roof, accelerated the fracture of the basic roof and produced a large energy event. Moreover, the energy accumulated in the suspended roof could not be quickly released and stored in the coal body in front of the working face. The coal body above the working face was affected by the superposition of the advanced abatement pressure and the drainage transfer stress. The superimposed stress exceeded the own strength of the coal body, thus resulting in its destruction, cracks and energy release and inducing the rock burst on the working face, such as large rib splitting.

3. Mechanism of Rock Burst Induced by Roof Water Drainage

3.1. Principle of Effective Stress

There are irregular water-rich areas in the roof of the upper coal seam in dozens of newly built mines with a mining depth of more than 550 m in Ordos. In order to avoid water inrush and reduce preparation time on the working face during the mining process, the water drainage during mining is equal to a mine for a “quasi-liberating seam” in the water-rich area [39]. The stress decreases in the drainage area and increases at the edge. Therefore, when the working face is advancing to the area affected by drainage, rock burst is more easily induced. It is of great necessity to explore how drainage affects the abutment pressure of the working face.

According to the column chart of the roof strata and the on-site pumping operation, the confined artesian aquifer of Group Zhiluo above the LW22106 coal seam mostly includes medium sandstones and fine sandstones, with the maximum value of confined water pressure up to 3 MPa. The artesian aquifer is saturated with high porosity and permeability, and the confined artesian aquifer of Group Zhiluo meets the principle of effective stress. The principle of effective stress means that the total stress carried by the saturated rock–soil mass is mainly borne by two parts [40]. One is the effective stress, namely the skeleton formed by solid particles, which is transmitted through the contact surface between particles; the other is the pore water pressure, namely the water in the pores. The principle of effective stress can be expressed by Formula (1):

$$\sigma = \sigma' + \mu \quad (1)$$

where σ is the total stress of the saturated rock–soil mass (MPa); σ' is the effective stress (MPa); and μ is the pore water pressure (MPa).

The bearing force of the artesian aquifer is borne by rock skeleton and pore water. After the drainage in the water-rich area, the pore water pressure and effective stress of the coal rock strata and the physical and mechanical properties of the coal rock mass will make changes within a certain range, resulting in the regional stress adjustment and redistribution of the coal seam and roof.

3.2. Stress Migration Law in the Process of Drainage

The above analysis demonstrates that the pore water pressure will gradually decrease and the effective stress of the artesian aquifer will change after the drainage in the water-rich area of the roof. The support of the artesian aquifer to the overlying strata will decrease within the drainage range, and the overlying strata will suffer from deformation and subsidence under the influence of its gravity. The rock mass on both sides of the water-rich area will provide the supporting reaction force. That is to say, the overlying strata load originally borne by the pore water pressure in the water-rich area is transferred to both sides of the water-rich area, which causes the stress redistribution of the coal seam and roof strata, further forming the pressure relief zone, pressurized zone and pressure stabilization

zone. To further explain, the stress of the coal seam and roof reduces within the drainage area, the stress concentrates at the edge of the area, and the range not affected by drainage is the initial stress zone.

In order to investigate the influence of drainage on the stress distribution of the coal seam and roof before and after drainage in the water-rich area, taking LW22106 of the Shilawusu Mine in Ordos as an example, a section is made along the strike of the working face, and the corresponding estimation mechanical model of abutment pressure after drainage is established to analyze the stress state above the working face before and after drainage, as shown in Figure 5.

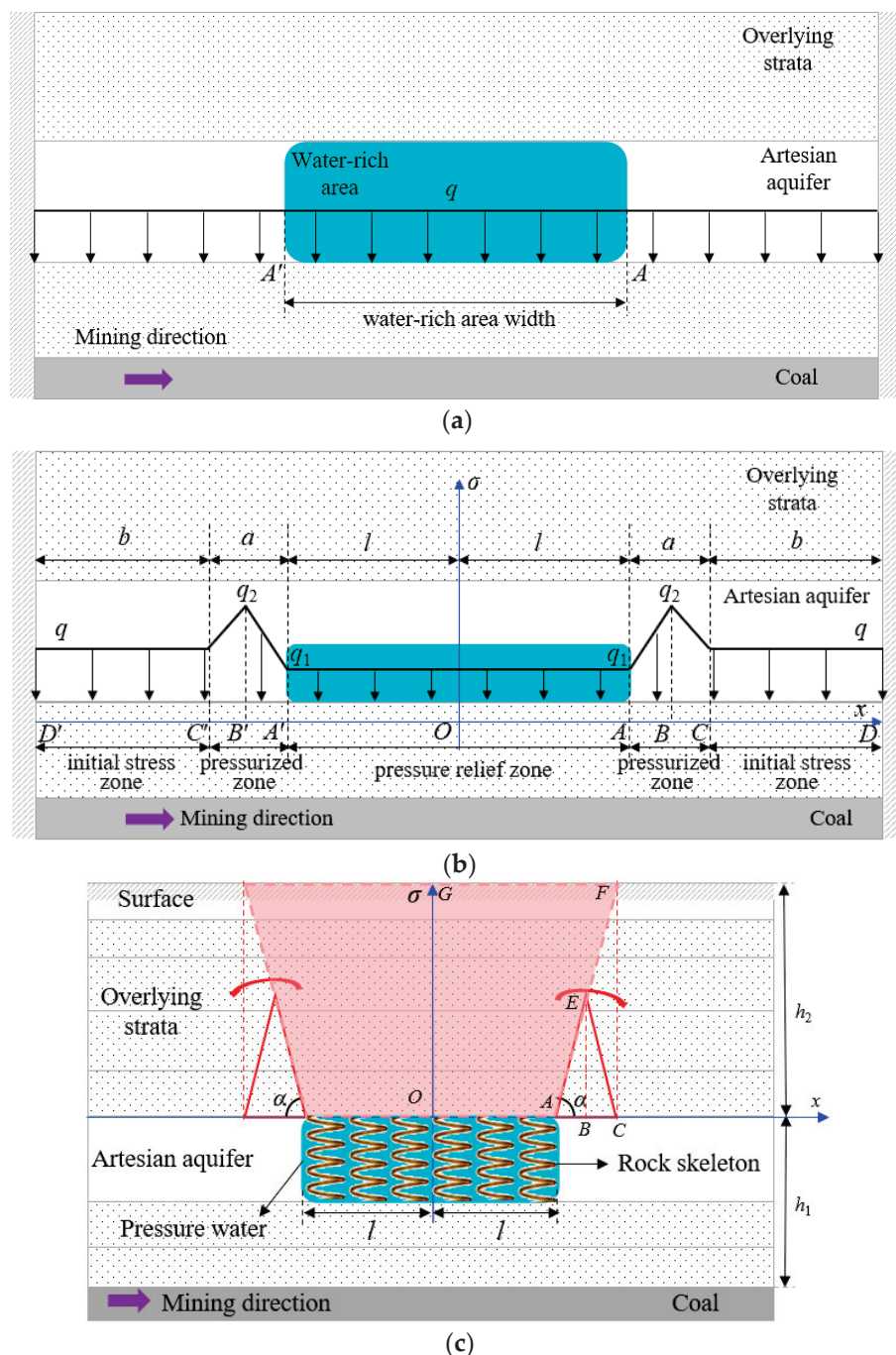


Figure 5. Stress distribution state before and after drainage: (a) before drainage; (b) after drainage; and (c) mechanical model for abutment pressure after drainage.

The following assumptions are made for drawing the general law: (1) for the model, the deformation of coal seam and roof strata meets three basic assumptions of material mechanics; (2) for the stress, the water level and water pressure in the area before and after drainage are uniform, all strata are uniformly stressed, and the differences in stress transfer caused by different rock joints are ignored; (3) for the boundary, only an ideal section of the working face is selected for the theoretical analysis because of the irregular shape in the actual water-rich area. In Figure 5a, before the implementation of drainage in the water-rich area, the strata of the artesian aquifer are intact, the overlying strata load q is uniformly transferred, and the load on the artesian aquifer is the original rock stress q . In Figure 5b, the origin O is the center of the water-rich area. After drainage in the water-rich area, the pore water pressure decreases, forming a pressure relief zone. The support capacity of the upper roof in the drainage range of the water-rich area decreases to q_1 ($A'A$ section), and the rock mass stress in $A'C'$ and AC sections increases, forming a pressurized zone. The stress distribution in the pressurized zone can be simplified into a triangular linear distribution (the maximum stress is q_2). The original rock stress in the $C'D'$ and CD sections remains unchanged, forming a pressure stabilization zone. Let the length of the OA (OA') section be l , AC ($A'C'$) section be a and the CD ($C'D'$) section be b , and then the stress distribution in $D'D$ section is as follows:

$$\sigma(x) = \begin{cases} q & (x \in D'C') \\ \frac{2(q_2-q)}{a}(x+l+\frac{a}{2}) + q_2 & (x \in C'B') \\ \frac{2(q_1-q_2)}{a}(x+l) + q_1 & (x \in B'A') \\ q_1 & (x \in A'A) \\ \frac{2(q_2-q_1)}{a}(x-l) + q_1 & (x \in AB) \\ \frac{2(q-q_2)}{a}(x-l-\frac{a}{2}) + q_2 & (x \in BC) \\ q & (x \in CD) \end{cases} \quad (2)$$

where q is the original rock stress (MPa); q_1 is the load in the pressure relief zone after drainage (MPa); q_2 is the maximum load in the pressurized zone (MPa); l is the half-width of the water-rich area (m); a is the range of the pressurized zone, expressed as $a = h_2 / \tan \alpha$ (m); and b is the range of the pressure stabilization zone (m).

As can be seen from Figure 5 and Formula (2), the stress distribution after drainage is ideally symmetric with respect to the origin O , which means equal stress is transferred to both sides after drainage. Therefore, it just needs to make a detailed analysis of the stress distribution in the OD section on the right side of the origin O , as shown in Figure 5c. In Figure 5c, h_1 is the distance from the top of the artesian aquifer to the coal seam (m); h_2 is the distance from the top of the artesian aquifer to the surface (m); and α is the displacement angle of overlying strata in the water-rich area caused by drainage. (Compared with mining, the support of the artesian aquifer on its upper strata still exists in spite of a decline in support effect. Moreover, the overlying strata over the aquifer will suffer from fracture bands, but caving bands will not occur or the overlying strata will not stay in a completely overhanging state).

In this model, the water-rich area is simplified to be composed of confined water and a rock skeleton. The rock skeleton can be regarded as an equivalent spring, exerting a supporting reaction force on the overlying strata. The supporting reaction force of the water-rich area on the overlying strata before and after drainage is expressed as:

$$q = \gamma h_2 \quad q_1 = k \gamma h_2 \quad (3)$$

where γ is the rock bulk density (kN/m^3) and k is the drainage pressure relief coefficient ($0 < k < 1$), negatively correlated with the drainage degree; that is, the higher degree of the drainage, the smaller the k .

After drainage, the gravity stress in the overlying OAFG region is transferred to the right of Δ_{ACE} , and the transferred weight Q_{OAFG} can be expressed as:

$$Q_{OAFG} = (2l + \frac{h_2}{\tan \alpha}) \frac{\gamma h_2}{2} - q_1 l \quad (4)$$

Assuming that stress transferred to the lower coal rock mass is approximately an isosceles triangular distribution [20], the load transferred to the right of the OAFG region σ_{OAFG} is as follows:

$$\sigma_{OAFG}(x) = \begin{cases} \frac{2q_2 \tan \alpha}{h_2} (x - l) & (l, l + \frac{h_2}{2 \tan \alpha}] \\ 2q_2 [1 - \frac{\tan \alpha}{h_2} (x - l)] & [l + \frac{h_2}{2 \tan \alpha}, l + \frac{h_2}{\tan \alpha}) \\ 0 & [l + \frac{h_2}{\tan \alpha}, +\infty) \end{cases} \quad (5)$$

where x is the distance between the transfer stress and the center of the water-rich area. The peak stress transferred from the OAFG region to the pressurized zone q_2 can be expressed as:

$$q_2 = \frac{Q_{OAFG} 2 \tan \alpha}{h_2} = (2l \tan \alpha + h_2) \gamma - \frac{2q_1 l \tan \alpha}{h_2} \quad (6)$$

When the working face is under the water-rich area (pressure relief zone), at the edge of the water-rich area (pressurized zone) and in a more distant area (pressure stabilization zone), the gravity stress of the working face from the overlying strata after drainage differs greatly, which can be expressed as:

$$\sigma_Z(x) = \begin{cases} q_1 + \gamma h_1 & (0, l] \\ (1 - k) \gamma (x - l) \tan \alpha + q_1 + \gamma h_1 & [l, l + \frac{h_2}{\tan \alpha}) \\ \gamma h_1 + \gamma h_2 & [l + \frac{h_2}{\tan \alpha}, +\infty) \end{cases} \quad (7)$$

Based on Formulas (2)–(7), the distribution function of coal seam abutment pressure caused by drainage can be written:

$$\sigma_S(x) = \begin{cases} \gamma h_1 + k \gamma h_2 & (0, l] \\ \frac{2(2l \tan \alpha + h_2 - kl \tan \alpha) \gamma \tan \alpha}{h_2} (x - l) + (1 - k) \gamma (x - l) \tan \alpha + \gamma h_1 + k \gamma h_2 & [l, l + \frac{h_2}{2 \tan \alpha}) \\ 2(2l \tan \alpha + h_2 - kl \tan \alpha) \gamma [1 - \frac{\tan \alpha}{h_2} (x - l)] + (1 - k) \gamma (x - l) \tan \alpha + \gamma h_1 + k \gamma h_2 & [l + \frac{h_2}{2 \tan \alpha}, l + \frac{h_2}{\tan \alpha}) \\ \gamma h_1 + \gamma h_2 & [l + \frac{h_2}{\tan \alpha}, +\infty) \end{cases} \quad (8)$$

As can be seen from Formula (8), the stress concentration caused by drainage shows a negative correlation with the pressure relief coefficient in the water-rich area and is positively correlated with the drainage degree. That is to say, the better the drainage effect, the greater the stress peak in the pressurized zone. According to the research [12,14] and the field observation of water pressure, the water pressure in the rich-water area gradually decreases to the safe mining standard (that is, no water disasters occur, such as water inrush) and eventually tends to be stable with the progress of drainage work. The results show that the stress concentration degree caused by drainage tends to be stable during mining on the working face; the pressure relief zone, pressurized zone and pressure stabilization zone are formed in sequence from the center to both sides; and the pressurized zone applies additional stress to the coal seam and its roof, adding a static stress source for the occurrence of rock burst.

3.3. Physical Simulation Verification

Starting from the typical geological conditions of the Ordos mining area, the similarity ratios are determined based on the rock strata condition, physical and mechanical properties of rock [41], geometric size of the test bed and three basic similarity laws. Among them,

the geometric ratio is 1/200, the bulk density ratio is 0.6, the strength ratio is 0.6/200, and the time ratio is 1/14. For the convenience of time simulation, the time ratio is set as 1/12; that is, 2 hours in the experiment equals the on-site 24 h. The model size is 5000 mm (length) $\times$ 250 mm (width) $\times$ 1680 mm (height). Sand is used as the skeleton material, and calcium carbonate and gypsum are used as cementing materials. Table 1 displays the proportions of similar simulation materials. A pressurized steel plate is arranged at the top of the model, and eight hydraulic jack devices are uniformly arranged above the steel plate to exert vertical forces on the model, which replaces the rock layer above that needs to be simulated.

Table 1. Proportion of similar simulation materials.

Serial Number	Lithology	Thickness/cm	Total Thickness/cm	Similar Materials/kg			
				Sand	CaCO ₃	Gypsum	Water
26	Sandy mudstone	5	158	85.23	2.56	5.97	9.38
25	Fine sandstone	4	153	66.67	5.00	3.33	7.50
24	Sandy mudstone	5	149	85.23	2.56	5.97	9.38
23	Fine sandstone	4	144	66.67	5.00	3.33	7.50
22	Siltstone	19	140	316.67	27.71	11.88	35.63
21	Fine mudstone	5	121	83.33	6.25	4.17	9.38
20	Medium sandstone	4	116	65.63	5.63	3.75	7.50
19	Sandy mudstone	5	112	85.23	2.56	5.97	9.38
18	Medium sandstone	3	107	49.22	4.22	2.81	5.63
17	Sandy mudstone	4	104	68.18	2.05	4.77	7.50
16	Medium sandstone	2	100	32.81	2.81	1.88	3.75
15	Sandy mudstone	5	98	85.23	2.56	5.97	9.38
14	Medium sandstone	5	93	82.03	7.03	4.69	9.38
13	Sandy mudstone	5	88	85.23	2.56	5.97	9.38
12	Fine sandstone	5	83	83.33	6.25	4.17	9.38
11	Sandy mudstone	16	78	272.73	8.18	19.09	30.00
10	Siltstone	4	62	66.67	5.83	2.50	7.50
9	Sandy mudstone	12	58	204.55	6.14	14.32	22.50
8	Fine sandstone	4	46	66.67	5.00	3.33	7.50
7	Siltstone	13	42	216.67	18.96	8.13	24.38
6	Medium sandstone	1	29	16.41	1.41	0.94	1.88
5	Sandy mudstone	4	28	68.18	2.05	4.77	7.50
4	Medium sandstone	10	24	164.06	14.06	9.38	18.75
3	Siltstone	6	14	105.00	7.88	18.38	13.13
2	2-2 Coal	5	8	48.21	5.63	2.41	5.63
1	Sandy mudstone	3	3	233.33	20.42	8.75	26.25
Total				2803.14	180.27	166.59	315.00

Water-rich areas exist in the coal seam roof. In order to analyze the stress evolution law before and after drainage, the no. 4 medium sandstone part in Table 1 is taken as an artesian aquifer. Several water pipes are placed in the aquifer during the building of the model. The water pipe is 2.1 m long, 1.4 m from the left boundary and 1.5 m from the right boundary of the model. The water pipe is provided with a cut-off valve, and the tail end is close to the rear steel plate for the convenience of drainage, as shown in Figure 6a. Ten UEI pressure boxes with high precision are staggered under the artesian aquifer and coal seam to monitor the stress transfer of the coal seam and roof during and after drainage, as shown in Figure 6b. The spacing between the test model and stress sensor is shown in Figure 6c.

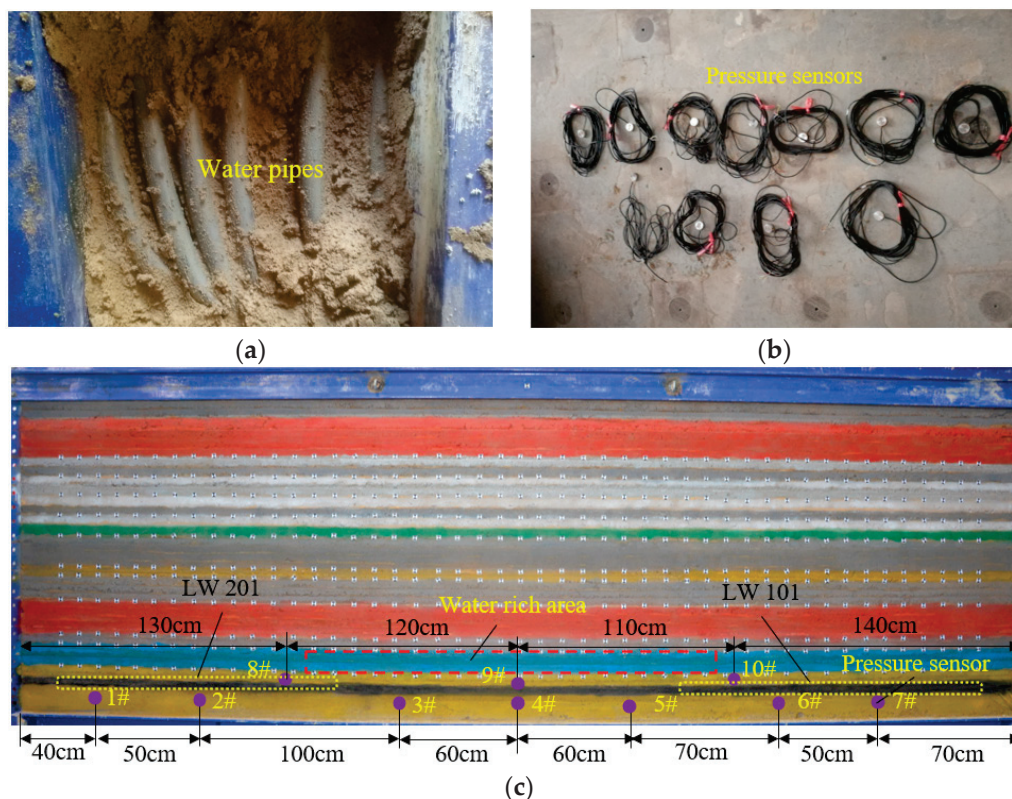


Figure 6. Physical simulation model: (a) water pipes laying; (b) pressure sensors; and (c) test model layout. The different numbers are stress measuring points.

In order to study the influence of water drainage on coal seam mining, a 170 cm coal pillar is set in the middle to avoid the influence of excavation on both sides of the working face. The mining sequence is as follows: (1) first, excavate LW101 from the position 20 cm to 170 cm away from the right boundary of the model, with a total length of 150 cm; (2) secondly, open the stop valve of water pipe to release all of the water, and stand for a week after the completion of water drainage; (3) finally, excavate LW201 from the position 20 cm to 160 cm away from the left boundary of the model, with a total length of 140 cm. Before the initial excavation, the stress monitoring instrument is used to clear the sensor stress to zero for comparison.

Figure 7a shows the stress changing curve of no. 8 and no. 9 measuring points after drainage. It can be seen that the stress at the no. 9 measuring point under the water-rich area shows an upward trend after drainage, with a sharp increase from the initial 0 kPa to 13 kPa between 0.5 h and 1 h after drainage, and eventually stabilizes to 16 kPa after 5 h. Meanwhile, the stress at the no. 8 measuring point at the edge of the water-rich area gradually declines from the initial 0 kPa to −7 kPa after 6 h. The drainage in the water-rich area will lead to the stress change in the coal seam, namely the stress decrease under the water-rich area and the stress increase at the edge of the water-rich area, thus forming the pressurized zone and pressure relief zone. Figure 7b is the stress changing curve of no. 1~no. 4 measuring points after drainage. It can be found that no apparent stress changes have taken place at no. 1 and no. 2 measuring points after drainage, which are far away from the water-rich area, and the change has been floating up and down within the allowable error range. The stress at no. 3 and no. 4 measuring points under the water-rich area decreases from 0 kPa to −1.75 kPa and from 0 kPa to −2 kPa, respectively. The stress changing curve of no. 8 and no. 9 measuring points displays that the influence range affected by drainage stress will not expand indefinitely, and the stress concentration simply appears in a certain range at the edge of the water-rich area.

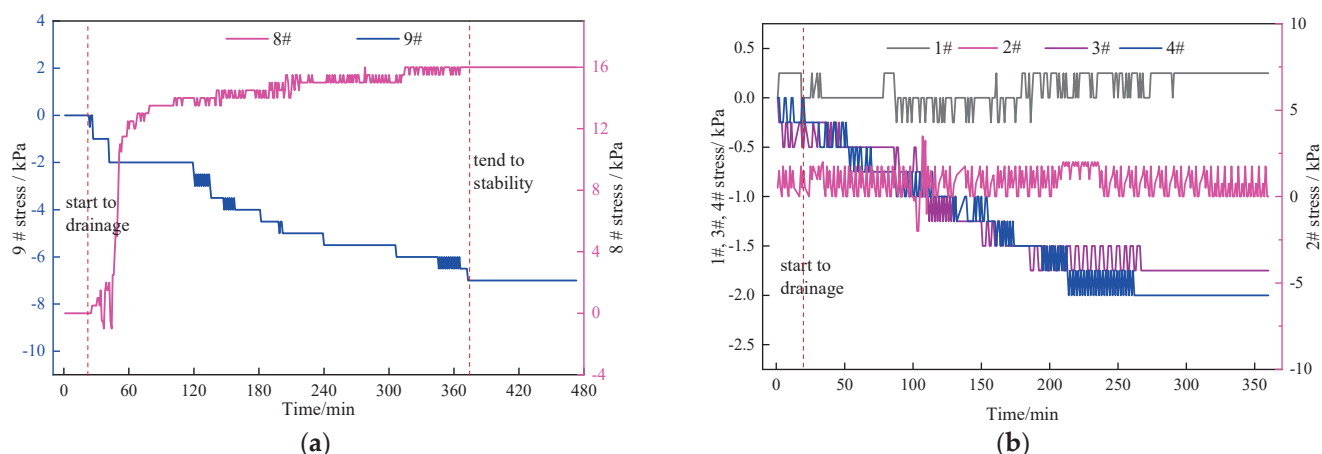


Figure 7. Stress changing curves of each measuring point after drainage: (a) no. 8, no. 9; and (b) no. 1–no. 4.

Figure 8a shows the stress changing curves at the no. 6 measuring point with LW101 excavated for 90~95 cm and at the no. 2 measuring point with LW201 excavated for 60~65 cm. At this time, the no. 6 measuring point is 100 cm away from the open cutting of LW101 and 5 cm away from the advancing position of LW101; the no. 2 measuring point is 70 cm away from the open cutting hole of LW201 and 5 cm away from the advancing position of LW201. It can be seen that the two measuring points basically keep a consistent stress growth trend, and the initial stress of the no. 2 measuring point is only 2.5 kPa lower than that of the no. 6 measuring point, which is within the allowable error range of the instrument.

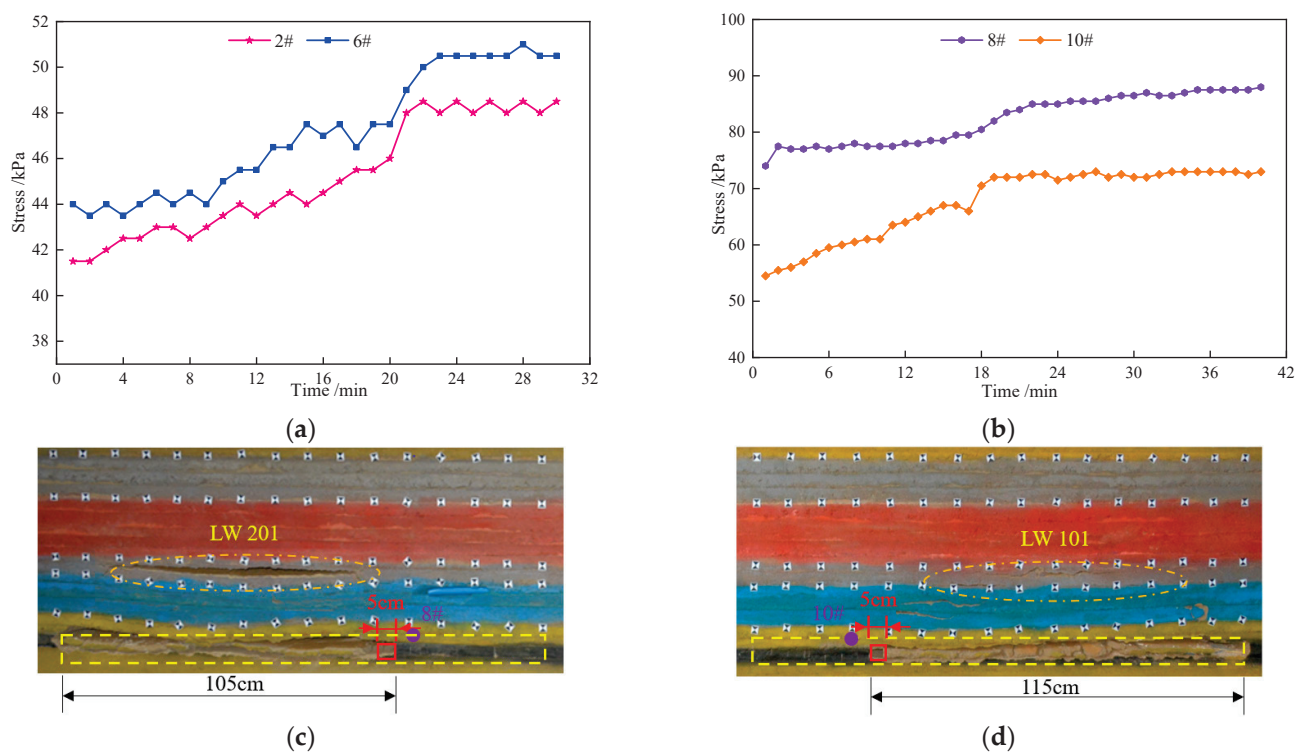


Figure 8. Stress changing curve of each measuring point during mining: (a) no. 2, no. 6; (b) no. 8, no. 10; (c) excavation location of LW201, no. 6; and (d) excavation location of LW101.

Figure 8b shows the stress changing curves at the no. 10 measuring point with LW101 excavated for 110~115 cm and at the no. 8 measuring point with LW201 excavated for

100~105 cm. At this time, the no. 10 measuring point is 120 cm distant from the open cutting of LW101 and 5 cm from the advancing position of LW101; the no. 8 measuring point is 110 cm from the open cutting hole of LW201 and 5 cm from the advancing position of LW201. It can be obtained that the two measuring points almost present the same stress growth trend, and the initial stress of the no. 8 measuring point is 19.5 kPa higher than that of the no. 10 measuring point. When the two working faces are advanced to 5 cm away from the measuring point, the stress of the no. 8 measuring point is 15 kPa higher than that of the no. 10 measuring point. The stress difference is similar to the stress increment of 16 kPa at the no. 8 measuring point after drainage in Figures 7a and 8c,d demonstrates the spatial relationship between the working face and the stress measuring point when LW101 is excavated for 110~115 cm and LW201 for 100~105 cm.

Above all, the influence range of drainage stress will not expand indefinitely, and stress concentration only appears in a certain range at the edge of the water-rich area. Compared with the working face not affected by water drainage, when the working face advances to the drainage-affected area, the mining disturbance stress and drainage transfer stress will be superimposed. If the superimposed stress reaches the critical stress, rock burst will be induced, which verifies the correctness of the theoretical calculation model.

3.4. Analysis of the Relationship between Drainage and Rock Burst

In order to evaluate the rock burst risk at point N , the rock burst risk evaluation index I_c [42] proposed by our research team is introduced. I_c stands for the ratio of vertical stress at point N to the uniaxial compressive strength of coal rock mass, expressed as:

$$I_c = \frac{\sigma_N}{[\sigma_c]} \quad (9)$$

where σ_N is the vertical stress at point N and $[\sigma_c]$ is the uniaxial compressive strength of coal rock mass. The classification of rock burst risk is shown in Table 2.

Table 2. Relationship between I_c and rock burst risk.

I_c	<1.5	1.5~2.0	2.0~2.5	>2.5
Rock burst risk	None	Weak	Medium	Powerful

In the mining process of the roof water drainage working face, the superimposed stress on the coal body varies according to the position of point N (as shown in Figure 9).

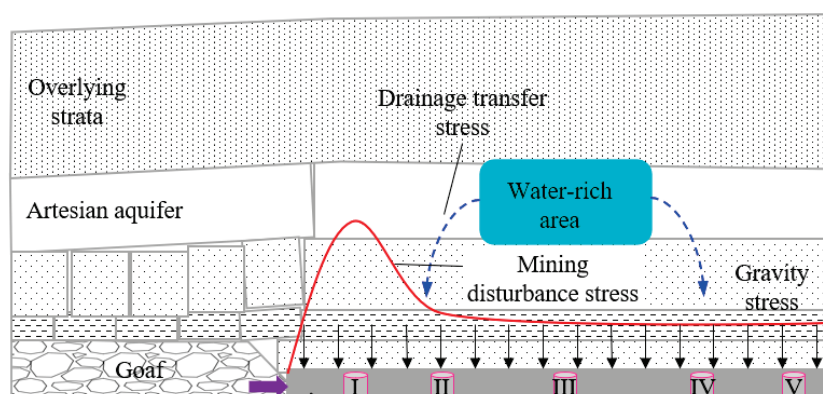


Figure 9. Stress environment of coal body at a certain point in the mining process of roof water drainage working face. The I–V is the position of point N .

The following situations typically exist: (1) affected by gravity stress and mining disturbance stress (I, V); (2) affected by gravity stress and pressure relief from drainage and pressurization from mining disturbance under the water-rich area; and (3) affected by

gravity stress and pressurization from drainage and mining disturbance at the edge of the water-rich area (II, IV). The stress state at point N can be approximated as:

$$\sigma_N = q + [\sigma_M - q] + [\sigma_S(x) - q] = \sigma_M + \sigma_S(x) - q \quad (10)$$

where σ_N is the superimposed stress at point N , σ_M is the mining disturbance stress at point N , and x is the horizontal distance between point N and the center of the water-rich area.

According to Formula (10), the vertical stress at point N is related to the mining speed and the drainage degree. When it is affected by both drainage pressurization and mining disturbance pressurization, the stress condition of rock burst induction can easily be reached.

The above analysis explains the rock burst mechanism induced by water drainage in the roof water-rich area of deep mines. That is to say, when the working face is passing the pressurization area by drainage, the coal rock mass is under the combined influence of gravity stress, mining disturbance stress and drainage transfer stress, and the superimposed stress easily exceeds the critical value of rock burst, which is the main reason to induce rock burst.

4. Field Application and Verification

4.1. Field Application

Water drainage is taken before mining on LW21106. Water pressure in the water-rich area decreases from 3 MPa at the beginning to 0.5 MPa after drainage. After the drainage is completed, the pressure relief zone and pressurized zone are formed in front of the working face, which facilitates the changes in the rock burst risk areas on the working face. In this case, the rock burst risk areas can be re-predicted and divided according to the location of risk areas.

Due to the weak rock burst liability of seam 2-2 in the Shilawusu Mine, the rock burst risk assessment of LW22106 has been carried out before mining. Field practice and research show that the peak value of mining disturbance stress is generally 1.5~1.8 times the original rock stress under the condition of a constant mining speed. It can be calculated that the peak value of mining disturbance stress is about 24.75~29.7 MPa with an average of 27.225 MPa, and the rock burst risk index $I_c = 27.225/17.6 = 1.55$. Thus, the working face is determined to have a weak rock burst risk. Taking the no. 4 water-rich area as an example, the multi-factor coupling evaluation method is adopted to divide the rock burst risk areas around the no. 4 water-rich area, including three weak rock burst risk areas and one medium rock burst risk area, as shown in Figure 10 and Table 3.

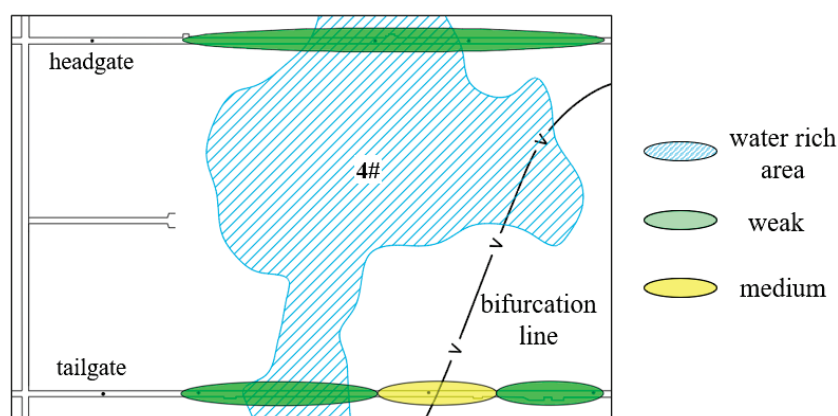


Figure 10. Division diagram of rock burst risk areas around no. 4 water-rich area before drainage.

Table 3. Division table of rock burst risk areas around no. 4 water-rich area before drainage.

Location	Serial Number	Range/m	Main Factors	Risk Degree of Rock Burst
Headgate	1	740~1100	Depth, mining disturbance	Weak
	2	740~830	Depth, mining disturbance	Weak
Tailgate	3	830~930	Depth, mining disturbance, bifurcation of coal	Medium
	4	930~1100	Depth, mining disturbance	Weak

However, the evaluation result does not take into account the influencing factors of drainage in the water-rich area. Therefore, it can be believed that the evaluation result is a classification method of rock burst risk areas simply considering the mining disturbance before drainage.

After the completion of water drainage, the pressure relief zone and pressurized zone are formed in front of the working face, which results in the change in the rock burst risk areas on the working face. According to the actual situation of LW22106, the values are as follows: $h_1 = 160$ m, where h_1 is defined as the distance from the top of the artesian aquifer to the coal seam; $h_2 = 500$ m, where h_2 represents the distance from the surface to the top of the artesian aquifer; $2l_1 = 40$ m, where $2l_1$ refers to the average width of the narrow area in the no. 4 water-rich area; $2l_1 = 260$ m, where $2l_1$ is the average width of the wide area; $\alpha = 86^\circ$, where α means the fracture angle of the overlying strata in the water-rich area after drainage; $k = 0.8$, where k is taken as the pressure relief coefficient (the water pressure in the water-rich area is reduced from the initial 3 MPa to 0.5 MPa after drainage, which reports that the pressure relief value is about 20% of the original stress of water-rich artesian aquifer, so $k = 1 - 0.2 = 0.8$); $[\sigma_c] = 17.6$ MPa, where $[\sigma_c]$ stands for the uniaxial compressive strength of coal; and $\gamma = 25$ kN/m³. By substituting each parameter into Equation (8), it can be obtained that the stress influence range after drainage is about 35 m, the peak value is about 17.5 m away from the edge of the water-rich area, the stress peak value is 33.4 MPa in the narrow zone of the no. 4 water-rich area, and the stress peak value is 46.3 MPa in the wide zone of the no. 4 water-rich area.

Given the mining disturbance stress and the stress transfer before and after drainage in the no. 4 water-rich area, it can be obtained that:

When the working face is advanced to the pressurized zone in the no. 4 water-rich area, the stress peak value $\sigma_{Nmax1} = 27.225 + 33.4 - 16.5 = 44.125$ Mpa, $I_c = 2.51$, $\sigma_{Nmax2} = 27.225 + 46.3 - 16.5 = 57.025$ Mpa and $I_c = 3.24$. It can be comprehensively determined that the working face has a powerful rock burst risk at this time.

When the working face is advanced to the pressure relief zone in the no. 4 water-rich area, the stress peak value $\sigma_{Mmax3} = 27.225 + 14 - 16.5 = 24.725$ MPa and $I_c = 1.40$, which indicates there is no rock burst risk on the working face at this time.

It can be seen that it is easy to induce the rock burst when the working face is advanced to the pressurized zone, which is also consistent with the rock burst location on 10 October. On this basis, the rock burst risk areas after drainage are re-predicted and re-divided for the surrounding area of the no. 4 water-rich area, including two no rock burst risk areas, four weak rock burst risk areas, one medium rock burst risk area and six powerful rock burst risk areas, as shown in Figure 11 and Table 4.

By comparing Figures 10 and 11 and Tables 3 and 4, it can be obtained that the number of risk areas increases from 4 before drainage to 13 after drainage, one in-plane rock burst risk area is added, and the risk degrees change from weak and moderate rock burst before drainage to no, weak, medium and powerful rock burst after drainage. It can also be found that the pressurized zone and the pressure relief zone on the working face and two gate roads before and after drainage in the water-rich area lead to great changes in the rock burst risk zones and risk degrees on the working face. The rock burst risk degree is remarkably reduced within the pressure relief zone, whereas it significantly increases in the pressurized zone. All rock burst locations on 10 October 2019 are in the no. 13 in-plane

pressurized zone, which further demonstrates that the division of the rock burst risk areas on the working face after drainage is extremely consistent with the field practice.

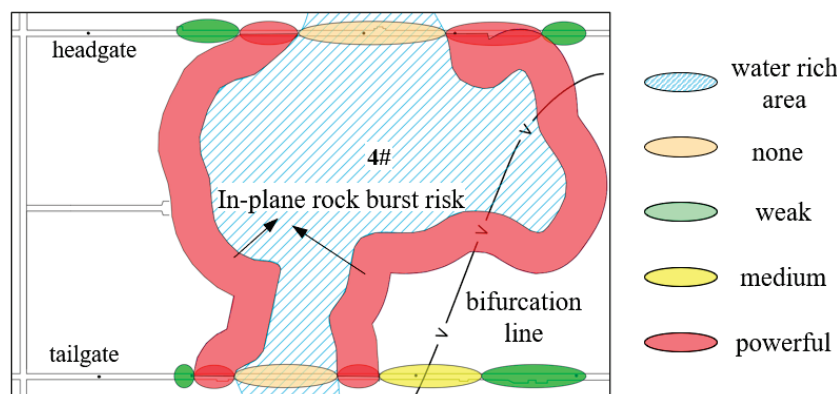


Figure 11. Division diagram of rock burst risk areas around no. 4 water-rich area after drainage.

Table 4. Division table of rock burst risk areas around no. 4 water-rich area after drainage.

Location	Serial Number	Range/m	Main Factors	Risk Degree of Rock Burst
Headgate	1	740~780	Depth, mining disturbance	Weak
	2	Distance from open-off cut 780~862	Depth, mining disturbance drainage pressurization	Powerful
	3	862~988	Depth, mining disturbance drainage pressure relief	None
	4	988~1038	Depth, mining disturbance drainage pressurization	Powerful
	5	1038~1100	Depth, mining disturbance	Weak
	6	740~830	Depth, mining disturbance	Weak
	7	Distance from open-off cut 830~918	Depth, mining disturbance Bifurcation of coal	Medium
Tailgate	8	918~953	Depth, mining disturbance bifurcation of coal, drainage pressurization	Powerful
	9	953~1043	Depth, mining disturbance drainage pressure relief	None
	10	1043~1078	Depth, mining disturbance drainage pressurization	Powerful
	11	1078~1100	Depth, mining disturbance	Weak
In-plane	12	no. 4 water rich area Right side 0~35	Depth, mining disturbance drainage pressurization	Powerful
	13	Left side 0~35	Depth, mining disturbance drainage pressurization	Powerful

4.2. Field Verification

Each microseismic event has rich temporal and spatial information [43]. Real-time monitoring and processing of microseismic events can identify the occurrence location and intensity of microseismic events, further determining the stress transition inside coal rock masses. Figure 12 shows the relationship between daily footage and daily microseismic energy of LW22106 from 1 September 2019 to 23 October 2019. Figure 13 is the plane projection of microseismic events above the 4th power of LW22106 during this period. It can be seen that mining Zone I from 1 September to 9 September was partly under the water-rich area and partly outside the influence range of drainage, and the microseismic energy just had a small change with the increase in mining speed. From 10 September to 18 September, mining Zone II was partly under the water-rich area and partly within the pressurized zone, and the microseismic energy increased significantly at a high mining

speed compared with Zone I. Mining Zone III from 19 September to 4 October was almost under the water-rich area, and the microseismic energy was still very small in spite of a high mining speed (maximum 8.8 m/d). From 5 October to 14 October, most of mining Zone IV was within the pressurized zone, and the high mining speed resulted in the maximum daily energy and a significant increase in total microseismic energy. Most of mining Zone V from 15 October to 23 October was not affected by water drainage, and the microseismic energy was changed little with the changing mining speed. The red line in Figure 13 represents the footage of the working face. It can be found that the microseismic events mostly occurred within the pressurized zone, which proved that this zone had a high degree of concentration with the working face being advanced here. Therefore, rock burst appeared on the working face on 10 October.

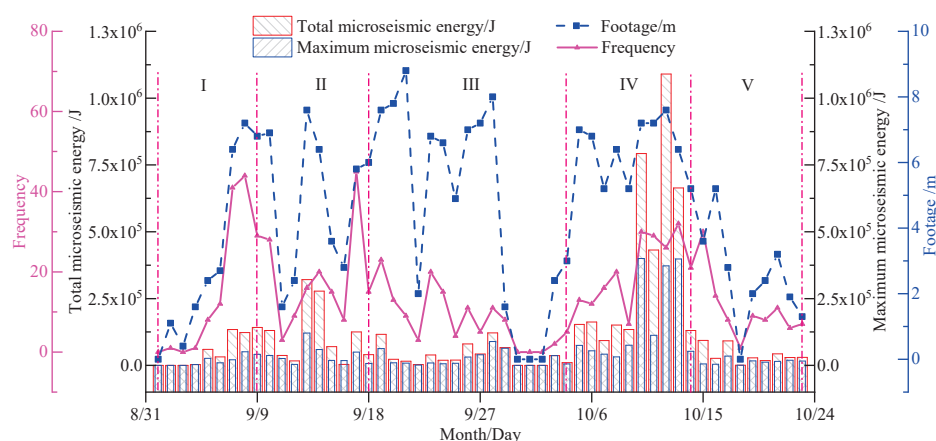


Figure 12. Curves of daily advance and microseismic energy.

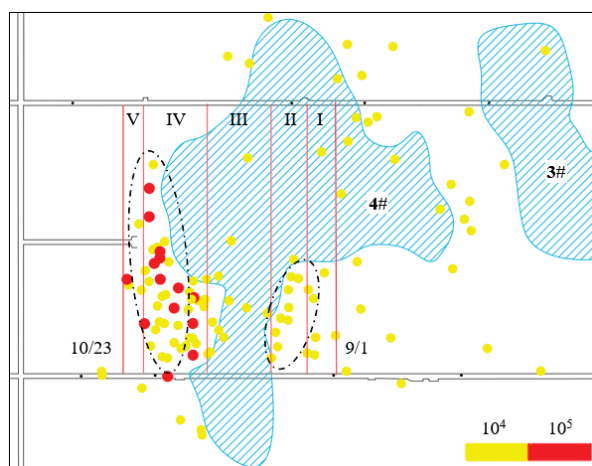


Figure 13. Plane projection of microseismic events above the 4th power of LW22106.

Reducing the mining speed can lower the peak of advanced abutment pressure and the overhanging distance of the basic roof, thus providing reserved time for rock energy release and decreasing the deformation rate of surrounding rock. However, a blind decrease in mining speed will cause huge economic losses. Taking into account the social resource utilization, corporate economic development and safety production, the dynamic control of the mining speed is suggested in the areas affected by water drainage.

In the mining process, according to the mining situation in the normal areas (no fault, no water-rich and other stress abnormal areas), the pressure step distance and preliminary reasonable mining speed can be determined through the microseismic statistical curve and theoretical calculation. When the working face is passing the pressurized zone, the mining speed should be reduced to ensure the stress peak cannot meet the condition of inducing

rock burst. The mining speed can be increased appropriately within the pressure relief zone, but the superimposed stress should not reach the critical value of rock burst.

5. Conclusions

Taking the roof water-rich LW22106 of the Shilawusu Mine in Ordos as the engineering background, this paper investigates the occurrence mechanism of rock burst induced by water drainage in deep mines through theoretical analysis, similar material simulation and microseismic monitoring and draws the following main conclusions:

(1) The influence range of drainage stress does not expand indefinitely. A stress concentration just appears within a certain range at the edge of the water-rich area and tends to have been stable during mining on the working face. The pressure relief zone, pressurized zone and pressure stabilization zone are formed in sequence from the center to both sides. The width of the pressure relief zone is consistent with that of the water-rich area, and the width of the pressurized zone is about 35 m on one side. The pressurized zone applies additional stress to the coal seam and its roof, adding a static stress source for the occurrence of rock burst.

(2) During the forward mining process on the roof water drainage working face, the stress environment is different due to the different positions of a certain point *N* in coal. When the mining is passing the pressurized zone, the coal rock mass is under the joint influence of gravity stress, mining disturbance stress and drainage transfer stress. The superimposed stress generally exceeds 2.5 times (UCS) the coal rock mass, and the maximum reaches 3.24 times that, far exceeding the critical value of rock burst (1.5 times UCS), which is the main reason for inducing rock burst.

(3) By calculating the stress distribution of the drainage-affected zone in the no. 4 water-rich area on the working face, the dynamic change in the rock burst risk areas before and after drainage has been predicted. The number of risk areas increases from 4 before drainage to 13 after drainage, and the in-plane rock burst risk areas are added. The risk degrees change from weak and moderate rock burst before drainage to no, weak, medium and powerful rock burst after drainage. The effect of stress superposition caused by different mining speeds is different. Therefore, it is proposed that the dynamic regulation of mining speed in front and at the back of the drainage area is an effective and efficient method for rock burst prevention and control. That is to say, the mining speed should be dynamically adjusted on the basis of the theoretical analysis and microseismic monitoring results.

This paper attempts to provide mine researchers and designers a classification method of the rock burst risk areas with consideration of drainage so as to predict the rock burst risk of drainage working faces in a more accurate way. However, the current research does not take into consideration the influence of dynamic drainage on the strength of coal rock masses. Further studies on how drainage affects the macro- and microstructures and mechanical properties of coal rock masses are still needed.

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Article

Coal Burst Prevention Technology and Engineering Practice in Ordos Deep Mining Area of China

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Abstract: With the coal mines in western China entering the field of deep mining, the problem of coal burst is becoming more and more serious. According to the characteristics of deep mining, it is an urgent problem that requires the development of an efficient and reasonable coal burst prevention and control plan to guide project practices. This study takes the typical deep mining area in Ordos as the research background, according to the stress state of the coal mining area and the load form of induced coal burst, which, in Ordos deep mining, is divided into the typical and atypical type. The former is caused by the superposition of high in situ stress and strong mining-induced stress, while the latter is due to the combination of high in situ stress, strong mining-induced stress, and external stress disturbances. Combined with theoretical analysis, numerical simulation, and field measurement, it is shown that the stress level of the Ordos deep mining area is higher than that of the shallow original rock, and the difference of the three-dimensional stress between coal and rock mass is greater. The concentration degree and influence range of mining-induced stress obviously increase. Coal and rock mass are more prone to instability and failure due to external disturbances. Based on the stress control theory, the prevention and control strategies of coal burst in different types of deep mining are put forward. In addition, the prevention and control technology system of coal burst in the Ordos deep mining area is established. The field engineering practice has been carried out to realize the efficient prevention and control of coal burst.

Keywords: coal burst; Ordos deep mining; typical type; stress disturbance; atypical type

1. Introduction

With the development of coal mining in China, more and more coal mines in western China have been subjected to coal burst, one of the most important disasters threatening safe production [1–4]. At present, the mining depth of coal mines in China has increased by 10~25 m every year. It can be predicted that the number of coal bursts in mines in China will continue to increase in the next 10 years [5,6]. The Ordos deep mining area is the largest coal production base in China, and coal bursts have occurred more than 10 times; how to effectively prevent and control coal bursts in this area will become the focus of scientific research on coal mine safety. Deep mining, compared to shallow mining, is characterized by high confining pressure, high temperature, and high pore pressure. A large number of studies show that when the coal seam reaches a certain depth, the mechanical properties of coal will change significantly [7]. It is mainly manifested in two aspects: firstly, the constitutive relationship changes from linear elasticity in the shallow part to visco-elastic nonlinearity in the deep part; and secondly, the stress release mode of coal and rock mass changes from shallow linear to deep non-linear. As a result, it is difficult to eliminate

the high-level stress in the coal and rock mass by a one-time pressure relief in the deep mining process. In addition, it is necessary to release the visco-elastic deformation energy by multiple rounds of, or even continuous, pressure relief. Especially under the influence of dynamic load superposition caused by the structural instability of hard rock strata and fault activation, a coal burst is easily induced. Therefore, the prevention and control of coal bursts in the Ordos deep mining area is much more difficult, and it is urgent to put forward targeted prevention and control technology according to its characteristics.

In view of deep coal mining, scholars at home and abroad have carried out a lot of research on the mechanical characteristics of coal and rock mass, the in situ stress field, and the mining-induced stress field. Combined with nonlinear theory, fractal theory, black box theory, and damage mechanics theory, a variety of coal burst mechanism models have been proposed, and a method has been suggested to prevent and control coal burst by changing the mechanical properties of coal and rock mass, and reducing stress concentration [8–19]. With the increase of the number of coal burst mines in the Ordos deep mining area and the shift of research focus to prevention and control, how to formulate efficient and reasonable prevention and control schemes for coal burst based on the characteristics of deep mining and guiding field prevention and control has become an urgent problem to be solved. For a more scientific prevention and control of coal burst in the Ordos deep mining area, the concept of typical and atypical coal bursts is put forward. Based on the influencing factors of coal bursts in the Ordos deep mining area, the prevention and control strategies for different types of coal bursts in the Ordos deep mining area are proposed, and the field engineering practice is carried out. The research results are expected to provide theoretical and technical support for coal burst prevention and control in this area.

2. Classification of Coal Burst in Ordos Deep Mining Area

2.1. Analysis of Stress State in Coal Mining

The coal mining engineering area in plate tectonics and mine engineering geological conditions are controlled by tectonic movement. Tectonic stress is produced by plate collision and extrusion, which will be transmitted to the secondary plate and tectonic block through the coal and rock mass. Tectonic stress plays a controlling role in the stress environment of coal mining. The redistribution of stress in the coal and rock mass is caused by tectonic movement to achieve a new equilibrium. The results of stress distribution in the coal and rock mass can be divided into the following four cases:

- (1) Region of stress reduction. The stress cannot reach the condition of coal and rock mass failure, resulting in the crust being in a stable state. When coal mining and other engineering activities are carried out, they will not cause dynamic phenomena that would affect human engineering activities, and it is safe.
- (2) Region of simple increased stress. The stress increases, but does not reach the critical condition for the failure of coal and rock mass, so it is in a relatively stable state. If coal mining activities and other engineering activities are carried out, it is necessary to take certain prevention and treatment measures to ensure safety.
- (3) Region of increased stress. The stress rises to a critical state, and if coal mining activities and other engineering activities are carried out, coal mine dynamic disasters such as coal burst may occur. It is necessary to take effective prevention and treatment measures to ensure safety.
- (4) Region of stress release. The stress level exceeds the failure limit of the coal and rock mass, which is not directly related to human engineering activities. There will be strong tectonic activity such as earthquakes, tsunamis, and volcanic eruptions, and stress will be released to achieve equilibrium.

When coal mines enter deep conditions, the stress state of the coal and rock mass is usually in state (2) and state (3). Therefore, according to the stress state of the coal and rock mass in the Ordos deep mining area, coal burst can be divided into two types, that is, typical coal burst and atypical coal burst.

2.2. Typical Coal Burst in Ordos Deep Mining Area

Typical coal burst in the Ordos deep mining area refers to the stress state of coal and rock mass being in state (3). That is, the stress in the coal and rock mass has reached the critical instability state. When the coal and rock mass are affected by mining engineering, the critical condition of coal burst can be exceeded. Typical coal burst in the Ordos deep mining area occurrence conditions can be expressed as follows:

$$\sigma_1 + \sigma_2 \geq \sigma_0 \quad (1)$$

where σ_1 is the original rock stress, σ_2 is the mining-induced stress, and σ_0 is the critical strength of coal and rock mass.

According to the above definition and analysis, there are two main sources of stress in typical coal burst mines, that is, high in situ stress and high concentrated mining-induced stress. The occurrence mechanism of typical coal burst in the Ordos deep mining area can be represented by Figure 1. Considering the characteristics of typical coal burst, the key point of the prevention and control is pressure relief, and its purpose is to slow down the rate of stress release.

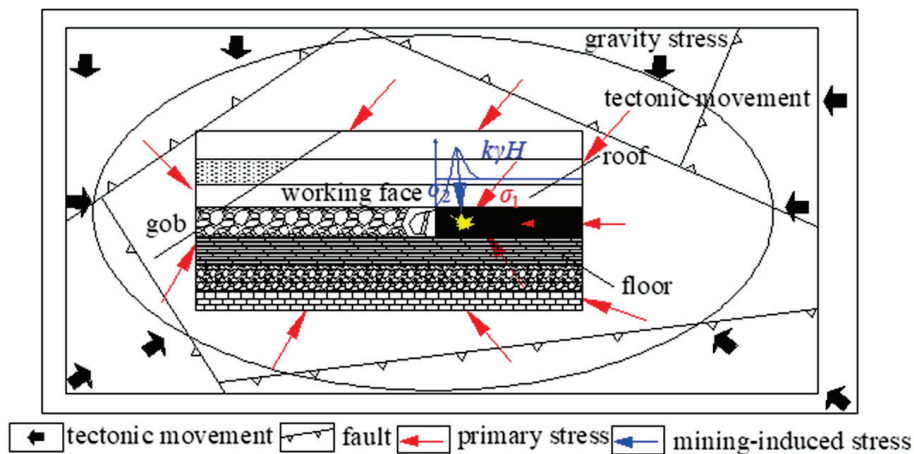


Figure 1. Schematic diagram of the mechanism of typical coal burst in the Ordos deep mining area.

2.3. Atypical Coal Burst in Ordos Deep Mining Area

Atypical coal burst in the Ordos deep mining area refers to the stress state of coal and rock mass being in state (2). That is, under mining disturbance, the stress of the coal and rock mass does not reach the critical state. When the coal and rock masses are subjected to external disturbances, such as roof weighting, structural instability of hard rock strata and fault slip, the stress in the coal and rock masses exceeds the critical condition. Atypical coal burst in the Ordos deep mining area occurrence conditions can be expressed as follows:

$$\sigma_1 + \sigma_2 + \sigma_3 \geq \sigma_0 \quad (2)$$

where σ_1 is the original rock stress, σ_2 is the mining-induced stress, σ_3 is the external disturbance stress, and σ_0 is the critical strength of coal and rock system.

According to the above definition and analysis, there are three main sources of stress in typical coal burst mines, that is, high in situ stress, high concentrated mining-induced stress and external disturbance stress. The occurrence mechanism of atypical coal burst in the Ordos deep mining area can be represented by Figure 2.

According to the characteristics of typical coal burst, the key point of the prevention and control is pressure relief, and its purpose is to slow down the rate of stress release, reduce external disturbances and weaken stress transmission capacity.

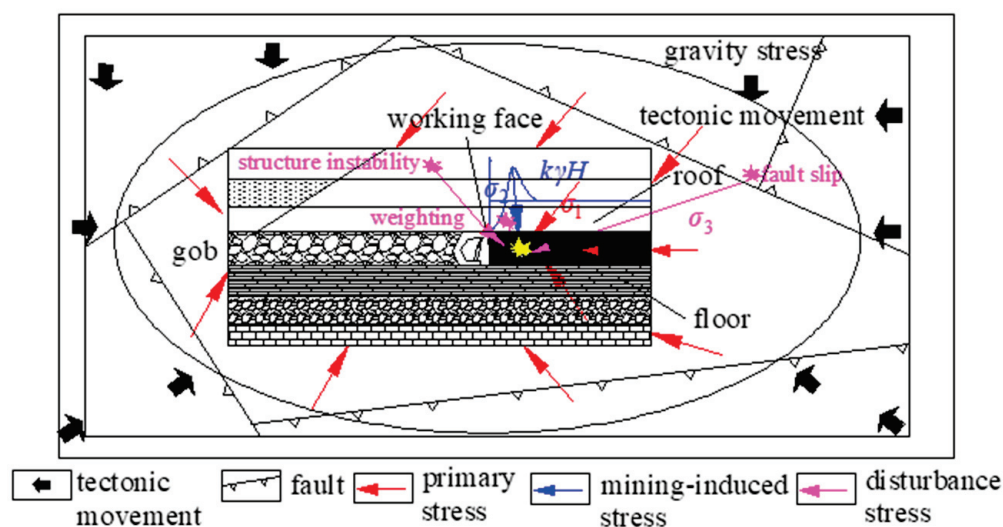


Figure 2. Schematic diagram of the mechanism of atypical coal burst in the Ordos deep mining area.

3. Analysis of Influencing Factors of Coal Burst in Ordos Deep Mining Area

3.1. Regional Geological Structure of Ordos Basin

Ordos Basin is located at the junction of the eastern and western tectonic domains in China. It is a superimposed basin controlled by multistage tectonic and sedimentary evolution cycles. The internal structure of the basin is simple, the stratum is gentle, and the activity is weak, which is mainly manifested in the structural forms of uplift, depression, wide and gentle fold, etc. According to the current structural development morphology and space-time distribution characteristics of the Ordos Basin, the basin includes a secondary structural unit in the Weihe Graben and six tertiary structural units in the western margin thrust zone, the eastern margin western Shanxi flexure zone, Yimeng uplift, Tianhuan depression, northern Shaanxi monocline, Weibei uplift and Weihe Graben.

3.2. Primary Stress Characteristics of Ordos Deep Mining Area

In order to analyze the difference of primary stress characteristics between deep mining and shallow mining in Ordos, the in situ stress data of 9 shallow mines in Ordos are obtained through field measurement and investigation [20]. The results are shown in Table 1. The depth of the 9 groups of data is between 76 m–375 m. The above 9 coal mines all belong to the same in situ stress field ($\sigma_H > \sigma_v > \sigma_h$), which is a typical stress field dominated by tectonic stress. In the same way, the in situ stress data of 11 deep mines in Ordos are obtained through field measurement and investigation [20]. The results are shown in Table 2.

Table 1. Measured data of in situ stress in the Ordos shallow mining area.

No	Name of Mine	Depth (m)	σ_H (MPa)	σ_h (MPa)	σ_v (MPa)
1	Buertai Coal Mine	375	9.34	8.72	9.94
2	Baode	347	9.84	8.19	9.20
3	Bulianta Coal Mine	230	6.57	5.15	5.76
4	Suancigou Coal Mine	303	7.72	4.06	7.60
5	Baoshan Coal Mine	112	4.06	2.08	2.81
6	Dadi Coal Mine	76	4.26	2.48	1.90
7	Talahao Coal Mine	167	6.69	2.52	3.48
8	Bojianghaizi Coal Mine	370	16.60	9.80	10.00
9	Huoluowan Coal Mine	165	4.62	2.20	3.60

Table 2. Measured data of in situ stress in the Ordos deep mining area.

No	Name of Mine	Depth (m)	σ_H (MPa)	σ_h (MPa)	σ_v (MPa)
1	Hongqinghe Coal Mine	719	27.87	16.61	16.39
2	Menkeqing Coal Mine	720	24.56	15.76	18.20
3	Shilawusu Coal Mine	660	27.16	10.91	16.24
4	Hulusu Coal Mine	650	25.74	9.39	15.49
5	Bayangaole Coal Mine	635	27.86	15.87	17.38
6	Muduchaideng Coal Mine	660	15.71	7.73	13.92
7	Nalinhe No.2 Coal Mine	550	19.22	14.30	13.75
8	Yingpanhao Coal Mine	720	25.50	20.30	18.36
9	Tarangaole Coal Mine	600	17.15	6.40	14.35
10	Hongqingliang Coal Mine	500	20.30	11.57	12.05
11	Changcheng No.2 Coal Mine	738	24.50	10.24	15.50

The depth of the 11 groups' data is between 500 m~738 m. The above 11 coal mines all belong to the same in situ stress field ($\sigma_H > \sigma_v > \sigma_h$), which is a typical stress field dominated by tectonic stress. The ratio of stress is an important index for analyzing the stress characteristics of in situ stress.

$$\begin{aligned} k_1 &= \sigma_H / \sigma_v \\ k_2 &= \sigma_h / \sigma_v \\ k_3 &= \sigma_H / \sigma_h \end{aligned} \quad (3)$$

where σ_H is maximum horizontal stress, σ_h is minimum horizontal stress, and σ_v is vertical.

The calculation results are shown in Table 3.

Table 3. Statistical table of in situ stress characteristic values in the Ordos deep mining area.

No	Name of Mine	Depth (m)	k_1	k_2	k_3
1	Hongqinghe Coal Mine	719	1.70	1.01	1.68
2	Menkeqing Coal Mine	720	1.35	0.32	1.56
3	Shilawusu Coal Mine	660	1.67	0.67	2.49
4	Hulusu Coal Mine	650	1.66	0.61	2.74
5	Bayangaole Coal Mine	635	1.60	0.91	1.76
6	Muduchaideng Coal Mine	660	1.13	0.56	2.03
7	Nalinhe No.2 Coal Mine	550	1.40	1.04	1.34
8	Yingpanhao Coal Mine	720	1.39	1.11	1.26
9	Tarangaole Coal Mine	600	1.20	0.45	2.68
10	Hongqingliang Coal Mine	500	1.68	0.96	1.75
11	Changcheng No.2 Coal Mine	738	1.58	0.66	2.39

It indicates that the minimum value of k_1 is 1.13, and the maximum value is 1.70. There are 5 coal mines with $1 \leq k_1 < 1.5$, and 6 coal mines with $1.5 \leq k_1 < 2$. By comparison and analysis of the measured data, k_1 is between 0.94 and 2.24. The value of k_1 in the shallow coal mine is smaller than that measured in the deep coal mine. Therefore, with the increase of the depth, the k_1 approximately shows an increasing trend. It indicates that the maximum horizontal stress in deep coal mine is stronger than that in shallow coal mine.

The minimum value of k_2 is 0.32, and the maximum value is 1.11. There are 2 coal mines with $k_2 < 0.5$, 6 coal mines with $0.5 \leq k_2 < 1$, and 3 coal mines with $1 \leq k_2 < 1.5$. Through comprehensive analysis and comparison, the k_2 in the shallow area is between 0.53 and 1.31, which is obviously lower than that in the deep area. Therefore, with the increase of the depth, the k_2 approximately shows a decreasing trend. It points out that the difference between the minimum horizontal stress and vertical stress in the deep coal mine is obviously larger than that in the shallow coal mine.

In the measured data of 11 mines, the minimum value of k_3 is 1.26, and the maximum value is 2.74. There are 2 coal mines with $k_3 < 1.5$, 4 coal mines with $1.5 \leq k_3 < 2$, 3 coal

mines with $2 \leq k_3 < 2.5$ and 2 coal mines with $k_3 \geq 2.5$. Based on the measured data, the measured value of k_3 in the shallow area is between 1.07 and 2.65, which is obviously lower than that in deep area. Therefore, with the depth rises, the value of k_3 obviously shows an increasing trend. To sum up, with mining depth increasing, the difference of coal and rock mass triaxial stress state becomes larger and larger. According to the Moore–Coulomb criterion [15], the larger the difference of the triaxial stress state, the more likely failure and instability will occur.

Therefore, when a coal mine in Ordos enters deep mining, the level of primary stress is higher than that in a shallow area, and the anisotropy is more obvious. As a result, the stress level of coal and rock mass accumulation is higher, and instability failure is more likely to occur. It provides foundation stress conditions for the occurrence of coal burst.

3.3. Mining-Induced Stress Characteristics of Ordos Deep Mining Area

In order to analyze the characteristics of mining-induced stress, a FLAC3D numerical model is established [21]. The characteristics of mining-induced stress of the same model with different depths are analyzed. According to the average of bulk density with 25 kN/m^3 , the vertical stress at different depths is calculated. Specific values of the triaxial stress are shown in Table 4.

Table 4. Numerical simulation of in situ stress parameters in mines with different depths.

No	Depth (m)	σ_H (MPa)	σ_h (MPa)	σ_v (MPa)
1	400	15.00	7.50	10.00
2	600	22.50	11.25	15.00
3	800	30.00	15.00	20.00
4	1000	37.50	18.75	25.00

A total of 240,000 units and 251,991 nodes has been established, and the size of numerical model is length of 300 m, width of 400 m and height of 50 m. The top of the model is free boundary, and the others are fixed boundary. The primary stress field is applied according to the data in Table 2, and equivalent load is applied to simulate the upper part of the model. The numerical model is shown in Figure 3.

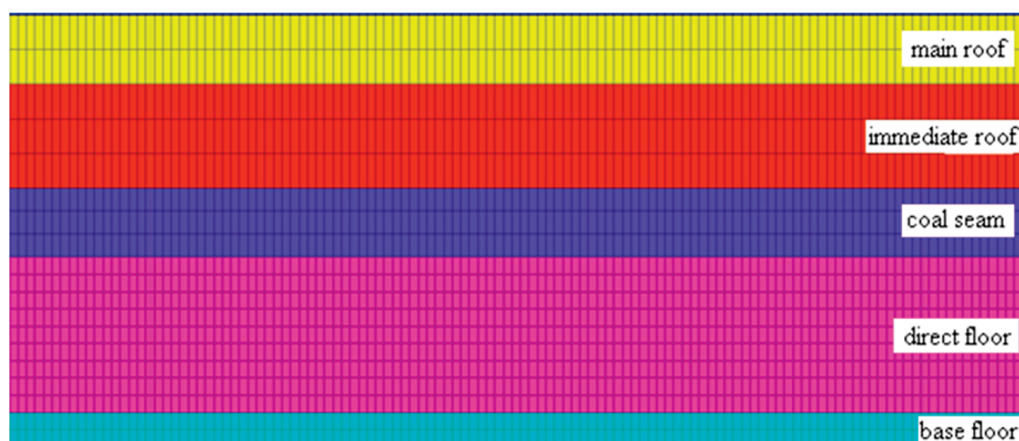


Figure 3. Numerical model of mining at different depths.

However, in the model, the length of the working face is 200 m and the thickness of the coal seam is 5 m. After the panel is mined 200 m, the vertical stress data within 100 m advance of the panel is extracted (shown in Figure 4).

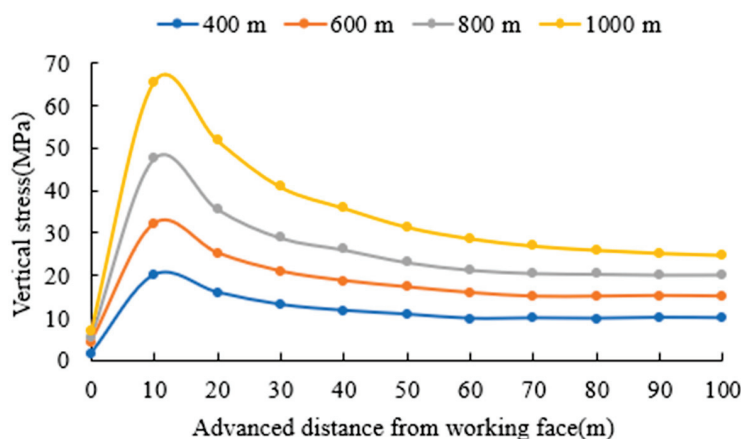


Figure 4. Advance stress distribution curves of mining at different depths.

As can be seen from the change in Figure 4, the influence range of concentrated stress is about 60 m for mining at depth of 400 m, 70 m for 600 m depth, 70 m for 800 m depth and 90 m for 1000 m depth. The influence range of concentrated stress shows an upward trend with the increase of mining depth. At 400 m depth, the concentrated stress peak is about 20.01 MPa, and the stress concentration coefficient is 2.01. At 600 m depth, the peak is about 32.11 MPa, and the coefficient is 2.14. At 800 m depth, the peak is about 47.42 MPa, and the coefficient is 2.37. At 1000 m depth, the peak is about 65.53 MPa, and the coefficient is 2.62. The degree of stress concentration increases with mining depth increasing. Therefore, when coal mines enter deep area, the disturbance of mining-induced stress is higher than that in shallow area. Both the degree and influence range of concentrated stress increase obviously, which provides the conditions of strong mining-induced stress for the occurrence of coal bursts.

3.4. External Disturbance Stress Characteristics of Ordos Deep Mining Area

External disturbances mainly include periodic weighting, overburden structure instability, and fault slip. Compared with shallow mining, deep mining has a higher level of basic stress and mining-induced stress. The external disturbance is more intense in deep mining, which is more likely to induce mine dynamic disasters such as coal burst. Taking the fault as an example, the numerical simulation method is adopted to analyze the external disturbance stress in deep mining. A total of 452,560 units and 462,429 nodes are established, and the size of the numerical model is length $\times$ width $\times$ height of 950 m $\times$ 400 m $\times$ 1000 m. Two faults are included in this numerical model, named as F25 and F18 (shown in Figure 5)

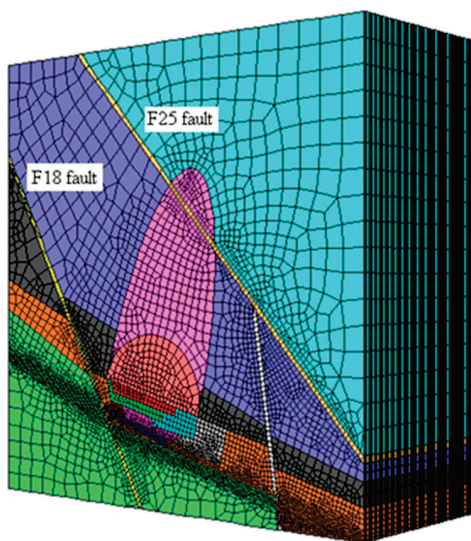


Figure 5. Numerical model including faults at different mining depths.

In this model, the primary stress at different mining depths is still assigned according to Table 4, and the length of the simulated working face is set as 160 m. When the panel is mined 200 m, the vertical stress at the point 10 m ahead of working face is extracted and the curve is drawn, as shown in Figure 6. At 400 m depth, the stress peak of F25 fault is 16.82 MPa with concentration coefficient of 1.68 and the influence range is about 50 m, the peak of F18 fault is 21.12 MPa with coefficient of 2.12 and the range is about 60 m. At 600 m depth, the peak of F25 fault is 28.81 MPa with coefficient of 1.92 and the range is about 70 m, the peak of F18 fault is 31.90 MPa with coefficient of 2.13 and the range is about 90 m. At 800 m depth, the peak of F25 fault is 41.78 MPa with coefficient of 2.09 and the range is about 70 m, the peak of F18 fault is 42.42 MPa with coefficient of 2.12 and the range is about 90 m. At 1000 m depth, the peak of F25 fault is 54.54 MPa with coefficient of 2.18 and the range is about 80 m, the peak of F18 fault is 56.37 MPa with coefficient of 2.25 and the range is about 100 m. With the increase of mining depth, the degree of fault stress concentration and the influence area exhibit an obvious increasing trend. Therefore, the deep mining fault is more likely to produce slip vibration, and the influence range of stress concentration is larger, and the disturbance to mining engineering is more intense.

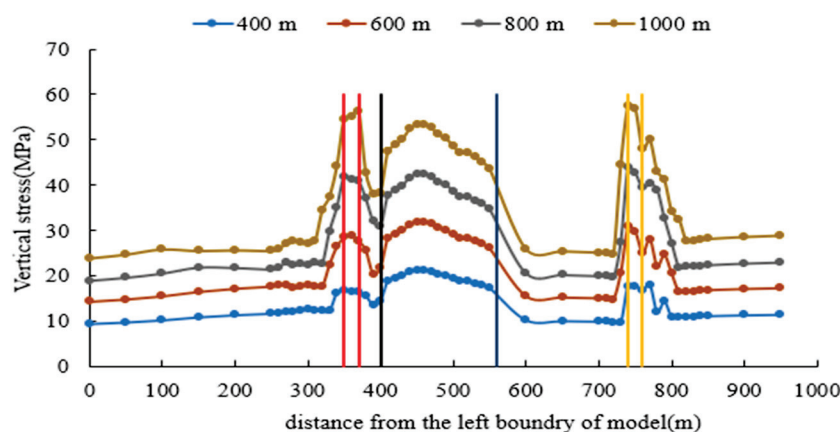


Figure 6. Vertical stress distribution curves of faults at different mining depths.

According to the above analysis, the reasons why coal mines in the Ordos deep mining area are more prone to coal bursts can be summarized as follows: (1) The level of primary stress in the Ordos deep mining area is higher than that in the shallow area, and the difference of the triaxial stress of the coal and rock mass is larger. The stress level accumulated in the coal and rock mass is higher. All these factors have promoted the occurrence of instability disasters in the coal and rock mass. (2) The influence of mining-induced stress is stronger in the Ordos deep mining area. The degree and influence range of concentrated stress obviously increase, which aggravate the accumulation of stress in the coal and rock mass. (3) Deep mining in Ordos is more prone to instability caused by external disturbance, and the influence scope and degree of faults and other structures are larger. It is more likely to lead to the instability of coal and rock mass.

4. Practice of Coal Burst Prevention and Control in Ordos Deep Mining Area

4.1. Prevention and Control Strategy of Rock Burst in Ordos Deep Mining Area

According to the two types of coal burst in the Ordos deep mining area, two corresponding strategies of prevention and control are proposed. Referring to the above research, the necessary conditions for the occurrence of typical coal bursts are high in situ stress and high concentrated mining-induced stress. Additionally, the necessary conditions for the occurrence of atypical coal bursts are high in situ stress, high concentrated mining-induced stress and external disturbance stress. Therefore, coal burst prevention and control strategies of the Ordos deep mining area are suggested as follows: Considering the typical coal burst in the Ordos deep mining area, coal burst prevention and control should avoid stress accumulation in coal and rock mass, and be appropriate to the coal mass pressure relief to improve

the overlying structure. The main point is to unload the pressure of coal and improve its mechanical properties. The typical strategy is a large range of coal pressure relief, proper treatment of roof and floor, and strong support. Especially, when the depth is over 800 m, portal supporting and “O” type sheds should be used. Correspondingly, for atypical coal burst prevention and control, partial coal pressure relief, large scale of roof treatment and strengthening support are critical means. This contributes to stress release caused by period weighting or overlying structure instability. Additionally, a broken zone is formed in the roof and coal seam to increase the attenuation of external disturbance stress.

4.2. Stress Control Technology System of Coal Burst in Ordos Deep Mining Area

Based on the stress control theory in coal mines [22], the prevention and control of coal bursts in the Ordos deep mining area can be divided into three stages: roadway excavation stage, panel preparation stage, and panel mining stage. According to the key points of coal burst prevention and control in each stage, and the actual production conditions, the corresponding stress control technologies are put forward. (1) Stress control technology in roadway excavation stage: due to the particularity of coal and rock properties, and the stress environment in the Ordos deep mining area, the roadway surrounding rock can show obvious flow deformation during roadway excavation. It is suggested to use large-diameter borehole pressure relief or unloading blasting technology to relieve the pressure of coal seams. When the coal strength is high, unloading blasting technology is preferred. (2) Stress control technology in panel preparation stage: this stage is implemented after the end of roadway excavation, before the panel mining. Due to the disturbance of the roadway to the coal and rock mass in the range of stope, especially in the roadway, which is closed by mining gob, it is easy to produce a highly concentrated stress area. Thus, in this period, according to the type of coal burst in the Ordos deep mining area, it is recommended to carry out pressure relief and pre-cracking measures for coal seam and roof, respectively. In addition, large-diameter borehole pressure relief and unloading blasting technology can be used to relieve pressure in the high-concentrated stress area on both ribs. Both of these ways are good to reduce the disturbance of coal mining. Additionally, it is proposed to use hydraulic fracturing, deep hole blasting and ground hydraulic fracturing to treat the overlying strata. (3) Stress control technology in panel mining stage: when the panel is mined, the effect from mining is more intense than that of roadway excavation, and the external disturbance such as roof weighting and overlying structure instability is brought. It is necessary to carry out the second round of pressure relief. The support system shall be strengthened in high-risk areas such as advanced support to further improve the safety of coal burst. It is a good proposal that large-diameter borehole pressure relief and unloading blasting technology need to be adopted to achieve the second round of pressure relief in the area of high concentrated stress. Additionally, “O” type shed, “U” type shed and strong hydraulic support should be adopted to strengthen the roadway stability.

4.3. Prevention and Control Practice of Typical Coal Burst in Ordos Deep Mining Area

Hongqingliang Coal Mine is located in the north of Yimeng uplift. The layout plan of Hongqingliang Coal Mine is shown in Figure 7. The depth of mines No.3-1 coal seam in this area is 458~538 m. The thickness of that is 2.86–6.85 m, with an average of 4.87 m. The structure of that is simple, most of which contain one layer of gangue, and some including two layers of 0.15~0.62 m gangue. The inclination angle of the coal seam is 0~6°, with an average of 3°. The lithology of the roof is mainly sandy mudstone and siltstone, and some are fine grained sandstone, conglomerate or coarse-grained sandstone. The lithology of the floor is mainly sandy mudstone and mudstone. The overlying strata of Hongqingliang Coal Mine belong to weakly consolidated strata and contain a lot of clay minerals. The strength of coal seams is high and can reach 14 MPa locally. The width of coal pillars between 2 panels is 20~25 m.

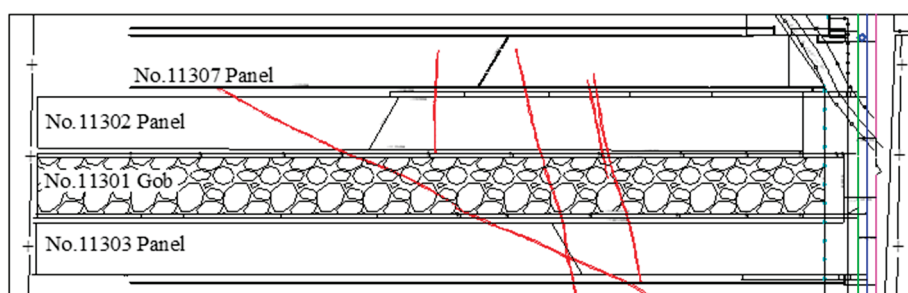


Figure 7. Hongqingliang coal mine excavation project plan.

Under the superposition of primary stress and mining-induced stress, the panel is easy to destabilize and lead to coal burst. Therefore, it belongs to typical coal burst. Therefore, Hongqingliang Coal Mine has formulated the three-in-one prevention and control measures of borehole pressure relief, high-strength bolt support, and advanced hydraulic support. The specific parameters of the measurements are as follows. (1) Roadway excavation stage: No.11302 Panel mainly adopts advanced borehole pressure relief technology. The diameter of pressure relief bore hole is 110 mm, the borehole depth is 20 m. The boreholes are arranged in a single row, with 1.0~1.5 m away from the floor. The plan and section of the pressure relief borehole are shown in Figure 8. (2) Panel preparation stage: No.11302 Panel mainly adopts strong bolt (cable) to further strengthen roadway support and control roadway deformation. (3) Panel mining stage: No.11302 Panel mainly adopts large diameter borehole pressure relief and advanced unit hydraulic support technology. The diameter of the pressure relief borehole is 110 mm, and the spacing is 3 m. The borehole depth in the coal pillar rib is 15 m. Another rib is 20 m, and the height from the floor plate is 1.5 m. The support distance of unit hydraulic support is 100~120 m. The plan and section of the pressure relief borehole are shown in Figure 9.

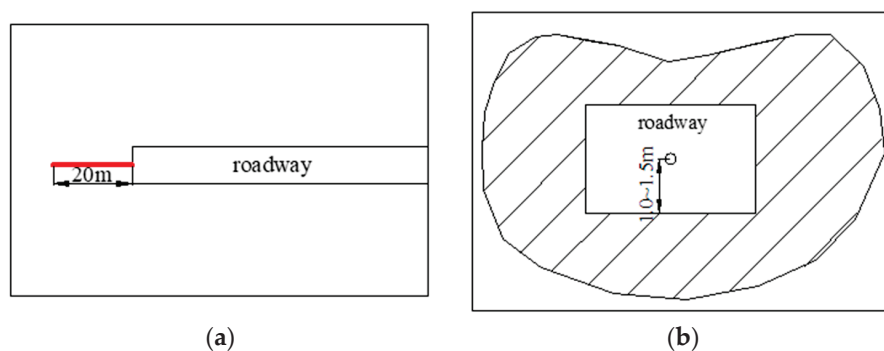


Figure 8. Plan and section of the pressure relief borehole in roadway excavation stage of No.11302 Panel. (a) Plan. (b) Section.

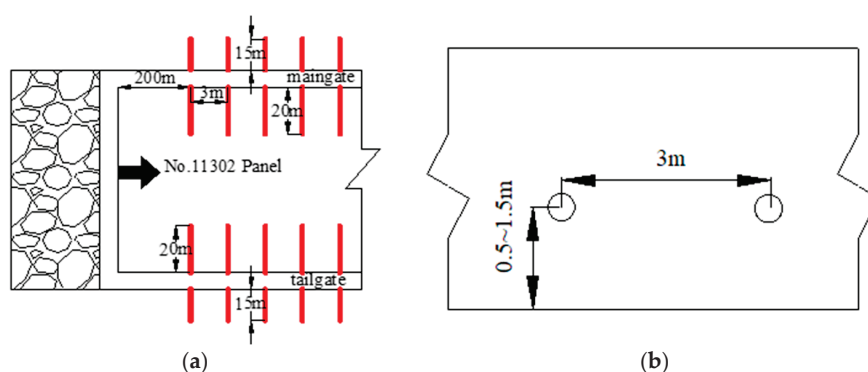


Figure 9. Plan and section of the pressure relief borehole in panel mining stage of No.11302 Panel. (a) Plan. (b) Section.

The maximum mining speed of No.11302 Panel is 12 m/day. Six months after the implementation of prevention and control measures, the microseismic monitoring event plan of No.11302 Panel is shown in Figure 10. In the microseismic system, green points means microseismic event energy $\leq 10^3$ J, yellow points means microseismic event energy $\leq 10^4$ J, and red points means microseismic event energy $> 10^5$ J. The microseismic monitoring results show that after a series of prevention and control measures in this mining area, the overall pressure of No.11302 Panel in Hongqingliang Coal mine is stable, and the energy level of microseismic events is mainly between $10^3 \sim 10^4$ J, and no high energy event above 10^5 J appears.

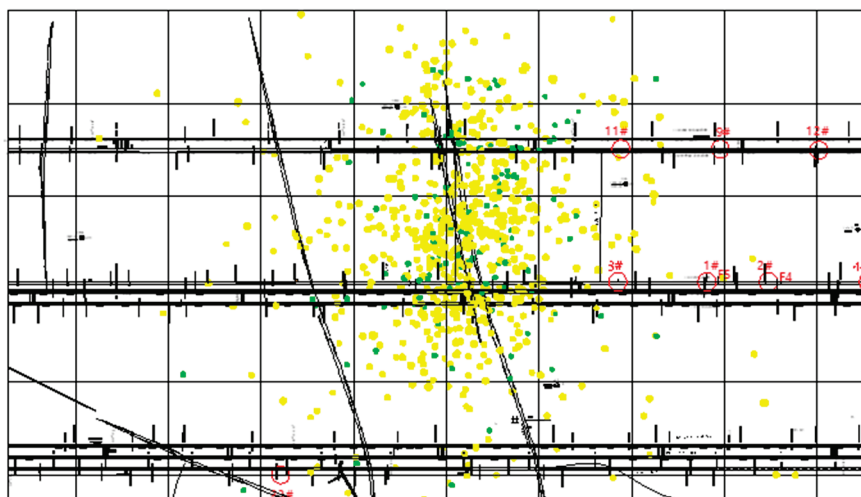


Figure 10. Microseismic event distribution of No.11302 Panel.

4.4. Prevention and Control Practice of Atypical Coal Burst in Ordos Deep Mining Area

Shilawusu Coal Mine is located in the south of the junction of Yimeng uplift and northern Shaanxi slope. Shilawusu Coal Mine mines exploits No.2-2 coal seam, with an average depth of 690 m. The immediate roof of #No.2-2 coal seam is sandstone or siltstone strata with a thickness that exceeds 10 m, and its uniaxial compressive strength is greater than 60 MPa. The width of coal pillars between 2 panels is 5~6 m. The layout plan of Shilawusu Coal Mine is shown in Figure 11.

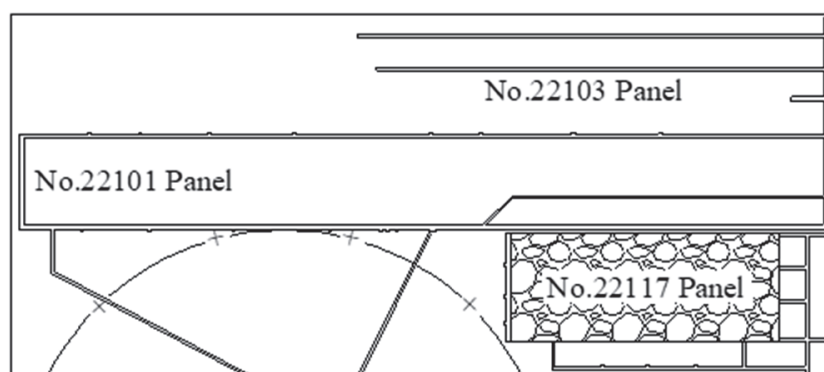


Figure 11. Shilawusu coal mine excavation project plan.

Shilawusu Coal Mine has high stress characteristics under the influence of North Shaanxi Slope. A coal burst is easily induced by the disturbance of hard rock strata instability. It belongs to atypical coal burst. Therefore, Shilawusu Coal mine has formulated the three-in-one prevention and control measures of roof hydraulic fracturing, borehole pressure relief and advanced hydraulic support. The specific parameters of the measures are as follows:

- (1) Roadway excavation stage: No.22101 Panel mainly adopts advanced borehole pressure relief technology. The diameter of pressure relief bore hole is 150 mm with the spacing of 2 m. The borehole depth is 20 m, which is arranged in a single row, and the borehole is 1.0~1.5 m away from the floor. The plan and section of the pressure relief borehole are shown in Figure 12.
- (2) Panel preparation stage: No.22101 Panel mainly adopts a single rib of large diameter borehole pressure relief and roof hydraulic fracturing technology. The diameter of the pressure relief borehole is 150 mm, and the spacing is 1 m. The borehole depth is 20 m, and the height from the floor plate is 1.2~1.5 m. The diameter of the hydraulic fracturing borehole is 42 mm. The fracturing position is the immediate roof with a thickness of 9.31 m, and the lithology is fine-grained sandstone. Additionally, another fracturing position is 19.84 m away from the No.2-2 coal seam, and the lithology is medium-grained sandstone with a thickness of 10.44 m. The plan and section of the pressure relief borehole and the hydraulic fracturing borehole are shown in Figures 13 and 14.
- (3) Panel mining stage: No.22101 Panel mainly adopts large diameter borehole pressure relief and advanced unit hydraulic support technology. The diameter of the pressure relief borehole is 150 mm and the spacing is 1 m. The borehole depth is 20 m, and the height from the floor plate is 1.5 m. The support distance of the unit hydraulic support is 120~150 m. The plan and section of the pressure relief borehole are shown in Figure 15.

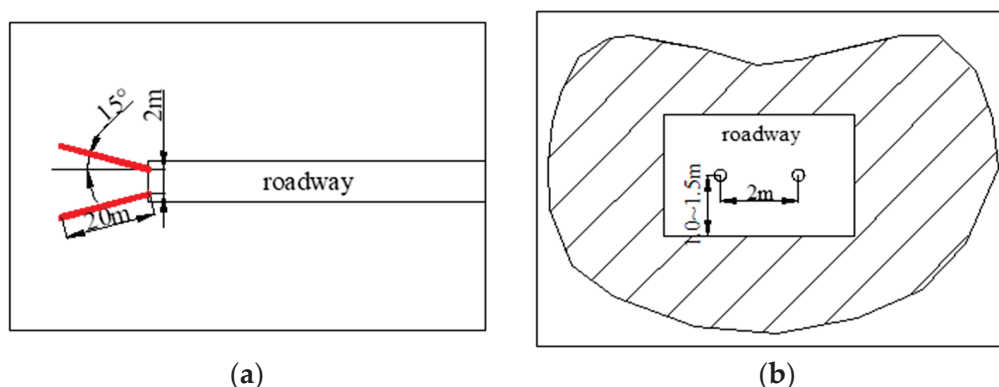


Figure 12. Plan and section of the pressure relief borehole in roadway excavation stage of No.22101 Panel. (a) Plan. (b) Section.

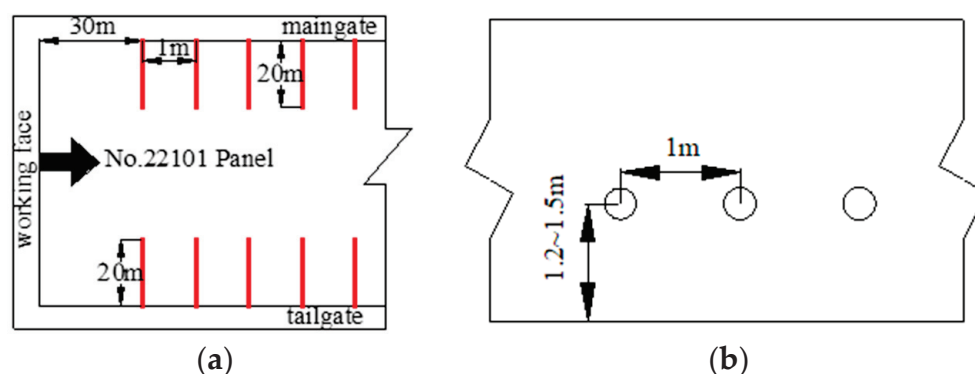


Figure 13. Plan and section of the pressure relief borehole in panel preparation stage of No.22101 Panel. (a) Plan. (b) Section.

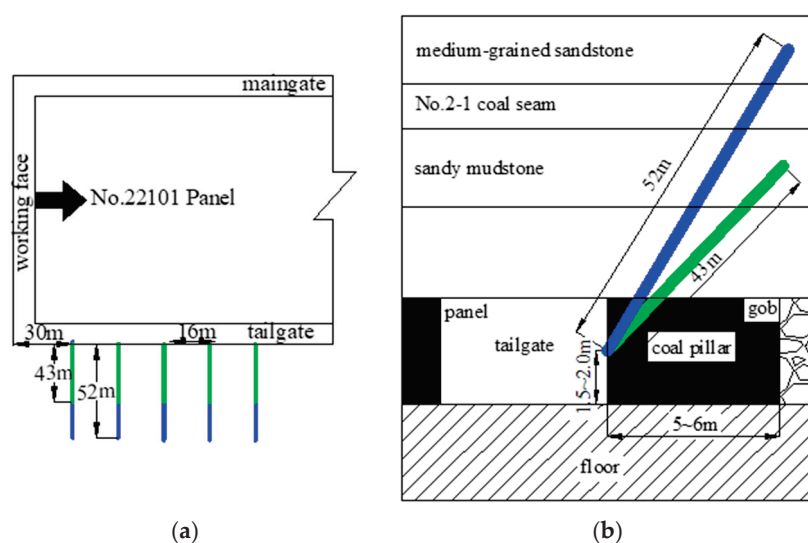


Figure 14. Plan and section of the hydraulic fracturing borehole in panel preparation stage of No.22101 Panel. (a) Plan. (b) Section.

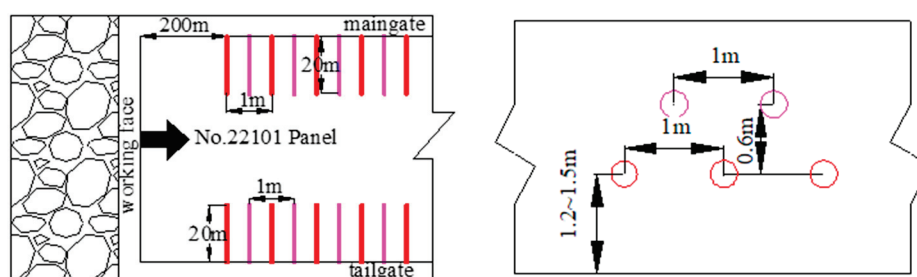


Figure 15. Plan and section of the pressure relief borehole in panel mining stage of No.22101 Panel.

The maximum mining speed of No.22101 Panel is 8 m/day. Through the implementation of roof hydraulic fracturing, the first weighting step distance of No.22101 Panel is 21.7 m, and the average periodic weighting step distance is 12.7 m. However, the first weighting step distance of the panel without hydraulic fracturing is about 60.5 m, and the periodic weighting step distance is 25~30 m. Obviously, the roof hydraulic fracturing technology effectively reduces disturbance of roof weighting to the panel. The microseismic monitoring data during the first weighting of No.22101 Panel are shown in Figure 16.

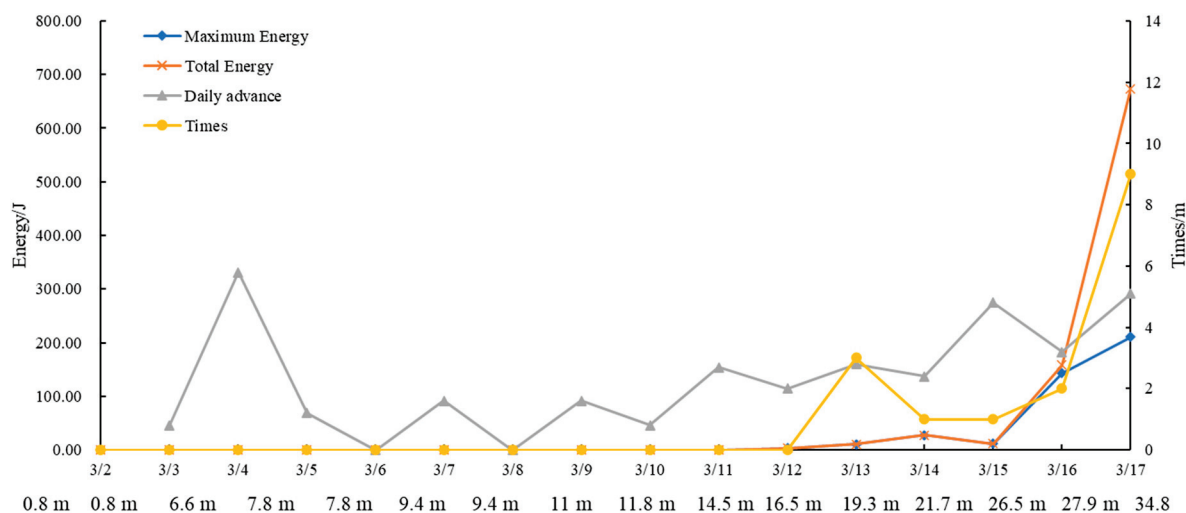


Figure 16. Microseismic data during the first roof weighting.

5. Conclusions

The prevention and control of coal burst in the Ordos deep mining area are systematically studied by using theoretical analysis, numerical simulation and field measurement methods. The following five conclusions are drawn:

- (1) According to the stress state and load form of coal and rock mass in the Ordos deep mining area, the coal burst is divided into two types, that is, typical coal burst and atypical coal burst.
- (2) The necessary conditions for the occurrence of typical coal burst are high in situ stress and high concentrated mining-induced stress. Additionally, the necessary conditions for the occurrence of atypical coal burst are high in situ stress, high concentrated mining-induced stress and external disturbance stress.
- (3) The level of primary stress in the Ordos deep mining area is higher than that in shallow areas. The influence of mining-induced stress is stronger in the Ordos deep mining area. Deep mining in Ordos is more prone to instability caused by external disturbance. The above three conditions are the fundamental reasons for the easy occurrence of coal bursts in the Ordos deep mining area.
- (4) Based on the control theory of coal burst, the prevention and control of coal bursts in the Ordos deep mining area can be divided into three stages: roadway excavation stage, panel preparation stage and panel mining stage. The corresponding stress control technology has been formed, and the dynamic prevention and control technology system of coal bursts in the Ordos deep mining area has been formulated.
- (5) The effective strategy of typical coal burst prevention and control is a large range of coal pressure relief, proper treatment of roof and floor, and strong support. Especially, when the depth is over 800 m, portal supporting and “O” type sheds should be used. Additionally, the practical strategy of atypical coal burst prevention and control is partial coal pressure relief, large range of roof treatment, and strengthen support.

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Article

The Impact of Surface Water Seepage on Seismicity and Rockbursting in Mines

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Abstract: Retrospective analysis of data obtained from long-term monitoring of technogenic seismicity and rockbursts at the Apatitovy Tsirk and the Rasvumchorr Plateau deposits (Russia) showed that there is a significant (by 50% or more) increase in the number of geodynamic events during spring snowmelt periods. An upswing of seismic activity within this rock massif occurs when following conditions are true: water reserve in the snow cover on the deposit area is more than $3 \times 10^8 \text{ m}^3$; snowmelt period exceeds 40 days; increase in water ingress rates continues for over 5 days and total water inflow volume exceeds the previous daily measurements by at least a factor of 2. Seismic activity of the massif starts to intensify after the snowmelt develops momentum. Major induced earthquakes occurred in the years when these conditions were met (for example, in 2005 there was a magnitude 2.3 earthquake; in 2009, $M = 1.6$ earthquake), and more than 1000 seismic events were recorded during the snowmelt period. It has been established that when mining reaches the depths of more than 500 m, seismic events during infiltration of atmospheric precipitation begin to occur from a depth of 100–200 m and are recorded to depths of about 900 m. A possible controlling factor of the seismic activation is the reactivation of tectonic faults, which occurs under conditions of the critically stressed state of the massif, due to a decrease in their normal compression during infiltration. Retrospective analysis of the factors contributing to a strong rockburst ($K = 10\text{--}11$) in 1990 at a bauxite mine in the South Urals shows that prior to this disaster there was an inrush of the Ai River waters into the mine workings through a large tectonic disturbance, which has not been previously taken into account when analyzing the mechanism of this geodynamic event. The intrusion of water into the fault located in the field of regional stresses and subsequent partial relief of its fault plane from normal stresses could have triggered the rockburst with fault-slip mechanism. The study of the relationship between amount of precipitation and the degree of water encroachment into the field, on the one hand, and seismicity, on the other hand, is needed to draw up recommendations on improving geodynamic and environmental safety of mining regions in order to ensure their sustainable development.

Keywords: rock mass watering; rockburst; seismic activity; mining-induced earthquakes; critical stress state; water inflow

1. Introduction

It is vitally important to be aware of possible geodynamic consequences of human activity, including manifestations of strong rockbursts [1,2], induced seismicity [3,4] and geodynamic activity [5,6], as this promotes better planning of sustainable development of industrial regions and ensures environmental integrity. The long-term existence of the rockburst problem indicates its complexity and multifactoriality. Different researchers continue studies in the field of integrated rockburst hazard provision [7,8], trigger mechanisms

and development of appropriate classifications [9,10]; seismological studies [11,12] and monitoring [13,14]; and studies of influencing factors [15,16] and rock behavior in zones of critical stress [17].

One of the factors that change the geomechanical state of the rock mass and affect the geodynamic events is its watering. For example, wetting of the rock mass is used as a preventative measure against rockbursts [18]. Water is considered as a lubricant that contributes to the activation of faults [19,20]. Hydraulic fracturing of rocks and fluid injection into boreholes are associated with seismicity [21,22]. The occurrence of seismicity during the flooding of the Champion Reef mine in India [23] and coal quarries in Russia in the 1990s [24] was explained by the reactivation of tectonic faults when their normal compression was reduced under the influence of hydrostatic pressure. Seismic activations during mine flooding in the deposits of South Africa [25] and Europe [26] are similarly explained.

The influence of atmospheric precipitation and surface water bodies on seismicity occurrence has been repeatedly discussed in seismology. Therefore, specialists are aware of the connection between the filling of water reservoirs and the occurrence of seismicity [27,28]. The authors of [29,30] have studied the connection between rains and weak earthquakes. However, the issue of changes in the geodynamic activity of the rock massif during its watering in running mining enterprises remains insufficiently studied.

The apatite–nepheline and rare-metal ore deposits in the Khibiny and Lovozero rock massifs, respectively, are some of the most rockburst-hazardous in Russia. They were exposed to the strongest rockbursts with magnitude $M_L = 5.1$ and mining-induced earthquakes with magnitude $M = 4.2$. Seismic events registered in the central part of the Khibiny rock massif are explained by the redistribution of stresses in the rock mass under the influence of large-scale mining operations, not only during ore extraction, but also during dumping and formation of tailings [31,32]. The relationship between the intensity of precipitation and rockburst hazard was also registered in [33]. The Urals is another area of strong rockburst occurrences with energy up to 10^{11} J. In 1990, a rockburst occurred and caused destruction to an area of about 45 hectares at the mine, which was located within 0.5–1 km from the large river Ai and was hydrogeologically connected with it.

Mining enterprises operating in difficult geomechanical conditions have to establish and trace connections between the rock mass watering and rockburst occurrence for monitoring and justification of measures to ensure geodynamic safety. The authors studied the relationship between rockbursts and rock mass watering based on the geodynamic monitoring data analysis and also on the framework of a geodynamic model of the upper part of the Earth's crust in a critically stressed state.

2. Study Site

2.1. The Apatite–Nepheline Deposits, Kola Peninsula

The apatite–nepheline deposits have been mined since the 1930s. Fifty years later, the situation with the occurrence of tectonic rockbursts has become more complicated. On April 16, 1989, a mining-induced earthquake of magnitude 4.2 occurred in the Kirovsky mine during a bulk blast; this event remains the strongest to date. The strongest rockburst ($M_L = 5.1$) at the Umbozero mine took place on August 17, 1999. The depth of the hypocenter was measured to be 300 m [34].

The Rasvumchorr mine develops the Apatitovy Tsirk deposit, and the Tsentralny open pit mine develops the Rasvumchorr Plateau deposit. Currently, the deposits are located in a zone of active mutual influence: the underground mine workings are on the walls of the open pit and underneath the open pit; the open pit dumps are located on the surface above the underground workings. Mining pressure is observed in the junction zone of the underground mine and the open pit more often than on the opposite flanks of the mine fields. The tectonic stress field is characterized by the sublatitudinal orientation of the maximum compression $\sigma_{\max}$ in the subhorizontal plane and the vertical orientation of the minimum compression $\sigma_{\min}$. The ratio $\sigma_{\max}/\sigma_{\min}$ is estimated as 2.5.

Seismological monitoring is carried out by an automated control system for rock mass conditions, registering seismic events with energy from 100 J and hypocenter location accuracy up to several meters.

The underground Rasvumchorr mine controls the junction area of the Tsentralny open pit, the ore pass area and the northwest wall of the pit. Figure 1 shows the epicenters of seismic events with energy class $K \geq 6$.

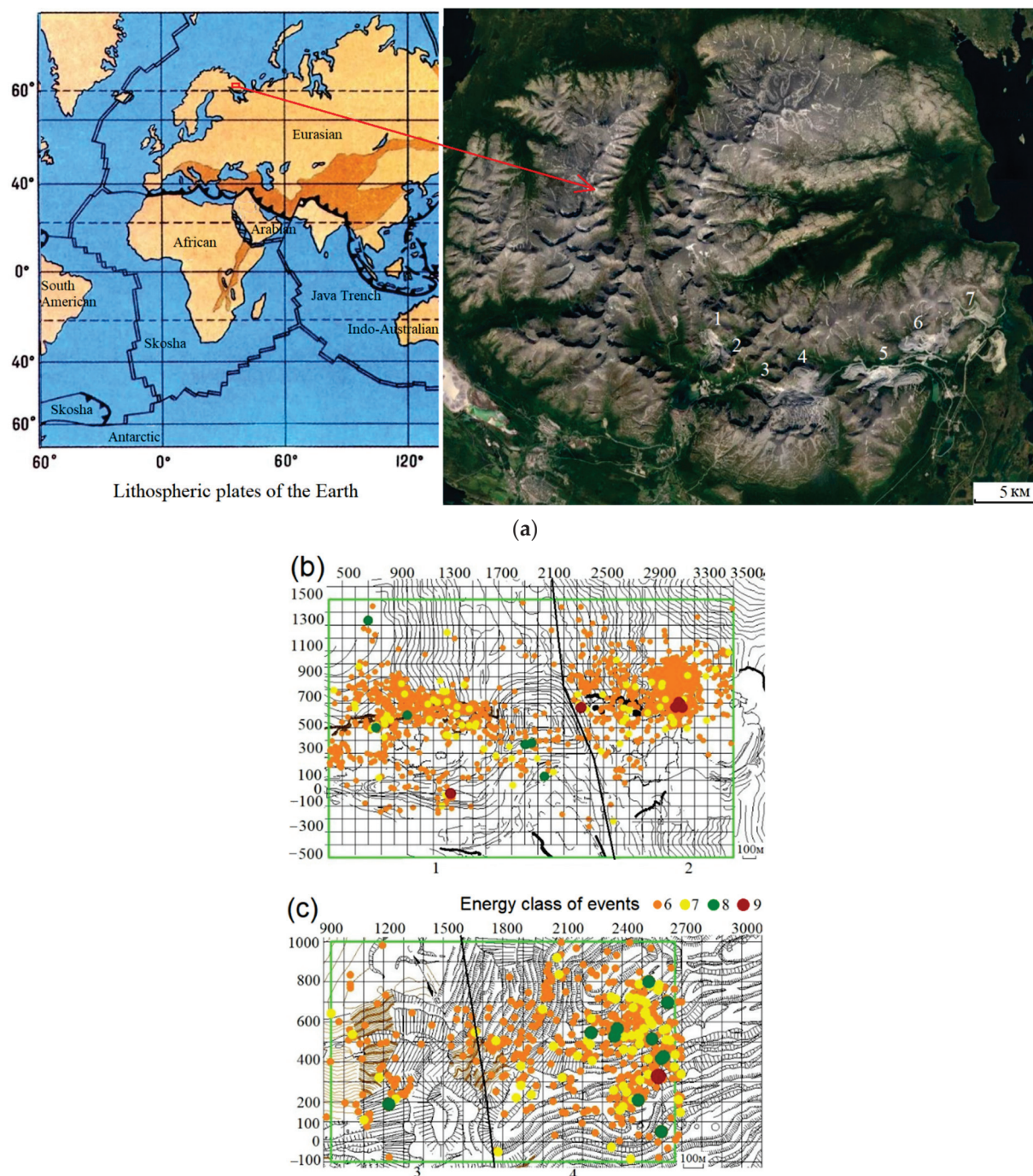


Figure 1. Distribution of epicenters of seismic events (2002–2018). (a) Lithospheric plates of the Earth (left) [35], the Khibiny rock massif (on right). Seismicity at the Kirovsky mine (b). Seismicity at the Rasvumchorr mine and Tsentralny open pit (c); vertical line divides the deposits. 1—Kukisvumchorr deposit; 2—Yukspor deposit; 3—Apatitovy Tsirk deposit; 4—Plateau Rasvumchorr deposit; 5—Koashva deposit; 6—Niorpakhk deposit; 7—Oleniy Ruchey deposit.

Since 1987, within the Kirovsky mine zone of seismological monitoring, 2500 such events were registered; since 2004 at the Rasvumchorr mine, 480 events; in the Tsentralny open pit (with an error in determining the hypocenter of more than 100 m), 300 events. Most of the events are concentrated in the area of mining operations and in the zone of influence of the underlying rock console, as well as in the zone of the junction of the underground Rasvumchorr mine and the Tsentralny open pit.

The rock mass of the junction zone and ore pass area of the Tsentralny open pit is characterized by the presence of radial faults, among which there are steeply dipping lamprophyre dikes, as well as faults in the form of spreusteinized zones that form a block structure of the rock mass. In the eastern part of the ore body, a diabase dike occurs in one of the radial faults, with which the Draznyashchee Echo Gorge and Glavny Fault [36] are connected. It is a sublatitudinal spreusteinization zone, 1 to 10 m thick, where strong seismic events (energy class $K \geq 6$) are recorded periodically in the spring and autumn periods.

The underground mine and open pit are flooded due to fractured and fractured-vein waters of the Palaeozoic aquifer complex and seepage of atmospheric precipitation through the failure zone, as well as due to groundwater rising during intensive snowmelt and prolonged rainfall.

2.2. A Bauxite Mine, Southern Ural

Underground development of the Novo-Pristanskaya group bauxite deposits began in 1952. In 1969, the Blinovo-Kamensky mine was put into operation with a production of up to 650,000 tons of ore per year; and in 1979, the Kurgazak mine was put into operation with a production of up to 450,000 tons [37]. The Kurgazak mine developed the Kurgazak bauxite deposit, presented by an ore body of plate form with a thickness of 2.5 m and in some places up to 4 m, occurring at 225–550 m. The ore body was up to 6 km long and up to 1.2 km wide, extending NE–SW with a dip of angle from 0 to 15 degrees; the host rocks were limestone. Soil limestones with a thickness of 20 m lie on clay limestones and marls with a thickness of 10–20 m, which, in turn, lie on heavily faulted clayey–sandy rocks. The depth of mining works was 340 m by 1990. The ore body was produced using the chamber-and-pillar mining method without backfill; the roof in the mined space was supported by numerous pillars ranging in size from 1.5×1.5 to 5×5 m [38].

On 28 May 1990, the mine experienced two consecutive strong rockbursts with energies of 10^{10-11} J, which destroyed pillars on an area of about 0.45 km² in the central part of the mine field. There were also soil heaving and fractures, and roof collapses in some sections of the workings. On the surface, this rockburst was noted as a strong shock; cracks appeared on the ground surface; a brick wall that protected the ventilation shaft from dust collapsed, and the plaster in an administration building cracked. The mechanism of this rockburst [38] is associated with the compression of the rock mass by horizontal forces, which led to the breaking of the soil and the destruction of the pillars. The author of [34], based on the calculation of the load on the pillars, has concluded that the rockburst was a chain reaction of pillar destruction, provoked by the crushing of the weakest of them. However, the phenomenon of the destruction of pillars by the movement of the ore body bed from bottom to top has remained unexplained.

The authors studied the stress state of the rock mass in the area of the deposit by tectonophysical methods after the rockburst of 1990. According to geomorphological features, the whole studied region is experiencing intensive recent uplift, which is explained by an excess of horizontal compression over vertical compression. The meanders of rivers deeply incised into the bedrock, for example, 150–200 m for the Ai River, and the location of its terraces at a height of 40 m above the water's edge demonstrate the elevation of the territory. A comparison of the topographic maps of different publication years shows a tendency of riverbed straightening for 109 years. Zones of tectonic faults are expressed in the modern relief by lowered areas, steep ledges and zones of karst development. For the area of the mine field, a subhorizontal location of the axis of maximum compression in the NW–SE direction has been defined (Figure 2a).

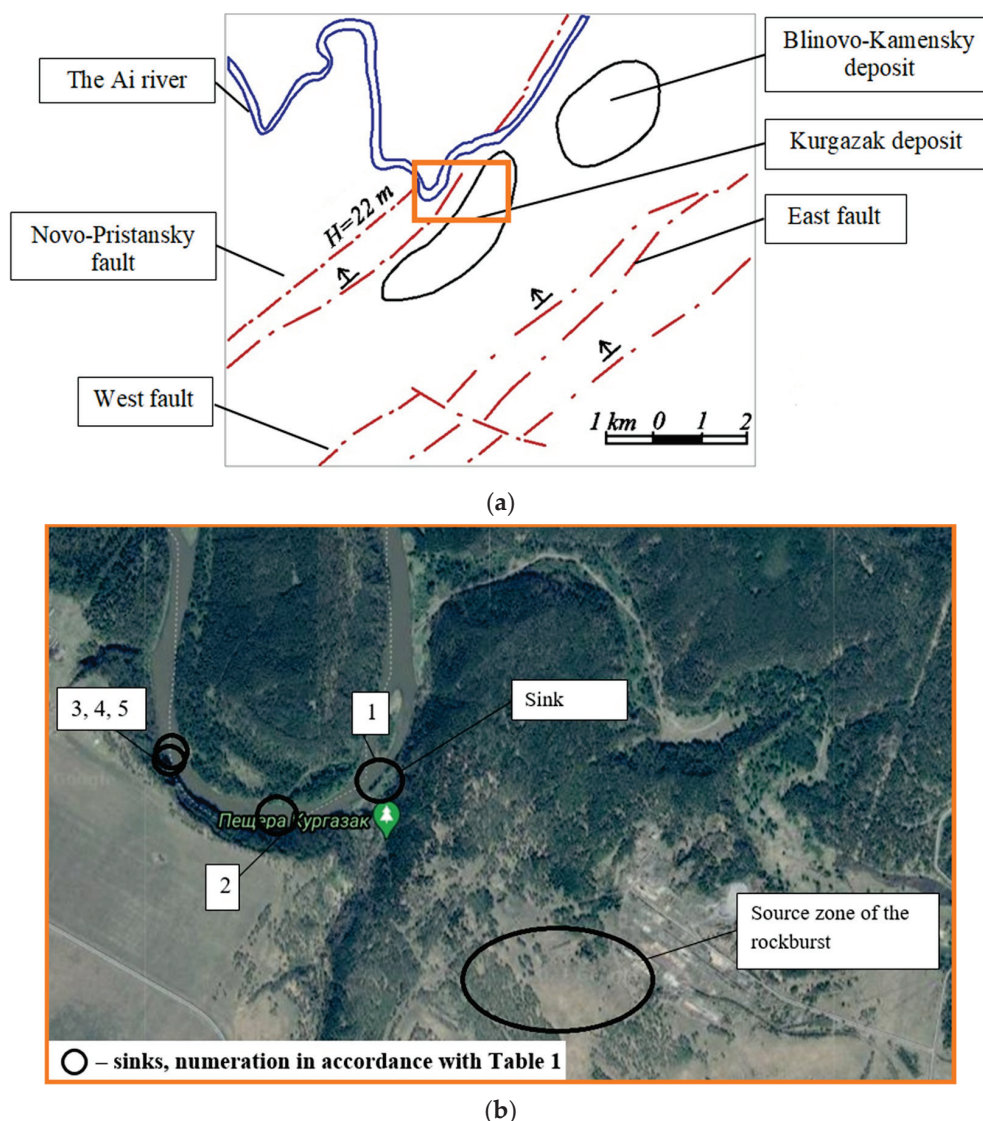


Figure 2. Study site of the Kurgazak mine: (a) Tectonic map; (b) space image of the region studied; 1–5: karst sinkholes according to the table in Section 4.1.

The mine field is bounded to the northwest by a large branch of the Novo-Pristansky fault of northeastern strike. To the southeast of the mine field occurs a zone of tectonic faults up to 1.5 km wide, also of northeastern strike, represented by the west and east thrust faults and their apophyses. Both the Novo-Pristansky fault and the west and east thrust faults have a southeastern dip, forming a tectonic plate including the deposit, with its dip to the southeast. The mentioned branch of the Novo-Pristansky fault is exposed by mine workings in different parts of the deposit. The amplitude of this fault ranges from 12 m in the northeast of the mine field to 22 m in the southwest. The smaller faults are also produced by mine workings: northeast strike faults with northwest and southeast dipping of fault planes and steeply dipping northwest shear faults (Figure 2a).

Mining operations at the mine were carried out in rather difficult hydrogeological conditions, which were associated with the presence of karst and the hydraulic connection of the mining area with the Ai River. Karst sinkholes appeared from time to time in the Ai riverbed, and an increase in water inflow in the mine Kurgazak has been registered (Figure 2b).

3. Theory and Methods

3.1. Tectonophysics Conditions of Faults' Reactivation

Rockbursts of “fault slip”, “slip burst” and “tectonic rockburst” types, according to the Russian classification, are accompanied by movements of the walls of tectonic faults, i.e., their reactivation [39–41]. The effect of fault reactivation under the influence of mining operations is explained by the favorable location of the fault in the stress field. Usually, the direction of movement is determined by a local mining-induced stress field formed as a result of mining activities and connected with the forming stress concentration zones. However, for strong rockbursts, as well as for induced earthquakes in other engineering activities, the correspondence is noted of the direction of movement along the reactivated faults to the regional stress field, i.e., the stress field of the undisturbed rock massif [42].

Strong rockbursts at the Karnasurt mine of the Lovozersky GOK and the Kirovsky mine of JSC Apatit were accompanied by displacement of the walls of large faults by up to 16 cm in the directions corresponding to the regional stress field. Focal zones of strong rockbursts had dimensions of up to 1 km and a hypocenter depth of 300–400 m [34,43].

It is obvious that the danger of dynamic fault reactivation depends on the values of tangential stresses in its fault plane, which depends on the orientation of the fault plane relative to the stress field. In this connection, the authors propose the use of the index $\hat{\tau} = \tau_n / \tau_{\max}$.

This index reflects the relative magnitude of tangential stresses acting at an arbitrarily oriented site (fault plane) and thus characterizes the possibility of shear displacement along the fault. The highest tangential stresses are achieved on the fault planes located close to the areas of maximum tangential stresses $\tau_{\max}$. The sites (fault planes) where tangential stresses are equal to zero are located perpendicular to any of the main stress axes. A relative value of the tangential stresses on the sites, given by the normal to the fault plane $\bar{n}$, can be estimated by the following formula [44]:

$$\hat{\tau} = \tau_n / \tau_{\max} = 2n_1n_3/x_2, x \quad (1)$$

where τ_n is the value of tangential stresses on the plane of the fault plane with the normal $\bar{n}$, n_1 and n_3 are the cosines of the angles between the normal $\bar{n}$ and axes of principal stresses σ_1 and σ_3 , and x_2 is the cosine of the angle between the axis σ_2 and a line in the plane of the fault plane perpendicular to the vector of displacement $\hat{\tau}_n$. Thus, the position of the fault plane relative to the sites $\tau_{\max}$ can be evaluated in the range from 0 to 1 in terms of index $\hat{\tau}$.

3.2. Reactivation of Faults during the Pore Pressure Measurement

In this paper, the authors rely on the theoretical statements presented in Gupta and Rastogi's work on the effect of water reservoirs on the seismic regime [27]. An increase in the pore water pressure P in the rock mass (due to a rise in the water level in the reservoir) decreases the normal compression σ_n of the walls of tectonic faults by the value P , while the tangential stress τ_n in their fault planes remains at the same level. A decrease in normal compression of fault planes with remaining values of tangential stresses leads to a decrease in shear strength by fault plane from τ to τ^* (Formulas (2) and (3) [27]) that promotes fault wing movement.

$$\tau = \sigma_n \times \operatorname{tg} \varphi + C \quad (2)$$

$$\tau^* = (\sigma_n - P) \times \operatorname{tg} \varphi + C \quad (3)$$

For open pits and mines, reactivation of faults is additionally promoted by the presence of significant-sized mined-out spaces, where the walls of faults can move during tectonic rockbursts since the presence of decompacted areas in the rock mass reduces its adhesion C . The authors assume that the seepage of precipitation in the mine fields increases the hydrostatic pressure in the zones of fault planes and that reactivation is experienced primarily by those of them that are favorably located in the field of active stresses. Satisfaction

of equality 2 or 3 for a particular fault plane represents the achievement of the critically stressed state in this plane, after which the dynamic displacement by the fault is possible.

“A rockburst is a brittle fracture of a critically stressed pillar or an edge part of the formation”—from this definition by [40] follows the idea of the source of a rockburst as a zone in which the critically stressed state of rock mass has been reached at the moment of fracture. In practice, the sizes of the rockburst sources are estimated by the results of the examination of faults in mine workings. Strong tectonic rockbursts with reactivation of faults at the Khibiny apatite deposits and in the Urals had focal areas of several hundred thousand square meters and linear dimensions of more than 1 km [7,38]. For example, during a rockburst on 17 August 1999, at the Umbozero mine (Khibiny) with magnitude $ML = 5.1$, the destruction area was more than $650,000 \text{ m}^2$, and the linear size of the focal zone reached 1–1.5 km. Cracks appeared on the ground surface. The rockburst at Kurgazak mine on 28 May 1990, with magnitude $M = 4$ resulted in a destruction area of about $450,000 \text{ m}^2$, but the linear size of focal zone reached 1–1.2 km. The depth of the hypocenter was estimated to be 300–350 m. Cracks appeared on the ground surface. If it is assumed that the focus of the rockburst was a zone of critically stressed rock mass state, then with its linear dimension $L = 1\text{--}1.5 \text{ km}$ and hypocenter depth $H = 0.3 \text{ km}$, one can obtain $L/2 > H$; i.e., the zone of critically stressed state occupied some space from the Earth surface to depth. The outcrop of cracks to the surface demonstrates that the fracture zone reached the Earth’s surface. In the case of the rockburst at the Umbozero mine, the Alluaiv Creek drained into the mine workings through the formed cracks.

According to Professor I.M. Petukhov, the zones of the critically stressed state of the upper part of the Earth’s crust are formed under the influence of horizontal compression forces, and their thickness depends on the interaction of the Earth’s crust blocks [40]. The facts given above about the ratio of the size of tectonic rockbursts’ centers L and the depth of their hypocenters H indirectly prove this effect in the deposit under study. Of course, the authors consider such a critically stressed state of the rock mass, taking into account the scale factor. The zone of the critically stressed state of the rock mass is characterized by structural hierarchy and discreteness of stress distribution. Local areas with the critical state are interspersed with relatively unloaded ones, but they collectively form a zone of critical state of a larger rank. Dynamic events at water infiltration are the result of successive disturbances of equilibrium in the rock mass sections, which are in the critically stressed state due to the above-described event of the hydraulic thrust of tectonic disturbances.

4. In Situ Studies

4.1. Monitoring of Water Inflows

The Rasvumchorr mine’s workings and boreholes are watered due to seepage of precipitation coming in through the caving funnel and, partially, due to groundwater levels rising during intensive snowmelt and prolonged rains. Starting from the +470 m level, ore mining is carried out in difficult hydrogeological conditions, i.e., in the zone of constant water saturation. At the junction zone of the underground Rasvumchorr mine and the Tsentralny open pit, watering of the rocks occurs due to surface water; in this connection, it is customary to use daily data on measurements of the mine water inflow in the ore passes of the Tsentralny pit located directly in the junction area.

Figure 3 shows the distribution of water inflow by seasons (spring–summer–autumn) for the monitoring period 2004–2018. The division by seasonal periods is carried out as follows: the spring period is established from the beginning of the decrease in the snow cover height and increase in water inflow until the end of snowmelt (zero snow level); the summer period runs from the end of snowmelt until September 1; the autumn period lasts from September 1 until the beginning of frost (average daily temperature is negative) and reduction in water inflow (to constant low values).

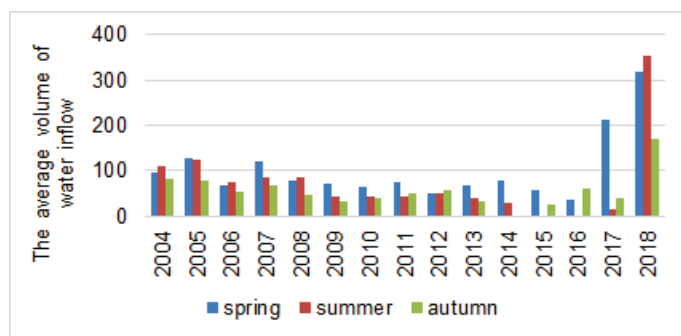


Figure 3. Average daily water inflow (m^3/day). The measurements were performed in the junction area of the Rasvumchorr underground mine and the Tsentralny open pit. There are no data on water inflow measurements during the summer period of 2015–2016.

It is noticed that the values of water inflows are higher in the spring period than in the autumn period, although it is more short-term. The pie chart shows the ratio of total water inflow in the spring and autumn periods according to the data for 2004–2018 (Figure 4).

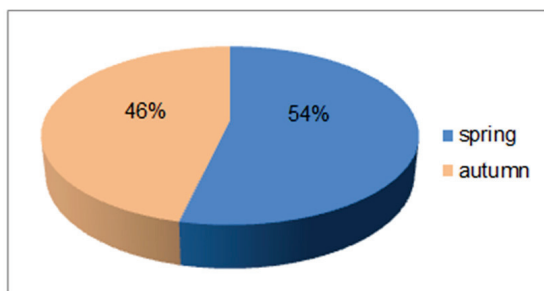


Figure 4. Ratio of the level of mine water inflow in spring–autumn period.

Annual water inflow indices are not uniform and depend on the amount of snowfall, liquid precipitation and positive air temperature (abundant snowmelt). The change in the maximum height of snow cover in May according to the Avalanche Safety Department of Apatit JSC is shown in Figure 5.

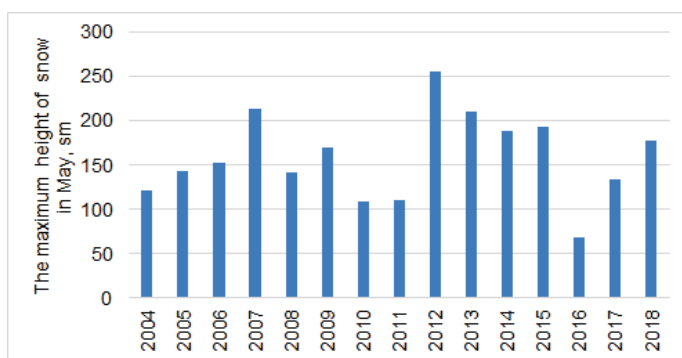


Figure 5. Maximum height of the snow cover per day in May.

In the Khibiny deposits, where there is a large snow accumulation in the caving zones and open pits, artificial fracturing is of considerable importance when the underground workings are watered by atmospheric waters. Shifting of large rock masses is accompanied by the formation of large cracks of the console part of the rock massif at a distance up to 100–150 m from the caving edge. Meltwater easily penetrates into the cracks.

At the Kurgazak mine, a seasonal increase in groundwater inflow was also noted. The correlation of inflows with infiltration losses of rivers was noted earlier during the development of deposits in karst rock massifs, and this was considered as direct evidence of the hydraulic connection of the river with underground mine workings [45]. The most expressive occurrences of such a connection were the formation of deep karst sinkholes directly in the Ai riverbed during floods and a sharp increase in mine water inflow. Some examples are given in Table 1.

Table 1. Characteristics of karst sinkholes in the Ai riverbed.

Discovery Date	Type of a Sinkhole	Note	Schematic Symbol, Figure 2
Spring of 1980	6 × 7 m in plan, depth 2 m	Backfilled in June of 1980	1
15 July 1984	By river width—10 m; by river length—14 m; depth—21 m.	Backfilled by 1 September 1984	2
21 June 1985	By river width—10 m; by river length—8 m; depth—6 m.	Backfilled on 2 August 1985	3
29 August 1985	By river width—5 m; by river length—4 m; depth—20 m.	Backfilled in autumn of 1985	4
Spring of 1990	Was formed on the site of sink 2	Existed to the moment of a rockburst on 28 May 1990	5

The mechanism of increase in river water inflows into bauxite mines along tectonic faults is considered in [45]. Karst cavities along the fault planes of tectonic faults are usually filled with loose clastic, sandy–clayey material. When the deposit is drained and the groundwater level is lowered, the filtration flow gradient increases, which triggers suffosion processes in karst cavities. The loose fine-grained material is directly washed out of karst cavities when tectonic faults are undercut by mine workings. This process develops from the bottom upwards and gradually reaches the Earth’s surface. As a result, karst sinkholes are formed, including those in the riverbed. In addition, zones of tectonic faults, as zones of high disturbance, are themselves groundwater reservoirs. Thus, a mine working of block 18, level—10 m, in the near-contact zone of fault 2 (Section 5.2 stripped a source of water with a flow rate of 1500–2000 m³/hour. Mining operations were stopped and a barrier pillar of 4 m was left along the fault.

From these observation results, the authors conclude that tectonic faults, which are conductors of river water into the mine workings, have well-washed fault planes, which ensures the transfer of hydraulic pressure along them when they are filled with water.

4.2. Monitoring of Seismic Events

The seismic monitoring in the Khibiny deposits has found that in late May–early June and in September–early October, the seismic activity annually increases in the form of an increased number of events and released seismic energy (Figure 6). In recent years, the greatest number of geodynamic events, accompanied by shaking of the Earth’s surface, crumbling of wall benches and damage to ore passes of the Tsentralny open pit, and violations of the working contour integrity of the underground Rasvumchorr mine, were registered in the spring and autumn.

The Kurgazak mine did not have a seismic station, and dynamic rock pressure was recorded by external signs. Shooting and dynamic failures at the roof of the stopping chambers were noted mainly in the northeastern part of the mine. Shocks from the soil of the stopping chambers, including in the central part of the mine field, were also noted. Four to five days before the rockburst of 28 May 1990, there was intense spalling in the pillars and shooting.

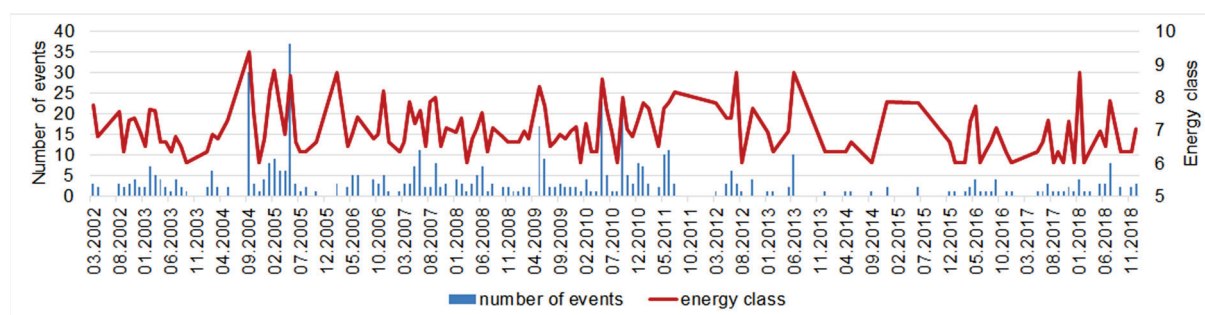


Figure 6. Monthly distribution of seismic events of energy class $K \geq 6$ in the control zone of the Rasvumchorr mine and the junction zone with the Tsentralny open pit.

5. Results and Discussion

5.1. Regularity of the Increase in Seismic Activity of the Rock Mass Whilst under the Impact of Surface Seepage during Snowmelt or Heavy Precipitation in the Khibiny Deposits

The tectonic stress field in the area of the Khibiny deposits is characterized by sublatitudinal subhorizontal orientation of the maximum compression axis and submeridional orientation of the minimum compression axis. The fracture faults are represented by smoothly curving, steeply dipping faults of northeastern and northwestern orientation. Their fault planes can be traced along the dip usually for tens and hundreds of meters and are often filled with secondary minerals. In the active stress field, such fault planes gravitate toward the τ_{max} sites and their index $\hat{\tau}$ is close to 1.

Modern seismological and geomechanical monitoring at mining enterprises allows revealing regularities of the influence of watering on the rock mass rockburst hazard. According to the data obtained, seismic activity increases annually in the spring season. At the Rasvumchorr Plateau deposit, seismicity activates earlier than that at the adjacent Apatitovy Tsirk deposit. This can be explained by the fact that the Rasvumchorr Plateau is a mined open pit, and therefore more cracks and tectonic faults, which are watered out during snowmelt, are exposed in this area. As a consequence, in the critically stressed state, faults are reactivated and seismic activity increases.

Detailed analysis of meteorological, hydrological and seismic data for the spring monitoring period from 2004 to 2018 has shown that the annual distribution of solid precipitation (snow) and water inflow is uneven and that the beginning of the snowmelt season varies from April to June. For example, in 2015 and 2018, it was early May; in 2016, it was late April, and in 2017, it was early June (Table 2). So on 7 May 2015, the height of snow cover was ≈ 193 cm; on 26 April 2016, it was ≈ 91 cm; the growth of water inflows was recorded in a few days, and in 2017, due to the cold spring, it shifted to June.

Table 2. Dynamics of annual snow melting.

Year	Day/Month	h (cm)	Δh (cm)	Start Date W (m^3 /Hour)
2015	7 May	193	−5	9 May
2016	26 April	91	−1	29 April
2017	7 June	120	−1	10 June
2018	6 May	177	−6	12 May

Since the water inflow growth is affected by the rate of snow melting and total watering of the rock massif, the authors have introduced the parameter of changes in the snow cover height per day, Δh (cm): $\Delta h = h_i - h_{i+1}$, where h_i is the snow height of the previous day. The Δh parameter shows to what extent the snow height has changed in one direction or another. Positive values indicate, for example, that there may have been blizzard transport of previously fallen snow in the measurement area. Figure 7 shows the results for 2015–2016.

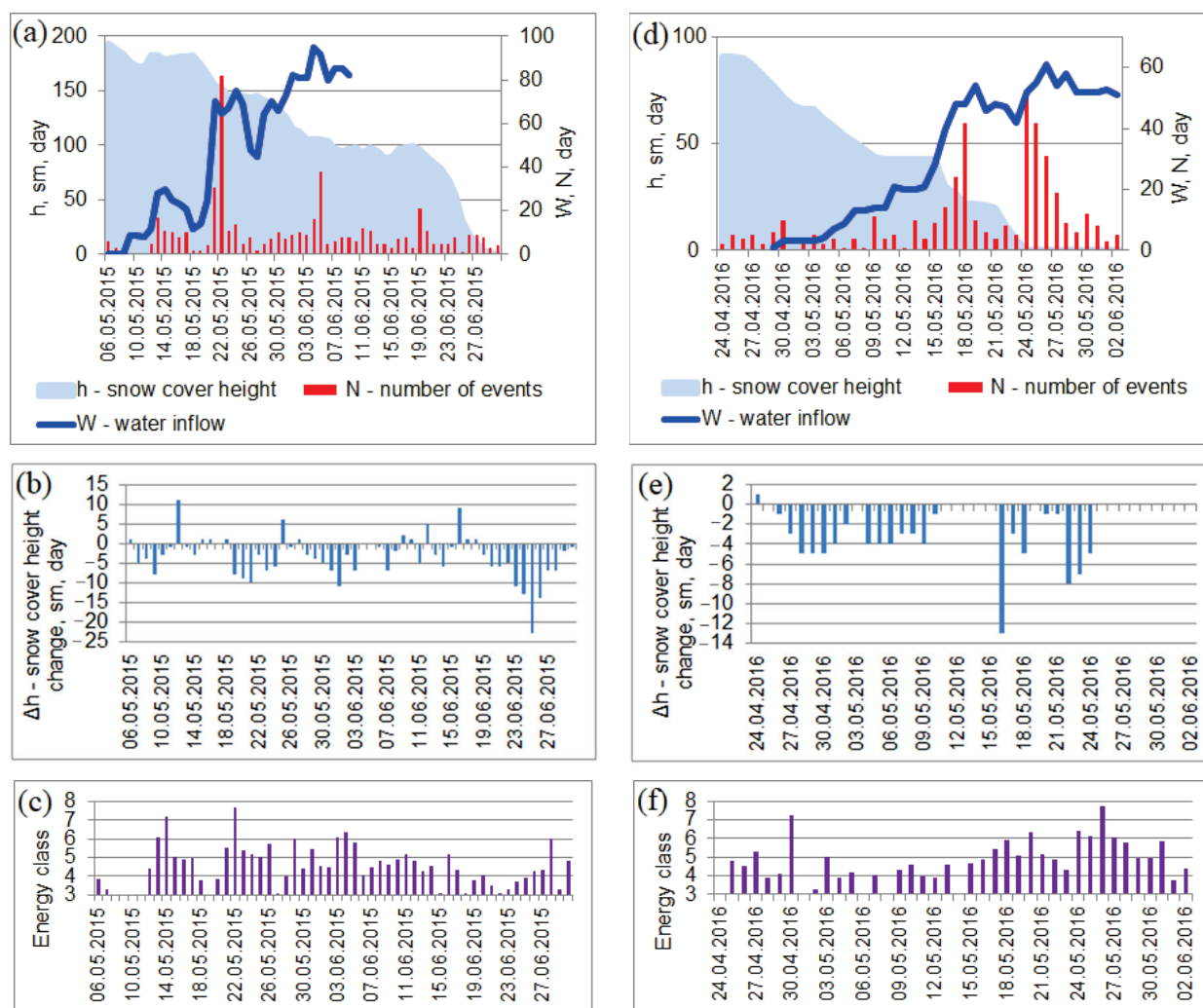


Figure 7. Influence of snow cover height and water inflow on seismicity changes. (a,d) the relationship of seismicity, water inflow, snow cover height; (b,e) snow cover height change; (c,f) energy class of seismic events change; (a–c) 2015 year; (d–f) 2016 year accordingly.

The above data show the increase in seismic activity (50% and more) both after a prolonged decrease in the snow cover height and after extreme changes in the snow cover height. Both coincide with a jump-like growth of water inflow. Positive air temperature leads to the formation of free water in the snow cover (when the snow reaches the melting temperature, i.e., 0° Celsius) and the beginning of its infiltration into the cracks on the surface of the rock mass. It is seen that the increase in the water ingress rates continued for over 5 days and the total water inflow volume exceeded the previous daily measurements by a factor of 2 (Figure 7a,d).

Figure 8 shows the results of a comparative analysis of seismic and meteorological monitoring data for the snowmelt period. Calculation of the water supply in the snow cover for the area of the Tsentralny open pit and the dumps was performed according to the formula $Z = 10hd$, where h is the average height of the snow cover (cm) and d is the snow density (g/cm^3). Taking into account the area of the open pit and dumps, the water reserve is equal to $Z \times S$ (m^3) (Figure 8b does not show the water reserve in the snow cover after 2015 due to the lack of data). The values of the water reserve in the snow cover show how much moisture is supplied for infiltration to the entire catchment area during the melting of the snowfall of the whole season, taking into account the liquid precipitation recorded in the spring period.

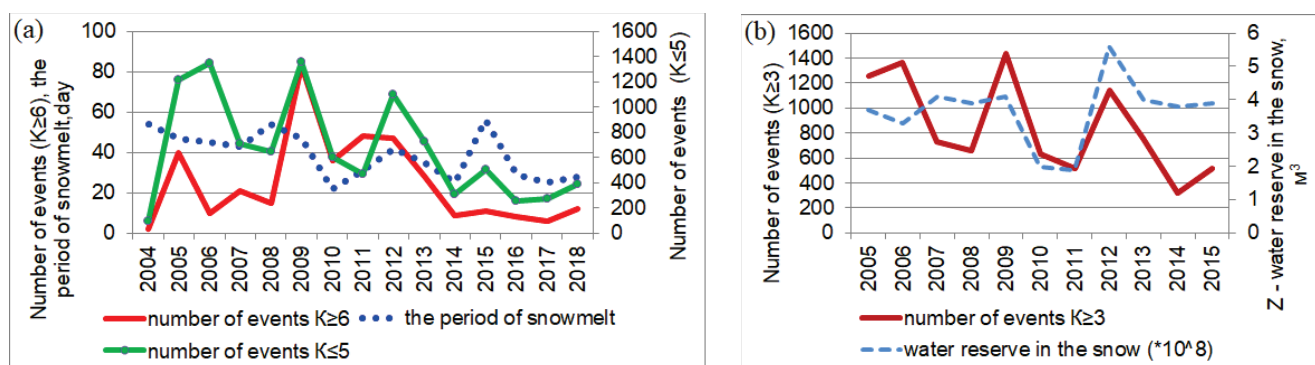


Figure 8. Comparison of seismic and meteorological monitoring data: changes in seismicity from snowmelt period (a), changes in seismicity from water reserve in snow cover (b).

Statistical analysis of the data shows that the cyclicity of geodynamic events in the studied area felt on the Earth's surface is 3–4 years. This may be contributed to by both prolonged snow melting (the period exceeds 40 days on average) and water reserve in the snow cover of more than $3 \times 10^8 \text{ m}^3$ (Table 3), which may be explained by peculiarities of the hydrogeological situation at the Apatitovy Tsirk and Rasvumchorr Plateau deposits. Unexpected effects of the major mining operation impact on the ground geodynamic regime are earth movements, rockbursts and induced earthquakes in surface mining operations—rare phenomena in mining practice. Examples are the events that occurred in 1995, 1998, 1999, 2003, 2004, 2005 and 2009 at the Rasvumchorr Plateau deposit, which were felt by residents of nearby cities.

Table 3. Seismic and meteorological monitoring data.

Year	The Period of Snowmelt (Day)	Initial Height of Snow Cover for the Snowmelt Period (cm)	Average Water Inflow during the Snowmelt Period (m ³ /hour)	Total Released Energy of Events (J)	Water Reserve in the Snow ($\times 10^8$) (m ³)	Number of Events $K \geq 6$	Number of Events $K \geq 3$
2004	54	121	1737	2.0×10^7	-	2	97
2005	47	143	1504	4.7×10^8	3.7	40	1258
2006	45	152	985	2.4×10^7	3.3	10	1361
2007	43	214	1889	7.4×10^7	4.1	21	729
2008	54	141	717	8.0×10^7	3.9	15	657
2009	47	170	722	6.8×10^8	4.1	83	1438
2010	22	109	315	4.2×10^7	2	36	635
2011	31	110	339	2.4×10^8	1.9	48	520
2012	42	254	701	5.7×10^8	5.6	47	1143
2013	35	210	864	6.8×10^8	4	28	755
2014	25	186	630	1.9×10^7	3.8	9	318
2015	56	193	551	1.5×10^8	3.9	11	517
2016	29	91	196	2.6×10^7	-	8	262
2017	25	121	2303	1.2×10^7	-	6	278
2018	28	177	3991	2.9×10^7	-	12	404

Note: average water inflow for the snowmelt period—from the beginning of water inflow growth to the end of snowmelt—period of increased watering (m³/hour).

In 2012 and 2013, tremors were registered in the open pit, felt in the underground workings of the Rasvumchorr mine. Most of the events of energy class $K \geq 6$ in 2012 were registered on May 18 and 19. The bulk blast produced in the Tsentralny open pit on May 18 contributed to the relaxation of stresses in the rock mass, resulting in a decrease in the overall energy level of the events.

Thus, a pattern can be traced, according to which during snowmelt or heavy precipitation the seismic activity in the rock mass increases. Some examples of seismic events and the nature of their occurrences during snowmelt are presented in detail in Table 4.

Table 4. Characteristics of seismic events, Rasvumchorr Plateau, 2005–2018.

Energy Class of the Main Event/Altitude Mark	Snow Melting Period	Energy Class of Foreshock/Aftershock Events	Height Range of Foreshocks/Aftershocks (m)	Description
Earthquake of 25 May 2005				
K = 8/altitude mark 445 m	1 May–17 June	K = 4 – 6/K = 4 – 8	200–700/150–850	A series of shocks was felt in the area of ore passes and in the administrative buildings. In the open pit, crumbling of rock was observed in the spreusteinization zone.
Earthquake of 25 May 2009				
K = 10/altitude mark—not determined (beyond the registration zone)	8 May–24 June	K = 4 – 7/K = 4 – 7	100–860/100–900	The main event as well as a series of foreshocks and aftershocks were felt in underground workings; in buildings of the Rasvumchorr, Tsentralny and Kirovsky mines; and in Kirovsk town. Rock falling off the walls and dust rising near the epicenter of the pit were noted at the mine. The events were triggered by a fault movement.
Shock of 18 May 2012				
K = 8/altitude mark—not determined (beyond the in the control zone)	16 May–27 June	K = 4 – 5/K = 4 – 7	340–600/100–700	Shocks were felt in the junction zone of the underground Rasvumchorr mine and the Tsentralny open pit, as well as shocks to the Earth's surface in the open pit.
Shock of 2 June 2013				
K = 8/altitude mark 436 m	16 May–20 June	K = 4 – 6/K = 4 – 7	100–600/100–700	Activation of seismic activity was registered in the junction area and in the ore pass area. Workers at the Rasvumchorr mine felt tremors on June 2. No damage was found.

Gupta and Rastogi explained the development of slips along the faults in the vicinity of dams by relief of normal stress due to pore pressure acting as a wedge [27]. It is assumed that a similar mechanism is also realized when mines are flooded [23–25]. Recent studies reveal new details in the seismicity evolution process in the dam area, such as seismicity migration [46–48] and fault activation at a distance of over one kilometer from the hydraulic fracturing location [5]. Some features of the fault activation process in response to cyclic and single-stage water injection are not yet entirely clear [49]. Table 4 shows that seismic events began to occur from a depth of 100–150 m, which can prove the hypothesis of a critically stressed state of the upper part of the Earth's crust in the area of the deposit. The zone of the critically stressed state of the Earth's crust near the surface in the area of the deposit under development could have been formed under the influence of interacting blocks of the Earth's crust in the tectonic stress field [50].

Thus, the mining area of the Khibiny rock massif can be considered as a part of the Earth's crust, within which the critically stressed state has been reached from the surface to a certain depth. In this case, both the geomechanical impact from mining operations and the seepage of precipitation cause an immediate redistribution of stresses in the rock mass, which leads to seismicity and strong tectonic rockbursts.

An upswing of seismic activity during the snowmelt period within this rock massif occurred when a range of conditions was met:

- Water reserve in the snow cover was more than $3 \times 10^8 \text{ m}^3$;
- The snowmelt period exceeded 40 days;
- Increase in water ingress rates continued for over 5 days and the total water inflow volume exceeded the previous daily measurements by a factor of 2.
- Some additional data is placed in Supplementary Materials.

5.2. A Possible Origin of the Strong Rockburst, Southern Ural

The rockburst at Kurgazak mine, one of the strongest rockbursts in the Russian mines, proceeded in several stages. The first rockburst occurred at 6.30 a.m. and was felt on the surface as a 5-point earthquake on the MSK-64 scale. At 8.40 a.m., there was a second shock, also felt on the surface, which caused the brick wall to collapse and cracks to appear in the mine building. In addition, according to eyewitnesses, 4 h later there were vibrations in the mine building and its rocking was noted again [51]. There are estimates that the hypocenter of this event was several kilometers deep [38]. According to the survey of the rockburst focal zone, fresh traces of displacement were noted on the surfaces of the shear fault planes. The dimensions of the focal zone along the strike reached 1.2 km with fractures reaching the surface, which can be regarded as evidence of the critically stressed state of the upper part of the Earth's crust at the time of the rockburst. During the repeated analysis of the mine documentation, the authors have found data showing that in the Ai riverbed, on the eve of the rockburst in 1990, another karst failure was found in the place where such failures had been formed repeatedly before (Figure 9). This previously unaccounted-for fact allows for further understanding of the mechanism of this rockburst.

At the present time, there are several ideas about the tectonophysical conditions of occurrences of strong rockbursts which have reactivations in large faults of mine fields and have depths of hypocenters below an area of mining operations. Available factual data on the conditions of rockburst occurrence in the Kurgazak mine allow the proposal of the following tectonophysical model of this event (Figure 9).

The stress state of the mine field area is characterized by the following orientation of the principal stress axes: the maximum compression axis σ_{max} is oriented along the azimuth 115° in an almost horizontal plane; the minimum compression axis σ_{min} is oriented along the azimuth 205° also in an almost horizontal plane; axis σ_2 is almost vertical. The mine field crosses a major fault with an amplitude of 22 m, which has an azimuth and dip angle of 130° and 60° , respectively. For this fault, $\hat{\tau} = \tau_n / \tau_{max} = 0.8$. As can be seen, the fault is located favorably in the field of regional stresses for the occurrence of reverse fault movement. The hanging wing of this fault in the field of regional stresses tends to shift upward into the area of the mined-out space (Figure 9).

A characteristic feature of the deposit development was regular water breakthroughs from the Ai River into the mine workings. From the sinkholes, sometimes more than 20 m deep in the riverbed, water flowed through tectonic faults into the mine, located hundreds of meters away from the river (Table 1). On the eve of the rockburst, another sinkhole was formed in the Ai riverbed at the place where it was crossed by a large fault.

Figure 9 shows that karst sinkholes had repeatedly been formed earlier, and the water from these faults had entered into the mine for weeks. On this basis, it is possible to assume that the fault planes had already been well washed out beforehand and represented macrochannels. The water inflow into these macrochannels from the surface resulted in hydraulic pressure of up to several megapascals at the depth of mining operations and below, which provided a reduction in the normal pressure of the reverse fault and the subsequent series of dynamic shifts. After the first movement (a rockburst), conditions may have been created to establish a hydraulic connection with the deeper levels of the rock mass, which caused the second and subsequent sudden movements. The hypocenters of these seismic events were below the mining area. The main movement occurred along the reverse fault, but, apparently, the shears transverse to the deposit strike were also reactiv-

vated, as evidenced by the presence of fresh traces of displacements on their fault planes.

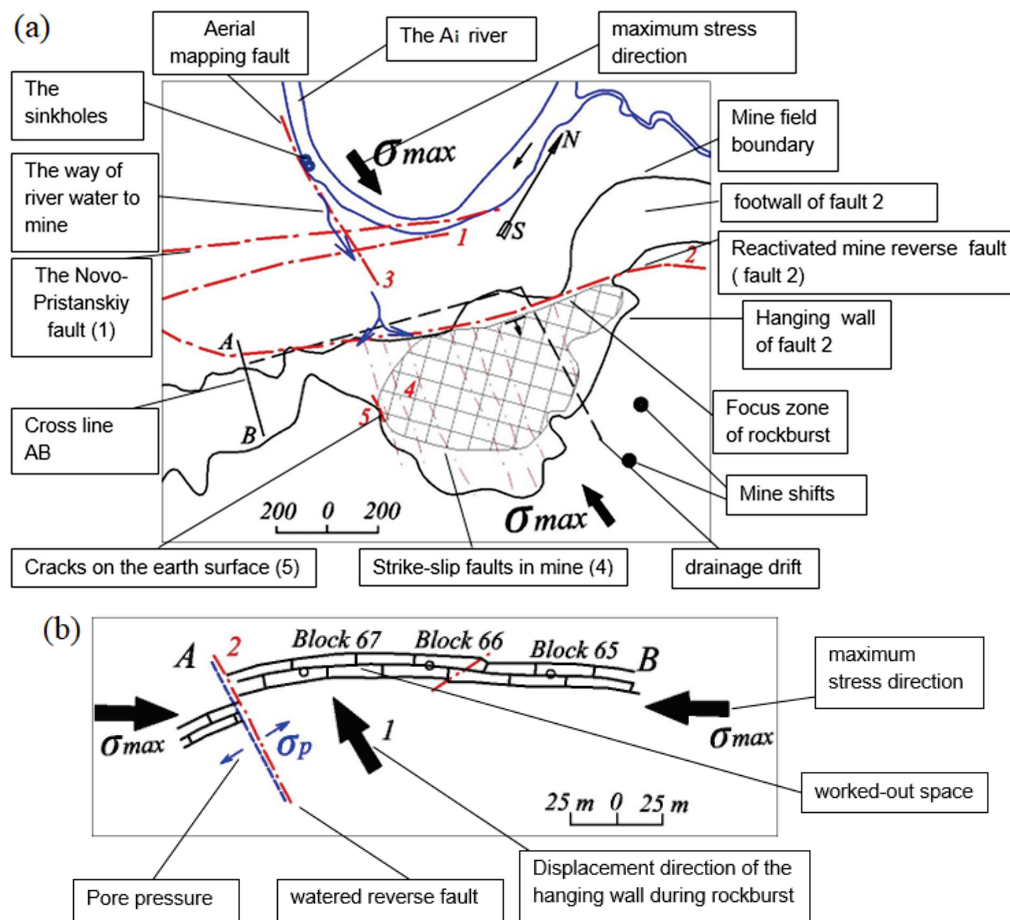


Figure 9. Tectonophysical diagram of the 28 May 1990, rockburst: Factual scheme of the bauxite deposit region (a); cross section along AB line (b) (according to (a)).

Thus, prior to this strong rockburst, there was an inrush of the Ai River waters into the mine workings along a large tectonic fault, which was not previously taken into account when analyzing the mechanism of this geodynamic event. The mechanism of this rockburst could be related to the reactivation of a large tectonic fault in the conditions of the critically stressed state of the rock mass with a decrease in the normal pressure of the fault plane during the water breakthrough from the surface reservoir.

6. Conclusions

Throughout the fifteen-year period of seismic monitoring at the Khibiny's Apatitovy Tsirk and Rasvumchorr Plateau fields, a seasonal trend of seismicity increase (by over 50%) during the spring snowmelt was recorded. An upswing of seismic activity during the snowmelt period within this rock massif occurred when a range of conditions was met:

- Water reserve in the snow cover was more than $3 \times 108 \text{ m}^3$;
- The snowmelt period exceeded 40 days;
- Increase in water ingress rates continued for over 5 days and the total water inflow volume exceeded the previous daily measurements by a factor of 2.

Despite the fact that mining is at depths of more than 500 m, seismic events during infiltration of water began to occur from a depth of 100–200 m and are recorded to depths of 900 m. The dimensions of the strong rockburst focal zones are several hundred meters vertically, which indicates that the rock massif in the area of these deposits is in a critically

stressed state practically from the Earth's surface to depths below the current mining levels. A possible controlling factor of the rock massif seismic activation during the period of intense snowmelt and precipitation is the reactivation of tectonic faults, which occurs under conditions of the critically stressed state of the massif, due to a decrease in their normal compression during infiltration.

Prior to a strong rockburst ($K = 10\text{--}11$) in the Southern Urals in May 1990, there was an inrush of the Ai River waters into the mine workings along a large tectonic fault, which was not previously taken into account when analyzing the mechanism of this geodynamic event. The intrusion of water into the fault located in the field of regional stresses and subsequent partial destressing of its fault plane from normal stresses could have triggered the rockburst with a fault-slip mechanism. With a depth of the rockburst hypocenter of about 300 m, the size of the source zone reached 1.2 km, which can be regarded as evidence of the critically stressed state of the massif at the time of the rockburst.

It is necessary to take into account the revealed mechanisms, i.e., the influence of natural factors on rockburst hazards, when preparing mining plans and conducting geomechanical monitoring.

Supplementary Materials: The following supporting information can be downloaded at: <https://www.mdpi.com/article/10.3390/su142215414/s1>.

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Article

Research on Mechanical Behavior and Energy Evolution of Coal and Rocks with Different Thin Spray-On Liners Thickness under Uniaxial Compressive Loading

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Abstract: Compared with the traditional supporting materials such as the bolts, mesh, and concrete, thin spray-on liners (TSLs) have the advantages of improving impact energy bearing and absorption capacity of rocks. In this study, the uniaxial compressive mechanical response feature of white sandstone, yellow sandstone, and coal with different coating thickness of TSLs were investigated through laboratory tests and theoretical analysis. The hard segments and soft segments of TSLs induced microphase separation, and formed the physical cross-linking. The hard segment evenly dispersed in the soft segment, which improved the mechanical strength of the TSLs. The confining pressure was affected by the TSL-coated coal and rocks, which improved the mechanical strength of the rocks and coal under UCS loading. It was discovered that the uniaxial compressive strength (UCS) increases with an increase in the TSL coating thickness, and UCS increments of the coated white sandstone, yellow sandstone and coal were 4.6%, 25.51%, and 43.75%, respectively. The energy absorption capacities of the coated coal, and yellow and white sandstones gradually decreased with an increase in their TSLs coating thickness. Meanwhile, the energy-absorbing capacity of a coated rock reduced gradually as rock coating decreases. Furthermore, the energy-absorbing capacity of coal is greater than that of yellow sandstone or white sandstone. By increasing the coating thickness, the total and dissipative energies in the yellow sandstone, white sandstone, and coal specimens were gradually increased, and energy increment was significantly more in coal (2–3 times), while the elastic energy remained nearly unchanged.

Keywords: coal and rock; uniaxial compression; TSLs; stress-strain curve; energy-absorbing

1. Introduction

With the development of deep excavation technologies for coal mine, problems such as high ground stress, high hydraulic pressure, high ground temperature and engineering excavation disturbance have emerged. These stresses destabilize coal strata, and cause frequent deep excavation disasters, especially rock burst [1] which refers to non-linear deformation of rocks caused by the elastic strain energy accumulated in them owing to severe excavation.

The rock burst mechanism has been researched by several universities and institutes [2]. It has caused geologic hazards and operation worker injuries and equipment unsafe. A rock burst accident scene accident diagram as shown in Figure 1. As an example of 12# mine of Pingdingshan Coal Group, rock burst prevention requires timely roadway

and ground support after excavation, and changes in rock stress condition must be considered. Some traditional techniques including using rock bolts, shotcrete, and wire mesh, or a combination of these.



Figure 1. Diagram of rock burst scene accident.

Rock bolt support techniques have been researched in China since 1956, and now they are a primary positive support technique used in mine excavation [3]. Rock bolts have a dominating effect, and initial stress and its dissipation play a decisive role in rocks support [4]; however, they have some limitations. Wire mesh can prevent small rocks from falling, and it diffuses the initial stress to the surrounding rocks by combining with plate and metal band, which is important to control loosening and deformation of surrounding rocks in the initial stage [5]. However, the yielding capacity support of wire mesh is effective when the surrounding rocks contain several deformations. Shotcrete seals the rock skin, thereby preventing rock weathering and slake reduction by bonding the joints and fractures. It is a brittle material that cannot withstand the rock tensile stress and can be destroyed easily; hence, it cannot stably support the rock and strata. Moreover, the thickness of shotcrete is more than 50 mm, which can be hazardous when it breaks and falls. Furthermore, its application has several problems such as low operation speed, increased dust, increased labor, high elasticity, and high logistic. Therefore, a new surface rock support technology with higher strength properties, rapid curing time and an improved yielding capacity is required to significantly improve rock stability.

A thin spray-on Liner (TSL) is a chemical membrane with a thickness of approximately 2–5 mm, which is sprayed onto the rock surfaces to support the rock mass [6]. TSLs comprise organic materials such as polyurethane, polyurea, acrylate, methacrylate and cementitious polymer. TSLs offer various advantages such as high adhesion properties, high tensile strength, prevention of air ingress in rocks and the ability to penetrate open joints and fractures of rock to be TSLs-rock composites [7–11]. They stick to the key loosen blocks of a rock mass and help interlock a fractured rock, and minimize the disintegration of loose rock. The support mechanism of TSLs presented in Figure 2.

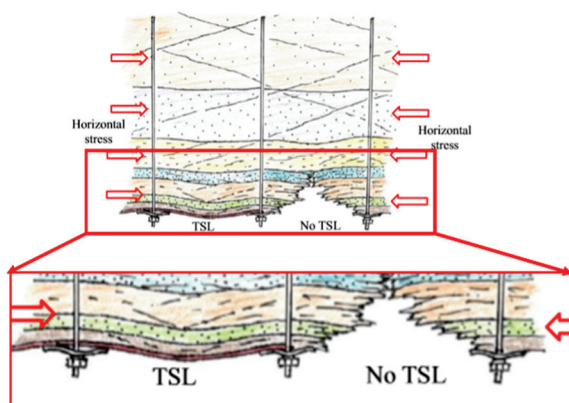


Figure 2. Support mechanism of TSL combined with rock matrix [12].

Stacey and Lau's study showed the mechanism of TSLs based on loading behavior and surface support behavior [13]. The support mechanism of a TSL are as follows [14–18]: (1) Basket mechanism [8,14,19]: The flexural and tensile strengths of a TSLs bearing the rock gravity load when it works as an independent support structure. It forms a bearing basket to support the loosened rocks or unstable key blocks. (2) Glue mechanism [15,17,20–22]: sprayed TSLs can penetrate the joints and fractures of a rock, adhere small rocks together as a whole like glue, and promote block interlock. (3) Wedge mechanism [14,18,23,24]: sprayed TSLs penetrate the joints and fractures and grout the dilatation space of crevices, such that friction force between rocks and strata restricts the rock from falling; hence, TSLs act as a wedge to improve the stability of rocks. (4) Membrane effect [15,16,25–29]: TSLs offer both the shotcrete and bolt mesh effects, and they can function as a sealing skin to prevent irregular gas emissions and water seeping from the ribs and can prevent further weathering. They rehabilitate highly stressed strata to stabilize the loosen rocks.

Although the uniaxial compression test conditions for the combination of TSLs and coal or rock have matured, there are few studies on the mechanical properties and energy evolution process of this combination. To address these issues, this study used professional equipment to control the slurry mixing ratio with high precision and an automatic spray gun to uniformly coat the specimen. The uniaxial compression experiment was performed on various combinations of TSLs and coal or rock, and the evolution law of elastic and dissipation energy during the failure of the coated rock specimen was analyzed. This study demonstrates the improvement of compressive properties of coal and rocks through TSLs coating, and to provides an experimental basis for the application of TSLs in the relevant support fields.

2. Mechanical Analysis of TSLs

2.1. Preparation and Tensile Testing of the TSLs

Generally, TSLs preparation involves mixing two materials. In this study, one was a pale-yellow sticky liquid, which was marked as A, and the other was a pale gray viscous liquid, marked as B. If A reacts smoothly with B, their viscosity is similar. The viscosities of A and B were 1479 mPa·s and 1493 mPa·s, respectively, which were measured using the NDJ-5S rotary viscometer (Shenzhen Dingxinyi Experimental Equipment Co., Ltd., Shenzhen, China).

A and B were blended and impinged in the reaction chamber at high temperature between 60–80 °C and pressure of approximately 10 MPa, and then sprayed into an elastic membrane. At this time, the TSL was bubbling and had a weak strength owing to a significant pressure difference.

To obtain the mechanical properties of the TSLs, it is necessary to perform tensile mechanical tests on them. A TSL sample was prepared by spraying materials A and B with a sprayer, as shown in Figure 3. The sample was stored in a container that could maintain a constant temperature and humidity, and then cut into a dumbbell shape with an electrohydraulic punching machine. Thereafter, its tensile strength was tested.

To determine the tensile properties of TSLs, ASTM D638 Type-IV dumbbell test specimens were molded and tested [30]. The dumbbell shape is shown in Figure 3; the thickness of the TSLs sprayed on the acrylate plate was approximately 3 mm, and the sample was leveled with a spatula to maintain a uniform thickness. To cure dry, and solidify the sample, it was stored in a container that maintain a constant temperature (25 °C and humidity (50%) for seven days.

After removing the sample from the container, it was cut with a CP-1 electrohydraulic punching machine in a dumbbell shaped mold. The scraper modified the sample surface to maintain a smooth outline. The constant load rate was 10 mm/min until the failure occurred.

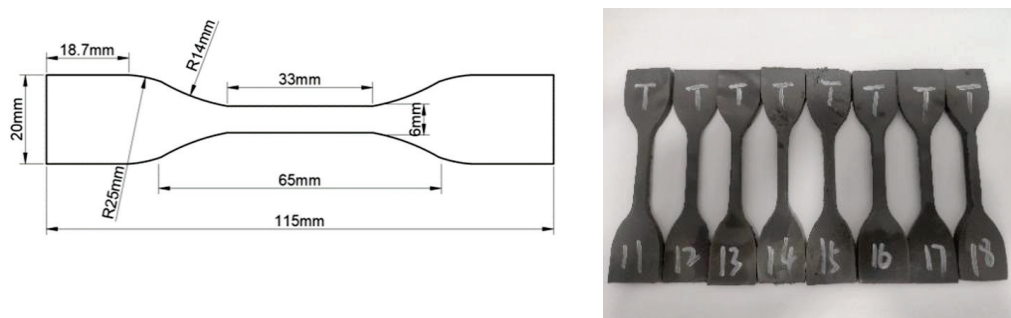


Figure 3. Type IV Dumbbell specimen [30].

The tensile loading of the sample induced the highest stresses in the narrow sections of the center parts. The specimen must self-align with the direction of the pull to prevent inaccurate results owing to eccentric loading. The tensile strength was tested under the displacement control mode, and the loading and displacement were recorded through the testing process. The result was only considered valid when the dumbbell specimen broke at the narrow section, Figure 4 shows the results of this experiment, wherein five specimens are valid damaged, and the breaking site of sample number 18 is not in the middle; hence, the breakage is invalid.



Figure 4. Damage diagram of type IV dumbbell specimen.

The tensile strength of TSLs, averaged tensile strength, elongation, and shore hardness of the TSLs are listed in Table 1.

Table 1. Test results of type IV dumbbell sample.

Specimen Number	Curing Time/d	Tensile Strength/MPa	Average Tensile Strength/MPa	Elongation/%	Shore A	Shore D
IV-TSL12	7	23.79	24.44	526.49	91.5	45.5
IV-TSL13	7	25.53		434.71	91	46
IV-TSL14	7	23.63		313.89	92	45
IV-TSL15	7	21.84		228.19	91.5	45
IV-TSL16	7	27.41		401.09	92	45.5
IV-TSL18	7	NA		NA	NA	NA

2.2. SEM Analysis of the TSLs

According to the information presented in Table 1, the tensile strength of IV-TSL 16 is maximal and that of IV-TSL 18 is zero. The scanning electron micrograph of the fracture surface after curing both IV-TSL16 and IV-TSL 18 for seven days is shown in Figure 5. The hard segment and soft segments of TSLs cause microphase separation and form physical cross-links when material A reacts with B. Previously, the hard segment was dispersed

evenly in the soft segment to improve the mechanical strength of the TSLs [31,32]. IV-TSL 16 was sprayed such that material A and B reacted completely; hence, the TSLs were stable. IV-TSL 18 exhibits a bump in the SEM, which prohibited an effective microphase separation, and its tensile strength was weak; therefore, it broke easily when tensile strength was applied to it.

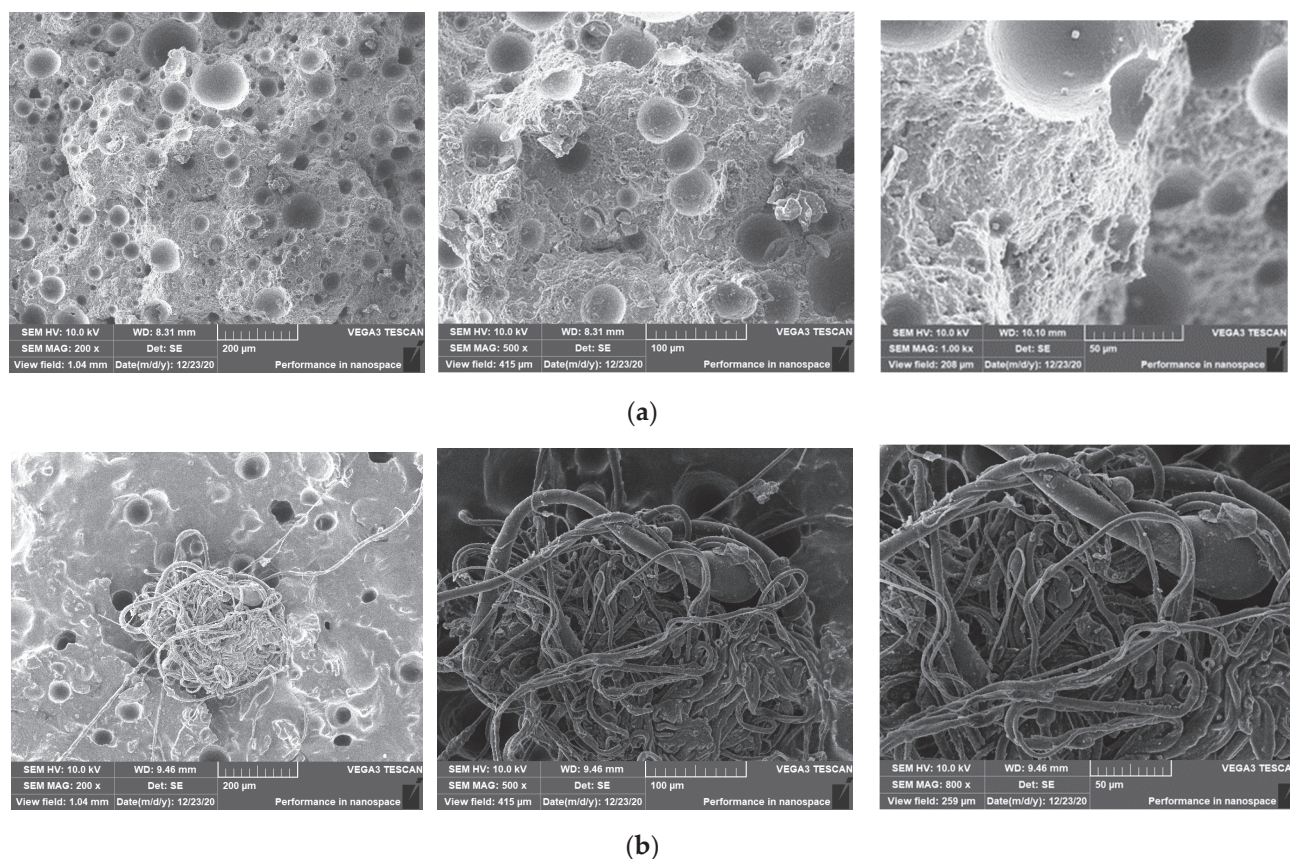


Figure 5. SEM image of the fracture of IV-TSL specimens: (a) IV-TSL16 specimen, (b) IV-TSL18 specimen.

3. Compression Test of the Combination of TSLs and Coal and Rock

3.1. Preparation of the Combination of TSLs and Coal-Rock

Preparation of the specimen: 18 samples of yellow sandstone and 15 samples of white sandstone were obtained from Sichuan Zigong, and 15 of coal were obtained from Zhaogu coal mine in Jiaozuo, Henan Province. These specimens were configured in cylindrical shape ($D = 50$ mm, $H = 100$ mm), and then used in UCS test.

The reaction of material A and B was performed in a Graco E-10 hp mini chamber. The specimens were then sprayed with different coating thicknesses of the obtained TSL, which were 2 mm, 3 mm, 4 mm, and 5 mm. Uncoated specimen and those coated with the TSL are shown in Figure 6. The specimen surface was leveled with a scraper to maintain a uniform thickness. The coated and uncoated specimens were solidified in a constant temperature (25 °C) and humidity (50%) curing box for seven days. The UCS experiments were conducted on MTS E45.305, and the loading velocity was 0.1 mm/min. The rock mass deformation destruction characteristics, acoustic emission spatial-temporal evolution pattern and micro destruction mechanism of coated and uncoated specimens were analyzed.

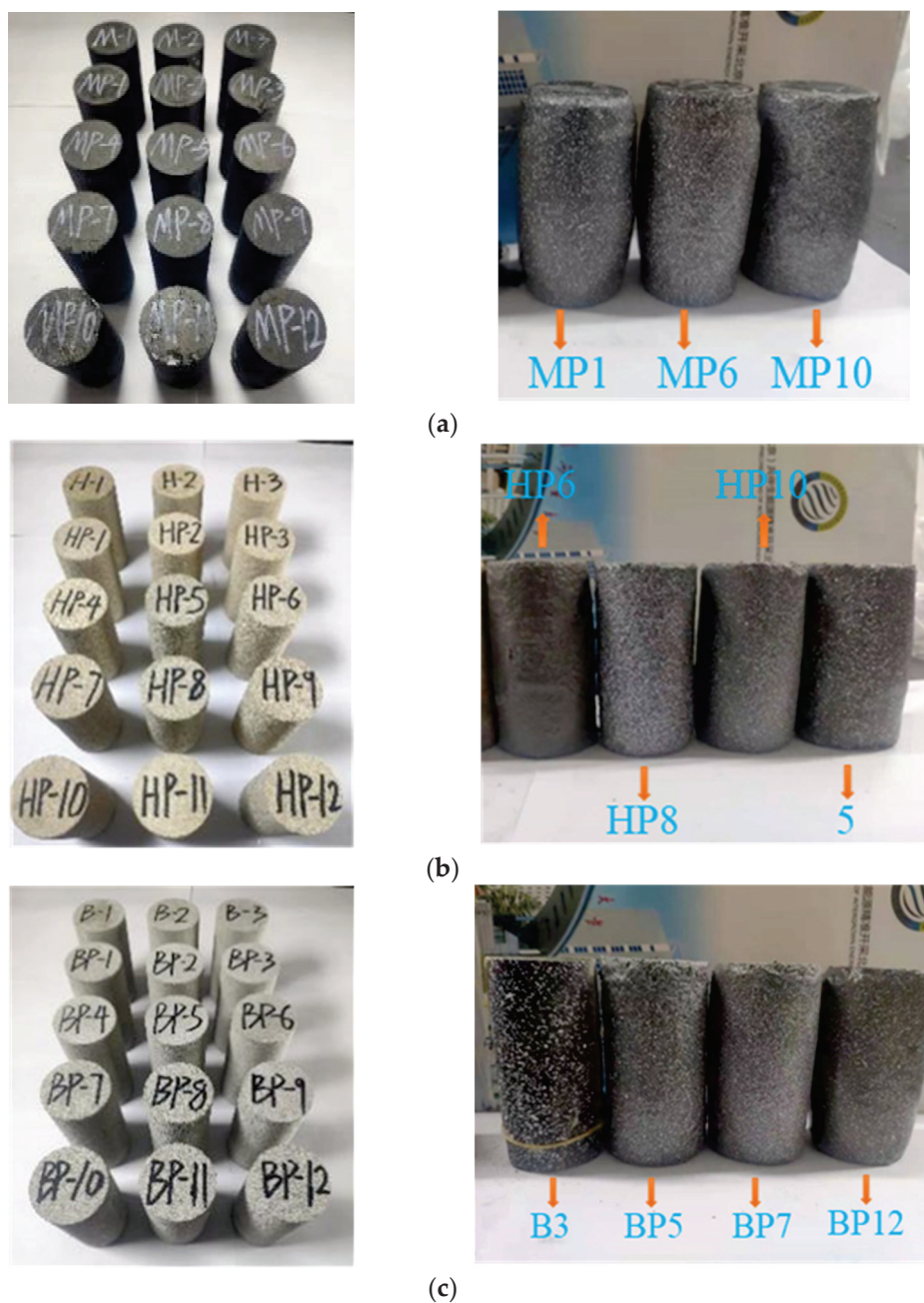


Figure 6. Different TSL uncoated and coated rock materials: (a) Coal specimens, (b) Yellow sandstone specimens, (c) White sandstone specimens.

3.2. Primary Parameter Analysis

The average density of Zhaogu coal, white and yellow sandstones, respectively. 1.467 g/cm^3 , 2.440 g/cm^3 , and 2.322 g/cm^3 , its volatility range of $1.446\text{--}1.490 \text{ g/cm}^3$, $2.418\text{--}2.455 \text{ g/cm}^3$, $2.276\text{--}2.362 \text{ g/cm}^3$, respectively. As shown in Table 2.

The primary wave (P-wave) indicated an astonishing property of the coal interior. Because the UCS has a positive correlation with the P-wave velocity under the uniaxial loading, P-wave was used to identify mechanical characteristics and estimate rock mechanism parameters [33–35].

Table 2. Physical parameters of coal and rock without spraying and with different spraying thicknesses.

	Thickness TSL Coated mm	No.	M/g	Velocity m/s	Elastic Modulus GPa	Uniaxial Compressive Strength MPa	Density g/cm ³
Coal	0	M2	275.5	1847.22	2.476	15.89	1.467
	0	M3	274.0	1344.86	2.647	17.62	1.446
	0	19	279.0	1837.61	2.356	13.80	1.467
	2	MP1	276.0	1892.34	2.411	19.28	1.467
	2	MP2	287.0	1742.40	NA	NA	1.470
	2	MP3	271.0	1929.65	1.743	23.15	1.456
	3	MP4	275.0	1821.82	2.715	22.03	1.462
	3	MP5	287.0	1728.45	NA	NA	1.468
	3	MP6	276.5	1869.53	2.696	23.31	1.484
	4	MP8	279.5	1668.48	NA	NA	1.490
	4	MP10	269.5	1920.42	2.003	22.34	1.456
	4	MP11	279.0	1804.20	2.521	20.16	1.466
White Sandstone	0	B1	469.3	2339.52	NA	NA	2.433
	0	B3	471.0	2267.32	4.546	41.51	2.434
	0	38	471.0	2180.94	4.388	42.30	2.440
	2	BP4	467.9	2187.54	NA	NA	2.436
	2	BP5	466.0	2184.88	3.811	38.42	2.430
	2	BP6	469.3	2253.59	3.763	37.92	2.418
	3	BP7	467.9	2200.44	4.252	42.99	2.426
	3	BP8	471.4	2275.45	3.879	40.98	2.452
	3	BP9	472.1	2227.99	3.570	45.18	2.453
	4	BP10	472.2	2185.14	4.198	42.36	2.446
	4	BP11	470.2	2258.71	NA	NA	2.455
	4	BP12	472.1	2241.89	4.294	45.30	2.438
Yellow Sandstone	0	H1	428.3	1793.17	3.448	38.28	2.289
	0	H2	428.2	1728.80	3.344	34.94	2.290
	0	H3	441.7	1702.78	NA	NA	2.278
	2	HP-4	427.5	1741.35	3.357	35.24	2.276
	2	HP-5	448.0	1639.02	NA	NA	2.362
	2	HP-6	429.5	1717.28	3.481	37.45	2.287
	3	HP7	439.3	1717.79	NA	NA	2.344
	3	HP8	449.4	1717.24	3.872	40.98	2.357
	3	HP-9	440.0	1868.12	4.016	39.61	2.350
	4	HP10	451.1	1718.00	4.197	42.36	2.361
	4	HP-11	429.5	1725.17	NA	NA	2.287
	4	HP12	430.5	1784.26	3.432	41.46	2.292
	5	5	440.5	1900.44	3.762	46.40	2.371
	5	7	442.5	2244.93	4.384	45.50	2.351
	5	20	444.0	2107.28	NA	NA	2.358

After being curing for seven days, the specimens were tested under ultrasonic velocity. Yellow sandstone (HP), white sandstone (BP), and coal (MP) were classified into groups based on their coating thickness: 0 mm, 2 mm, 3 mm, 4 mm, and 5 mm. Each group contained three samples. As listed in Table 2, the average velocities of white sandstone, yellow sandstone, and coal were 2238.92 m/s, 1790.80 m/s and 1783.91 m/s, respectively. The P-wave velocity gradually increased with the density improvement of the specimen. It was maximal for the white sandstone and minimum for coal because the bigger grain clearance and porosity, the smaller the density of the specimen; therefore the ultrasonic attenuation was significant during its propagation.

From Table 2, it is evident that the elastic modulus and uniaxial compressive strength of white and yellow sandstone are significantly higher than that of coal. This could be because the sandstones are more compact than coal.

3.3. Loading System

MTS E45.305 loading system, the axial loading about 0–300 kN, loading velocity is 0.005–500 mm/min, which can record data and plot stress-strain curve in Figure 7.

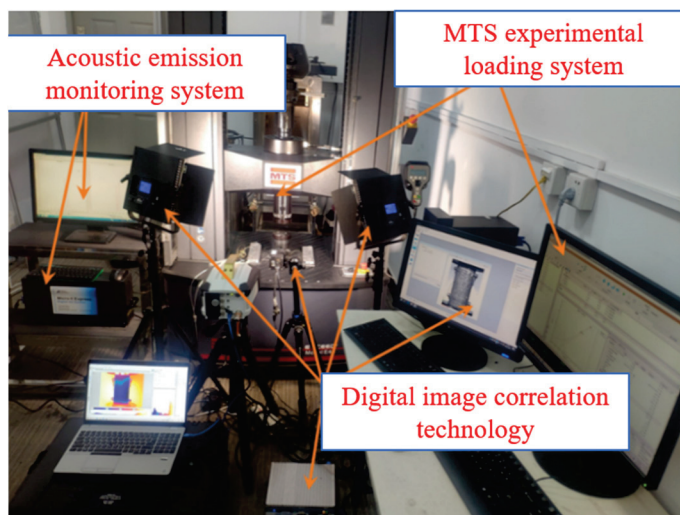


Figure 7. Uniaxial compression test detection system.

4. Compression Test Results and Analysis

4.1. Analysis of Compressive Mechanical Properties of Specimens

1. Mechanical behavior of coal specimen coated with TSL.

Figure 8 shows a comparison of the stress-strain curves of the uncoated coal and those coated with 2 mm, 3 mm and 4 mm of TSL. Each group of 2 mm, 3 mm and 4 mm TSL coated coal had one specimen that broke due to wrong operation; therefore, these specimens' curve were not included in the Figure 8. Compared with the uncoated coal, the UCS gradually increased with the coated thickness in the coated coal. It is evident that the stress-strain curve changes with the coating thickness and the shape of the uniaxial compressive stress-strain curve of the uncoated coal were sharper than those of coated specimen [36,37]. However, the curve shape of the coated coal was relatively round and gentle, the stress increased and decreased several times with the strain, and there was a step-like stress decreasing trend in the post-peak stage. The maximum peak stress (15.77 MPa) on the stress-strain curve of the uncoated coal was less than that on the coated coal (21.22 MPa (2 mm), 22.67 MPa (3 mm), 21.25 MPa (4 mm)). Thus, the stress gradually increased with the thickness of the TSL coating.

TSLs reinforce coal by imparting a confining pressure-like effect to limit its lateral deformation. Sprayed TSLs penetrate the cracks of the coal and provide considerable adhesive strength, which offers the potential support to its structure. Similar to the confining pressure load rendered by TSLs, the properties of the coal itself are also affected to a certain extent. The primary fractures inside the coal specimen are compacted by the confining pressure before the axial compression is applied. The higher the confining pressure, the better the fracture closure.

As evident from the curve of stress-strain shown in Figure 8, the TSL coating increased both the peak pressure and the residual strength of the coal specimens. Compared with uncoated coal, the peak pressure of coal coated with 2 mm, 3 mm and 4 mm of TSL improved by 34.56%, 43.75% and 34.75%, respectively. The post-peak pressure curve of TSL coated coals was more gradual than that of the uncoated coal, and a step-like pattern was observed on it, which indicates that the TSLs significantly improved the residual strength of the coal, and the confining pressure effect exerted by them reinforced the specimens.

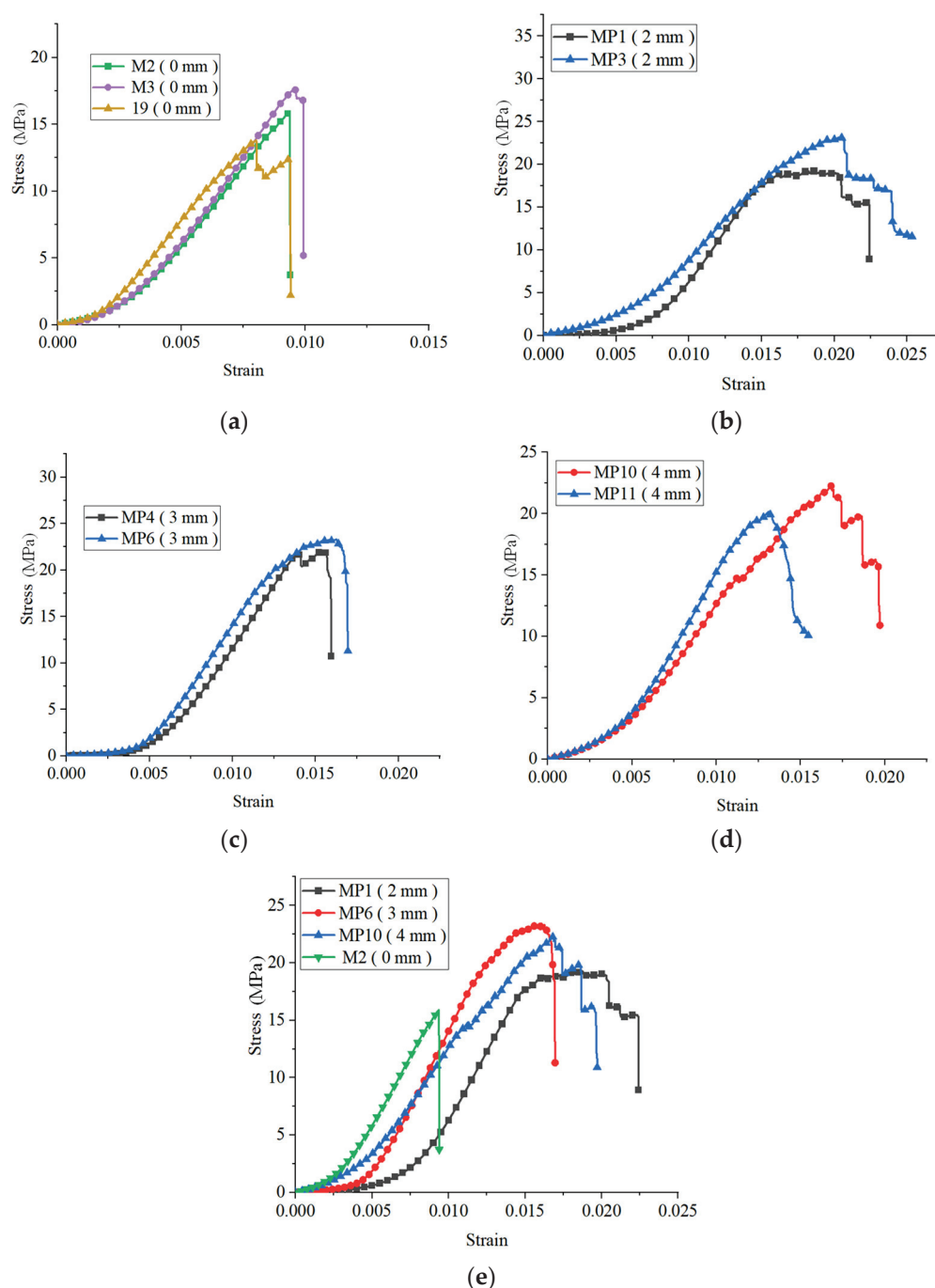


Figure 8. Comparison of stress and strain curves of coal with different spray thickness: (a) TSL uncoated coal, (b) 2 mm TSL coated coal, (c) 3 mm TSL coated coal, (d) 4 mm TSL coated coal, (e) TSL coated coal with different thickness.

2. Mechanical behavior of yellow sandstone specimen coated with TSL.

Figure 9 shows a comparison of the stress-strain curves of the uncoated yellow sandstone and those coated with 2 mm, 3 mm, and 4 mm of TSL. Similar to the results of the coal specimens, the UCS of the yellow sandstone specimen increased gradually with an increase in the coating thickness. In Figure 9, the uniaxial compressive stress-strain curve of the uncoated yellow sandstone was sharp, and the stress increased and decreased only once with the strain, whereas the curve of the coated ones was relatively round and gentle, and the stress increased and decreased several times with the strain. The peak stress (36.61 MPa) in the stress-strain curve of the uncoated yellow sandstone was less than that of the coated

yellow sandstone (36.65 MPa (2 mm), 40.30 MPa (3 mm), 41.91 MPa (4 mm)). Thus, the stress gradually increased with the thickness of the TSL coating.

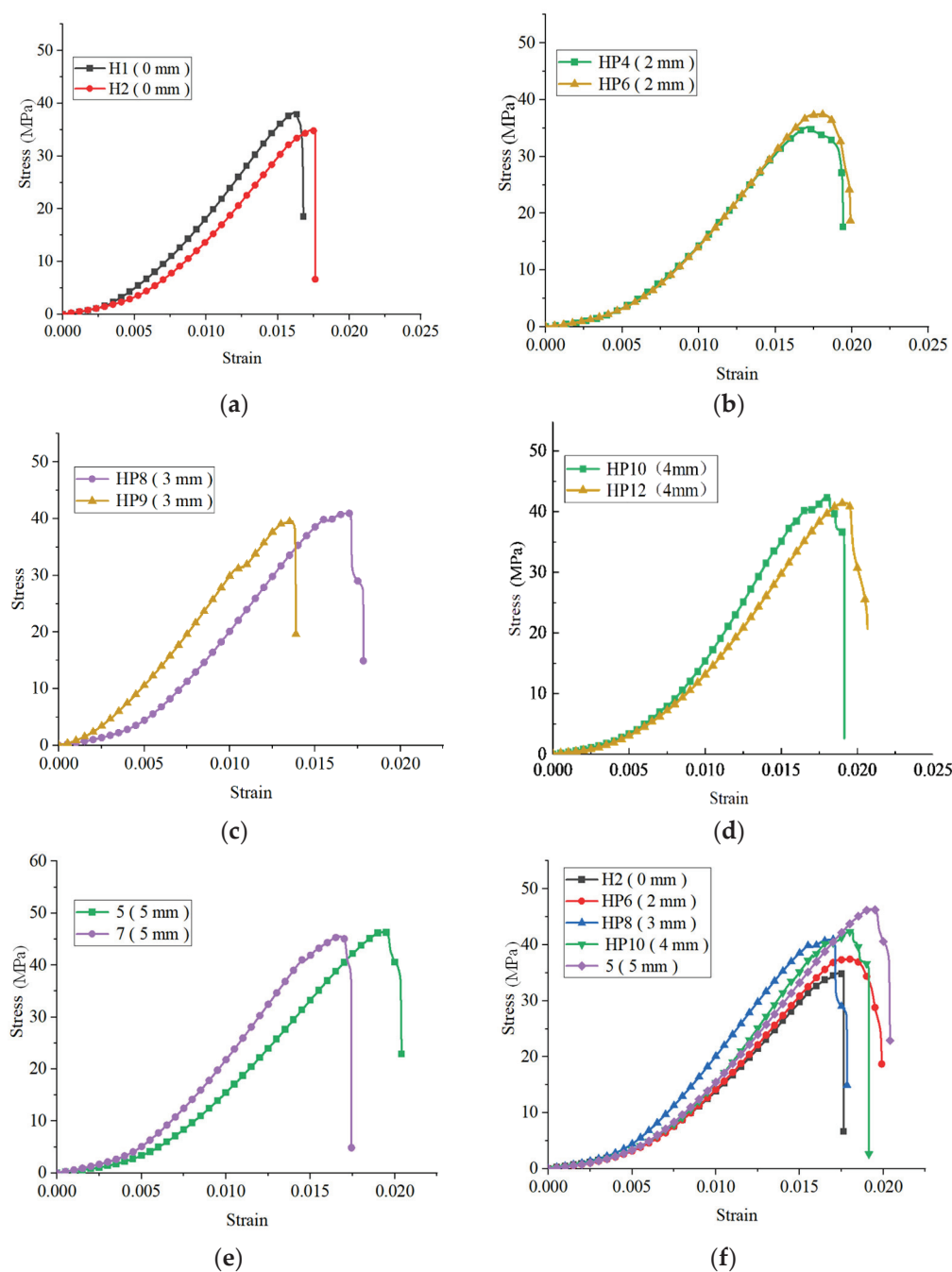


Figure 9. Stress-strain curves of yellow sandstone with different spray thickness: (a) TSL uncoated yellow sandstone, (b) 2 mm TSL coated yellow sandstone, (c) 3 mm TSL coated yellow sandstone, (d) 4 mm TSL coated yellow sandstone, (e) 5 mm TSL coated yellow sandstone, (f) TSL coated yellow sandstone with different thickness.

3. Mechanical behavior of white sandstone specimen coated with TSL.

Figure 10 shows a comparison of the stress-strain curves of the uncoated white sandstone and those with 2 mm, 3 mm, and 4 mm of TSL coating. Similar to the results of the coal specimens, the UCS of the white sandstone specimen increased gradually with the increasing in the coating thickness. The uniaxial compressive stress-strain curve of the uncoated white sandstone was sharp, whereas the curve of the coated white sandstone was

relatively round and gentle, the stress increased and decreased several times with the strain, and a step-like stress decreasing trend in the post-peak stage was observed. The peak stress (41.9 MPa) in the stress-strain curve of the uncoated white sandstone was smaller than that of the coated white sandstones (38.17 MPa (2 mm), 43.05 MPa (3 mm), 43.83 MPa (4 mm)). Hence, the stress gradually increased with the TSL coating thickness.

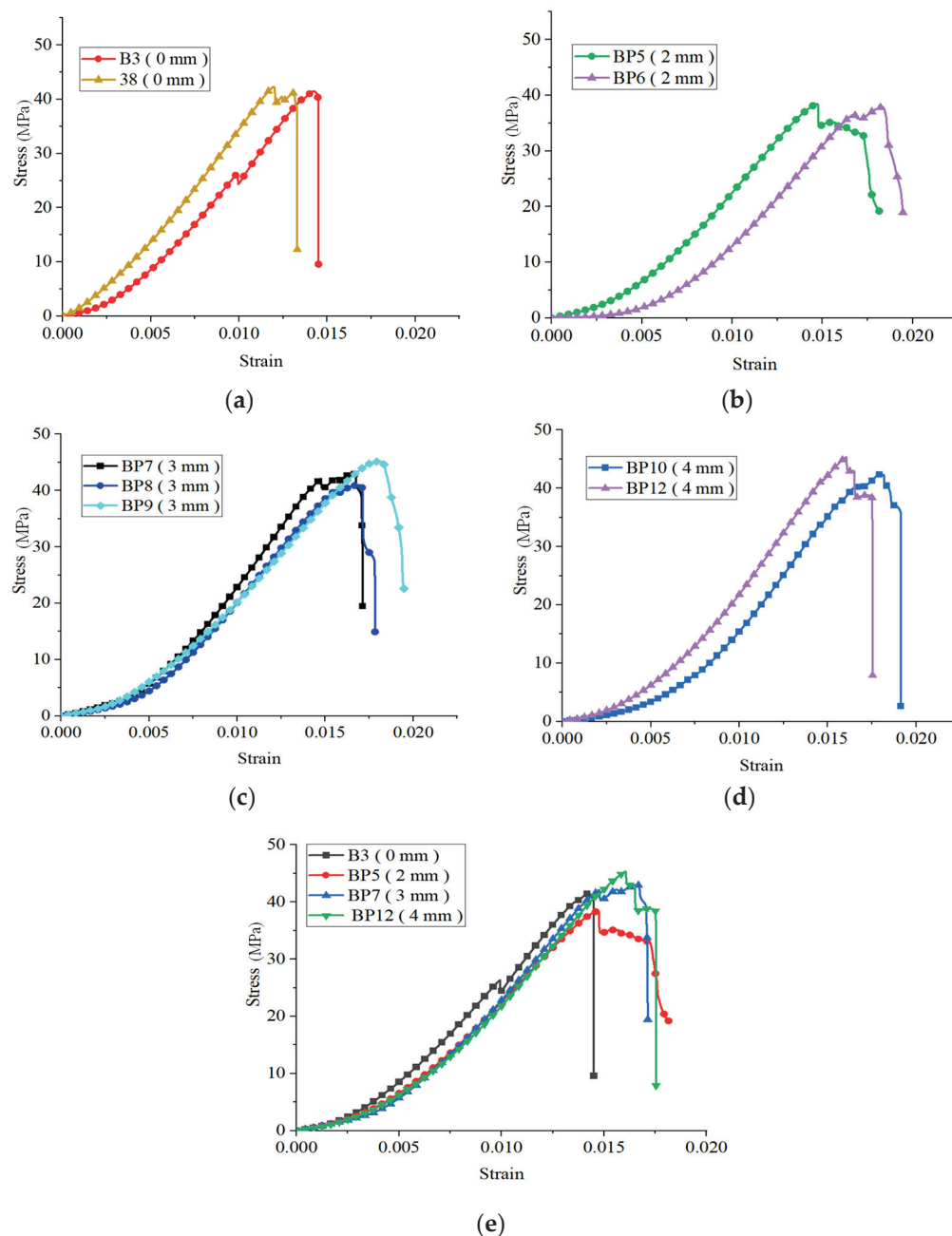


Figure 10. Comparison of stress and strain curves of white sandstone with different spray thickness: (a) TSL uncoated white sandstone, (b) 2 mm TSL coated white sandstone, (c) 3 mm TSL coated white sandstone, (d) 4 mm TSL coated white sandstone, (e) TSL coated white sandstone with different thickness.

According to the above Figures, the stress of the TSL-coated yellow and white sandstones improved significantly and increased with the TSL-coating thickness. The stress of coated coal also improved slightly, the suitable coating thickness was 2–3 mm.

4.2. Specimen Energy Evolution Analysis

1. Energy evolution analysis method

During the UCS experiment, the evolution of machine energy can be expressed by five different energies: Elastic energy is stored in the rock interior and escapes when the rock is deformed, it is the main source of energy disaster in rocks. Plastic energy occurs owing to the misalignment of the original crack and the yield of a secondary crack. Surface energy is the energy required for crack propagation when the fracture extends; hence, the plastic and surface energies are dissipative. Radiant energy will be produced when the coal and rock is loaded, and is a considerably lower than deformation energy. Kinetic energy will only be generated when the magnitude and rate of elastic energy release reach a certain level. To simplify the energy evolution of the rock during the uniaxial compression, kinetic and radiant energy were not considered. The plastic and surface energies combined to provide the dissipative energy [38–40].

According to some study of researchers, elastic energy is denoted as follows:

$$U_{sp} = \frac{\sigma_p^2}{2E} \quad (1)$$

where σ_p is stress on the curve, U_{sp} is elastic energy on the stress-strain curve, and E is the elastic module. The dissipative energy can be calculated as the area under a stress-strain curve; therefore, the whole energy can be calculated as follows:

$$W_{\Sigma p} = \int_0^{\varepsilon_p} \sigma d\varepsilon \quad (2)$$

$$U = U_{sp} + U_{hp} \quad (3)$$

where σ is the stress of previous point p , ε_p is the strain of point p , U_{hp} is the dissipative energy on the stress-strain curve, and U is the total energy on the curve.

2. Energy evolution analysis of yellow sandstone

As can be seen from Equations (1) and (2), The total, elastic, and dissipative energies can be obtained through the stress-strain curve data. To obtain these energies, Origin software (OriginLab Corporation, Northampton, MA, USA) was used, as shown in Figure 11.

According to the law of conservation of energy, only elastic, and dissipative energies exist in the rock system. The elastic energy is primarily used to resist deformation and the dissipative energy is used for the initiation, expansion and development of the cracks in rock and coal, and their combined macroscopic effect is irreversible deformation.

As evident from Figure 11, the growth rate of initial dissipative energy of the yellow sandstone is higher than that of the elastic energy. In the initial deformation stage, the micro-crack closure and friction in the yellow sandstone consumed maximum energy. However, with an increase in deformation, the micro-cracks gradually closed; hence, the effective contact area inside the yellow sandstone increased, and the elastic energy accumulation and dissipative energy growth rates began to decrease. After the elastic curve crossed the dissipative energy curve, the growth rate of elastic energy gradually became more than that of dissipative energy, and the tendency of elastic energy is consistent with that of total energy. Although the dissipative energy curve increased slowly before the peak, it increased sharply after the peak. The total energy of uncoated yellow sandstone and the energy of those coated with 2 mm, 3 mm, 4 mm, and 5 mm of TSL was 0.2637 MJ/m³, 0.3255 MJ/m³, 0.32023 MJ/m³, 0.33706 MJ/m³, 0.39105 MJ/m³, respectively; the maximal elastic energy was 0.21246 MJ/m³, 0.20142 MJ/m³, 0.21684 MJ/m³, 0.21374 MJ/m³, and 0.16322 MJ/m³, respectively; the maximal dissipative energy was 0.21402 MJ/m³, 0.27522 MJ/m³, 0.29147 MJ/m³, 0.33623 MJ/m³, and 0.35126 MJ/m³, respectively. Therefore, it can be deduced that maximal dissipative energy increased with the coating thickness,

and the ductile of TSL impart the confining pressure to specimen and reinforces to the yellow sandstone.

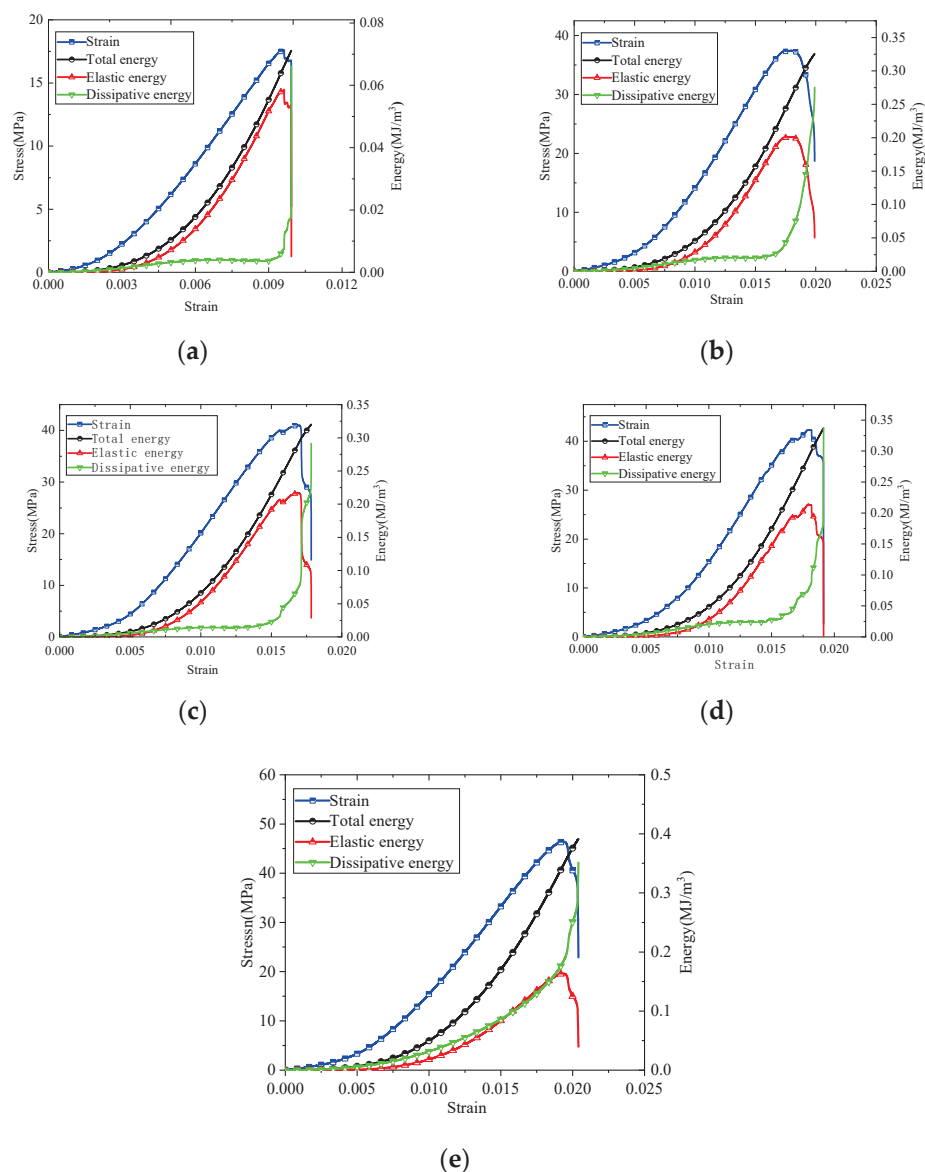


Figure 11. Quantitative analysis of energy dynamics of yellow sandstone with different coated thicknesses: (a) TSL coated 0 mm, (b) TSL coated 2 mm, (c) TSL coated 3 mm, (d) TSL coated 4 mm, (e) TSL coated 5 mm.

3. Energy evolution analysis of coal

The variation trend of the uncoated coal specimen is the same as that of the uncoated yellow sandstone. The initial dissipative energy of the TSL-coated coal is greater than its elastic energy. Some step-like patterns were observed after the peak of dissipative and elastic energy curves of the TSL-coated coal. Compared with the uncoated coal, the dissipative energy growth rate of TSLs coated coal increases greatly and less sharply after the peak. As evident from Figure 12, the total energy of uncoated coal and those coated with 2 mm, 3 mm, 4 mm of TSL was 0.07113 MJ/m³, 0.20807 MJ/m³, 0.18087 MJ/m³, and 0.21907 MJ/m³, respectively; the maximal elastic energy was 0.05863 MJ/m³, 0.07708 MJ/m³, 0.10076 MJ/m³, and 0.12464 MJ/m³, respectively; and the maximal dissipative energy was 0.06604 MJ/m³, 0.19157 MJ/m³, 0.15719 MJ/m³, and 0.18942 MJ/m³, respectively. Therefore it can be inferred that TSL sprayed with high pressure and penetrated the micro-cracks in coal adhered it together. TSL bonded the coal, which is anisotropic, absorbed some energy, and

reinforced its strength. Furthermore, the strength of coal increased with the thickness of the TSL coating.

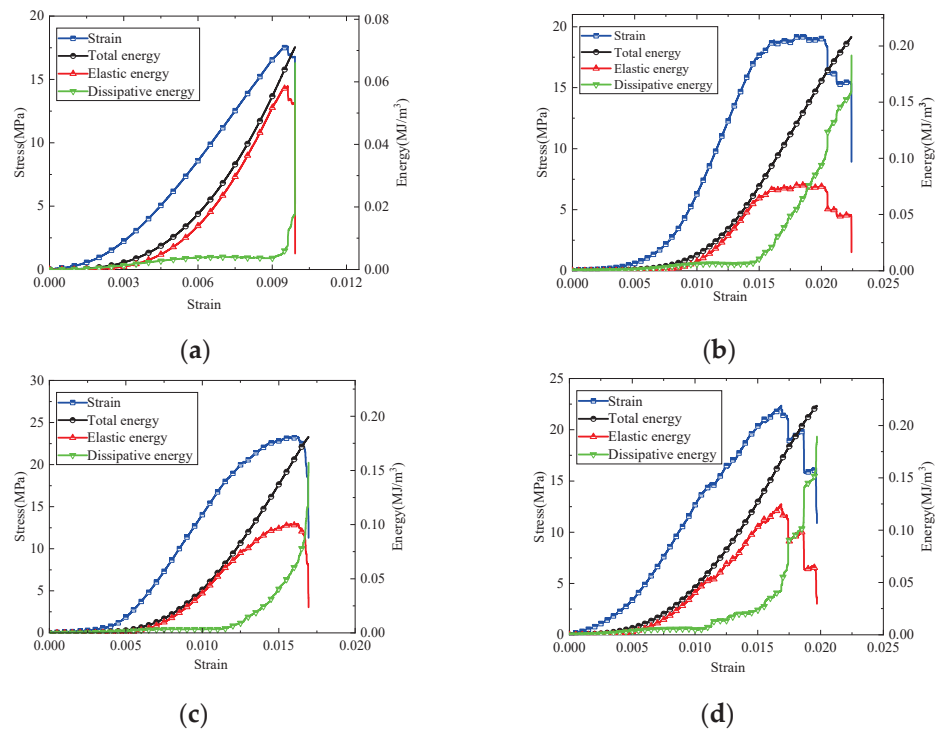


Figure 12. Dynamic energy quantitative analysis of coal with different coated thicknesses: (a) TSL coated 0 mm, (b) TSL coated 2 mm, (c) TSL coated 3 mm, (d) TSL coated 4 mm.

4. Energy evolution analysis of white sandstone

The variation trend of coated white sandstone specimen was the same as that of the coated yellow sandstone. There were some step-like patterns after the peak of dissipative and elastic energy curves of the TSL-coated white sandstone. Although the dissipative energy was low before the peak and increased slowly, after the peak, it increased sharply. As can be seen from Figure 13, The total energy of uncoated white sandstone and those coated with 2 mm, 3 mm, 4 mm was 0.29584 MJ/m³, 0.33794 MJ/m³, 0.33329 MJ/m³, and 0.34513 MJ/m³, respectively; the elastic energy was 0.23926 MJ/m³, 0.19359 MJ/m³, 0.21737 MJ/m³, and 0.23898 MJ/m³, respectively; the dissipative energy was 0.27082 MJ/m³, 0.28955 MJ/m³, 0.28877 MJ/m³, and 0.33785 MJ/m³, respectively. Furthermore, the TSL also significantly improved the residual strength of the white sandstone.

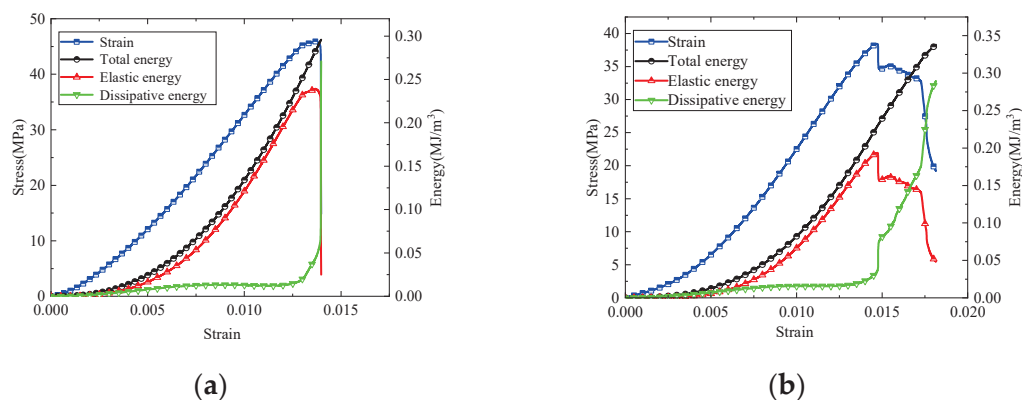


Figure 13. Cont.

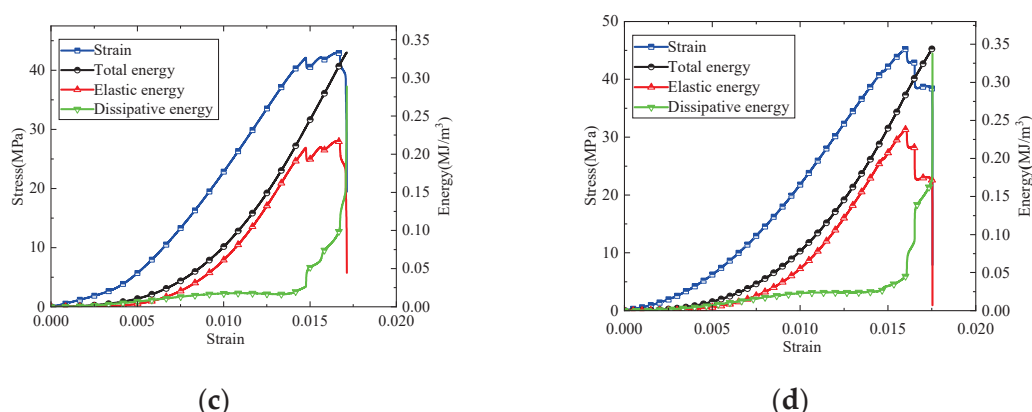


Figure 13. Quantitative analysis of energy dynamics of white sandstone with different spray thickness: (a) TSL coated 0 mm, (b) TSL coated 2 mm, (c) TSL coated 3 mm, (d) TSL coated 4 mm.

5. Discussion

The UCS improvement of TSL-coated coal over uncoated coal was more significant than that of coated white and yellow sandstone over their respective uncoated counterparts. Coal is anisotropic, and its elastic modulus and uniaxial compressive strength are less than that of the white and yellow sandstones. Sandstones are compacted rocks, and the increment of the UCS in their TSL-coated specimen is small unless the coating thickness is increased. The TSL-coated coal specimen appeared “fat” and swollen in the middle under uniaxial compression load, as shown in the Figure 6. The coal specimen could have been completely destroyed without TSL coating.

TSLs can penetrate the cracks and fissures to reinforce the strength of rocks and coal, the film covering on the surfaces of rock and coal has tensile and shear strength to support them.

With an increment in the coating thickness, the total and dissipative energies of the yellow sandstone specimen increased gradually, the elastic energy was nearly unchanged, and the total energy proportion decreased gradually. The total, dissipative, and elastic energies of coal specimen also increased gradually, and the total energy is approximately 2–3 times more than that of the uncoated coal specimen; thus, the difference was distinct. The total energy of white sandstone increases gradually, and the elastic and dissipative energies were nearly unchanged; the dissipative energy did increase a little but this increase was insignificant.

As evident from Figure 12, the total energy is exerted by the machine on the rock before the peak of the uniaxial compression, which indicates the energy absorbing capability of the rock. For coal specimens, because the total and elastic energies have a positive correlation with the coated thickness, when the total energy is added with the increment in the coating thickness, the energy absorbing capability improves. Elastic energy is energy stored when the rock is deformed, and its magnitude is related to the rock’s characteristics and its elastic deformation. The rock will be destabilized and destroyed when the elastic energy which is stored inside the rock, attains its maximum extent.

With an increment in the coating thickness, the elastic energy in the peak of coal is increased, which indicates that both the energy of storage capacity and the energy-absorbing capacity increased. For the yellow sandstone, with an increment in the coating thickness, the total energy in the peak of yellow sandstone increased gradually but the elastic energy was nearly unchanged; therefore, the energy-absorbing capacity was increased. The total, elastic, and plastic energies of white sandstone were not varied; therefore, the energy-absorbing capacity was nearly constant. The energy-absorbing capacity of coal gradually decreased, and its strength reduced.

TSLs can improve the strength and total energy capacity of coal and rocks through different coating thickness, which can maintain and reinforce the stability of the coal and rocks.

The suitable coating thickness is approximately 2–3 mm considering its cost and operation time. The TSLs' performance can be adjusted based on the on-site different requirement.

6. Conclusions

In this paper, TSLs were prepared by controlling the mixing ratio of slurry with high precision, and uniaxial compression experiments were performed on the different thickness combinations of TSL coating on coal and rocks. The improvement in the mechanical properties of the TSL-coated rock specimens and the evolution law of elastic and dissipation energy during the failure process of the sample were analyzed. The following conclusions can be drawn from the study:

- (1) The hard and soft segments of TSLs cause microphase separation and form the physical cross-linking, as seen in their microscopic images. The hard segment was dispersed evenly in the soft segment, which improved the mechanical strength of the TSL.
- (2) The UCS improved with an increased thickness of TSL coating on coal, yellow sandstone, and white sandstone, and the improvements were more obvious because UCS was minimal. The UCS increment was 4.6%, 25.51%, and 43.75% in white sandstone, yellow sandstone, and coal, respectively. The confining pressure was affected by the TSL-coated coal and rocks, which improved the mechanical strength of the rocks and coal under UCS loading.
- (3) Compared with the uncoated coal, the maximal stress resistance of coal coated with 2 mm, 3 mm, and 4 mm of TSLs improved 34.56%, 43.75%, and 34.75%, respectively. The post-peak pressure curves of TSL-coated coal and rocks were more gradual than that of their uncoated counterpart, and they had a step-like pattern, which indicated that the TSL significantly improved the residual strength of the coal and rocks, and increased the reinforcement effect of the confining pressure exerted by the TSL on the coal specimen. The reinforcement of TSLs will be weakened if they are debonded or delaminated from the coal and rock.
- (4) With an increment in the coating thickness, the maximal total, and dissipative energies of yellow sandstone specimen were gradually increased, and the maximal elastic energy was nearly unchanged; The maximal total, elastic, and dissipative energies of TSL-coated coal increased 2–3 times than that of uncoated coal; thus, the increment in coated coal was significant. For the white sandstone, the maximal elastic energy was nearly unchanged, and the maximal total and dissipative energies increased. The energy-absorbing capacity of the coated rocks gradually reduced as the rock strength decreased, and it was higher in coal than that in the yellow and white sandstone. Because coal is an anisotropic material, its total energy is weaker than that of yellow and white sandstones. The TSL reinforcement was significantly more effective for coal.

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Article

Comprehensive Water Inrush Risk Assessment Method for Coal Seam Roof

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Abstract: In order to prevent coal mine water inrush accidents, it is necessary to appropriately assess the water abundance of coal mines based on drilling and geophysical data. This paper studied a comprehensive risk assessment method of water inrush. First, a water inrush risk index was proposed based on the analytic hierarchy process-entropy method (AHP-EM) and the water-rich structure index was proposed based on the geological data coupled calculation, then weighted two indices above which established the comprehensive water inrush risk assessment method. Secondly, eight factors were chosen as risk control factors of water inrush: core recovery, aquifer thickness, distance from the indirect aquifer to the coal seam, aquiclude thickness, height of water-conducting fracture zone, sand-mud ratio, total layers of aquifer and aquiclude, and the equivalent thickness of sandstone. Finally, the No. 2 coal seam of Dahanze coal mine was taken as the research object, the factors were calculated, and a comprehensive water inrush assessment model was constructed. With site investigation and observation, the water inrush risk assessment model of the No.2 coal seam roof is consistent with the actual mining situation, which verifies the validity of the model. In addition, this method was used to evaluate the water-richness of the weathered bedrock fractured aquifer in the Zhangjiamao coal mine. The practical application of the two mines has verified the generality of the approach. The research could provide scientific assistance for mine water hazard mitigation and mining safety.

Keywords: risk assessment; water inrush risk index; water-rich structure index; Dahanze coal mine; Zhangjiamao coal mine

1. Introduction

Coal resources still make up a significant proportion of China's energy consumption. The development of China's economy is inextricably linked to the extraction of coal resources [1]. However, since the 21st century, hundreds of water inrush accidents have occurred in China's coal mines, which seriously affected the safety of coal miners and coal production [2]. The Jurassic coal field of Ordos Basin is the one of the 14 large-scale coal bases in China. However, its mine production and construction are generally affected by roof water hazards. Water risks in coal mines are caused by a variety of factors. Coal mine hydrogeological conditions are not clear, and the groundwater occurrence lacks in-depth research; furthermore, the water-rich areas lack scientific predictions. Resolving the potential groundwater yield zonation of sandstone aquifers occurring at depths of several hundred meters has been an important and challenging objective of the hydrogeological research focused on preventing flood hazards in coal mines [3]. Although, many studies have been carried out on mine hydrogeology, there are still many problems such as imperfect groundwater hazard prevention plans and measures, abnormal water inrush, and water inrush in coal mines [4]. In the early stage of mine construction, fully understanding hydrogeological conditions of the mine would benefit water hazard prevention and shorten the construction period.

Scholars have conducted extensive research in the Ordos Basin Jurassic coal field on the prevention and control of roof and floor water inrush hazards in coal mining. The existing studies mainly focus on numerical simulation, theoretical analysis, and laboratory tests. Chen et al. [5] used the Visual Modflow, which predicted water inflow in the first mining working face of Dahaize coal mine, Yulin. Gao et al. [6] used PHASE to analyze the water inflow with different influencing factors and used SPSS software for nonlinear regression analysis to obtain the evaluation model of control factors of coal seam water inrush. Zeng et al. [7] obtained the relative important control factors through the coupling calculation of the analytic hierarchy process (AHP) to compile the thematic map of water-rich evaluation, and finally superimposed on the thematic map of roof fracture safety evaluation to comprehensively evaluate the risk of water inrush.

Many academics have investigated the hazard analysis of coal mine water inrush in other regions. Wu et al. [8,9] proposed the “three maps-two predictions” method to solve the quantitative evaluation of water inrush in coal roofs, which has been successfully applied in the Jinggezhuang coal mine and Donghuantuo coal mine. Nampak and Lee et al. [10,11] applied the Dempster–Shafer theory of evidence model to the groundwater productivity potential map, using the geographic information system (GIS) technique. Subsequently, Ruan et al. [12] used the Dempster–Shafer evidence theory to expand the improved AHP, obtained the weight coefficient of influencing factors, and evaluated the risk of water inrush in the air return roadway of Wangjialing coal mine. Cheng et al. [13] established a prediction and assessment model of roof water inrush risk in multi-coal seam mining by combining the traditional AHP and the entropy method (EM) using the least square method. Lu et al. [14] used the fuzzy analytic hierarchy process (FAHP) and EM to analyze the hydrogeological and engineering geological conditions of Yangliu coal mine, obtained the overall weighting of each factor, and created a risk assessment zoning map using the spatial superposition capability of GIS. To determine the location of water-rich areas, Zhang and Wang [15,16] applied Fisher Discriminant Analysis (FDA) dimensional reduction technology to project original normalized data into low-dimensional space and create a thorough zoning map of water-rich levels in the study area. Bo et al. [17] combined the risk evaluation index system with set pair analysis (SPA), integrated quantitative and qualitative indicators, and established the risk evaluation model of coal floor water inrush. In order to prevent and cure water inrush from sealed holes in the wellbore, Han et al. [18] proposed a new risk assessment model and concentric annular hole grouting method, predicted the water inflow area, and effectively inhibited the groundwater migration of multiple aquifers.

Some academics have done extensive research to address the problem of water disasters caused by geological circumstances of thin bedrock and a thick loose layer. Yang et al. [19] constructed a fuzzy comprehensive evaluation model to study the water inrush caused by coal mining. Zeng et al. [20] introduced the methods of information entropy and unascertained measure to establish a prediction model for coal mine safety. Wang et al. [21] combined the extensive evaluation method with borehole measurement data to establish a matter element model for the evaluation of coal mine safety. Lian et al. [22] studied the causes of water inrush from loose aquifers, and established the importance ranking and correlation degree of each influencing factor, accurately assessed the risk of water inrush from loose aquifers, and improved theoretical approaches for water inrush accident prediction and prevention.

The roof water hazard prediction and assessment processes have been used for many years, such as the “three maps-two predictions” method, nonlinear regression analysis, FDA model, AHP, EM, etc. The AHP only considers the experience evaluation of experts. The EM only uses objective data to calculate weights. However, there has been little research on the prediction of water inrush from coal seam roofs coupling of multiple evaluation methods, and the results need to be improved to meet the requirements of fine study of mining hydrogeological conditions. In this study, developed the water inrush risk model using the AHP-EM and proposed the water-rich structure index model based on the geological data-coupled calculation then weighted two indices above, which established

the comprehensive water inrush risk assessment method. It is critical to provide scientific guidance for mine water hazards prevention and control.

2. Materials and Methods

2.1. Geological and Hydrogeological Conditions

Dahaize Coal Mine is located in Yulin city, Shaanxi Province, China (Figure 1). It is about 14.7 km wide from north to south and 12.5–23.3 km long from east to west, covering an area of about 280 km². The coal-bearing stratum of the mine is the Yan'an Formation of Jurassic, and the Dahaize coal mine is designed to produce 15 million tons per year. The Dahaize coal mine belongs to the Jurassic coal field of Ordos Basin. The mine production is greatly affected by roof water hazards. Therefore, it is urgent to fully understand the mine hydrogeology and improve prevention methods and control measures for water hazards.

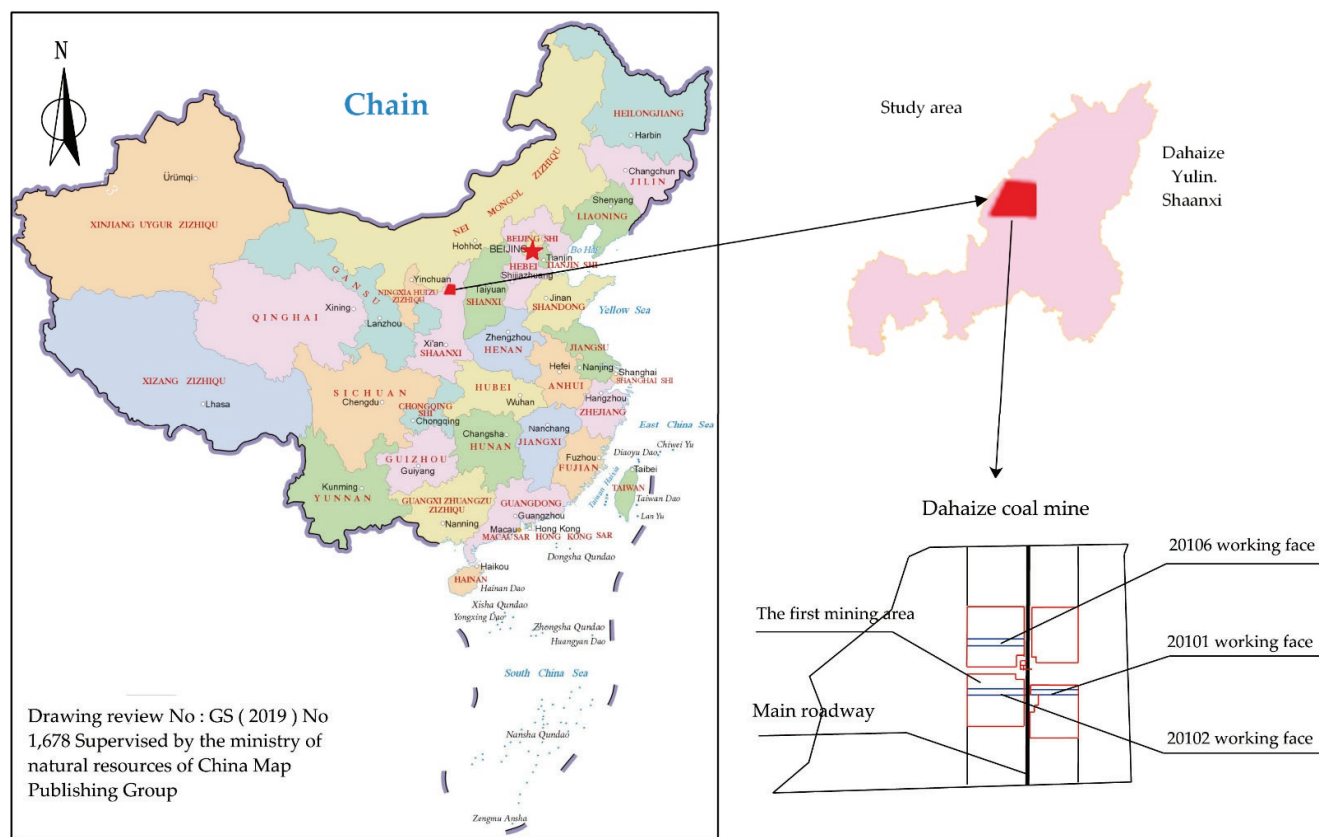


Figure 1. The location of the mining area.

The strata of Dahaize coal mine belong to the Ordos Basin subdivision of northern China, and the sedimentary and geological structure characteristics of each age are nearly consistent with the strata of north China. According to the drilling data, the strata are successively from old to new as Middle Jurassic Yan'an Formation (J_{2y}), Middle Jurassic Zhiluo Formation (J_{2z}), Middle Jurassic Anding Formation (J_{2a}), Lower Cretaceous Luohe Formation (C_{1l}), Neogene system (N_{2j}), and Quaternary system (Q). The mine is located in the center of the northern slope of Shaanxi, a secondary structural unit in the central Ordos Basin. On the whole, the stratum is slightly dipped westward, the dip angle of the stratum is nearly horizontal, and the folds are not developed. There are no major faults in the mine, and there is no magmatic intrusion, which results in the simple type of the strata structure type. The No. 2 coal seam of Dahaize coal mine is shallowly buried, covers the whole study area, and gradually becomes thinner from south to north with an average thickness of 5.95 m. According to groundwater occurrence conditions, the water in the aquifers of Dahaize coal mine can be classified into the following three categories: the unconsolidated rock

pore phreatic water of Quaternary, the fracture or pore confined water of Cretaceous, and the fracture confined water of Jurassic. The geological profile of the mining area is shown in Figure 2. Petrological characteristics, hydraulic properties, tempo-spatial structure, and variation characteristics of aquifer and aquiclude are the basic hydrogeological conditions.

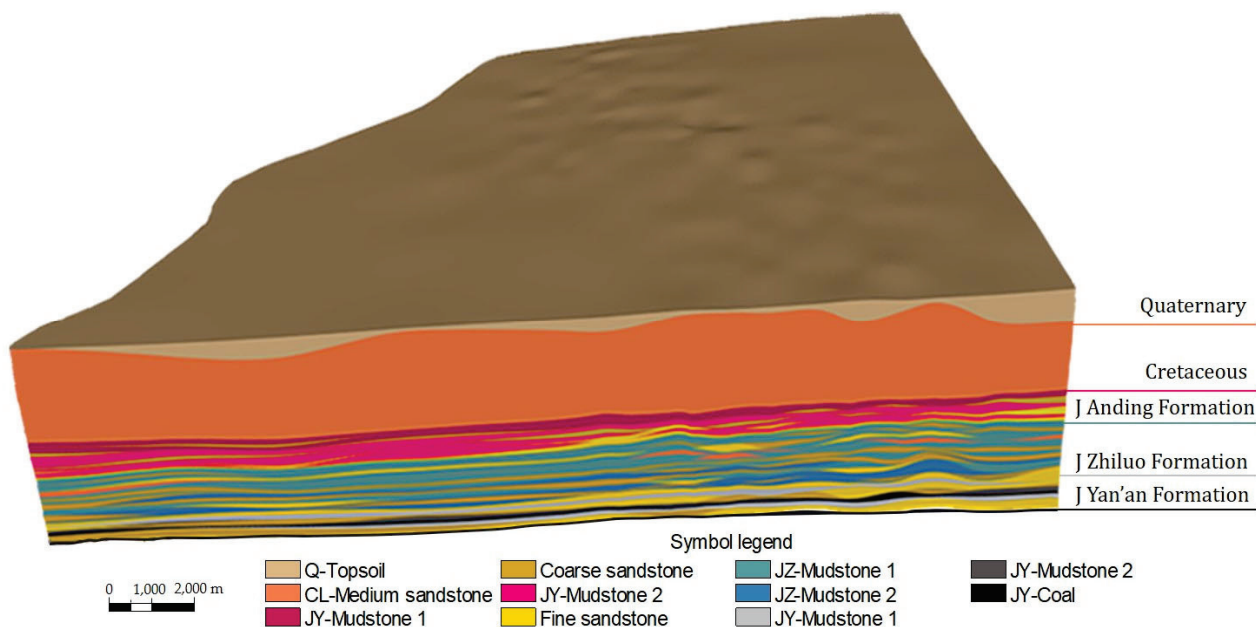


Figure 2. Geographical profile of mining area.

The aquifers are mainly composed of (gravel) coarse grained sandstone, medium grained sandstone, and fine sandstone. The microscopic characteristics of these aquifers are the developed pores, the good connectivity, the low density, the high effective porosity, and the good permeability. The average resistivity of aquifers is greater than $50\Omega\cdot\text{m}$. The logging curve of the sandstone section shows high resistivity, although they have fluctuation. The average resistivity is lower than that of mud rock, and the logging curve is concave, which means that is a low resistivity stratum in the Jurassic range and can be judged as an aquifer [23]. The aquicludes are mainly composed of some fine-grained sedimentary rocks with argillaceous (silty) sandstone, silty mudstone, sandy mudstone, and mudstone. The microscopic characteristics, resistance, and potential values of these rocks are the opposite of aquifers.

2.2. Comprehensive Water Inrush Risk Assessment Method

A new approach for forecasting and evaluating the water hazard of coal seam roofs is provided based on the geologic and hydrogeological characteristics of the Dahaize coal mine. The working steps of the methodology shown in Figure 3. These are the evaluation steps.

1. The water inrush risk index was established by combining AHP and EM with the moment estimation method;
2. The water inrush structure index was used to calculate the water-rich in the study area, and establish the water inrush model;
3. The weighted superposition of the above two models were carried out by using the weighted formula to establish a comprehensive water inrush risk assessment model.

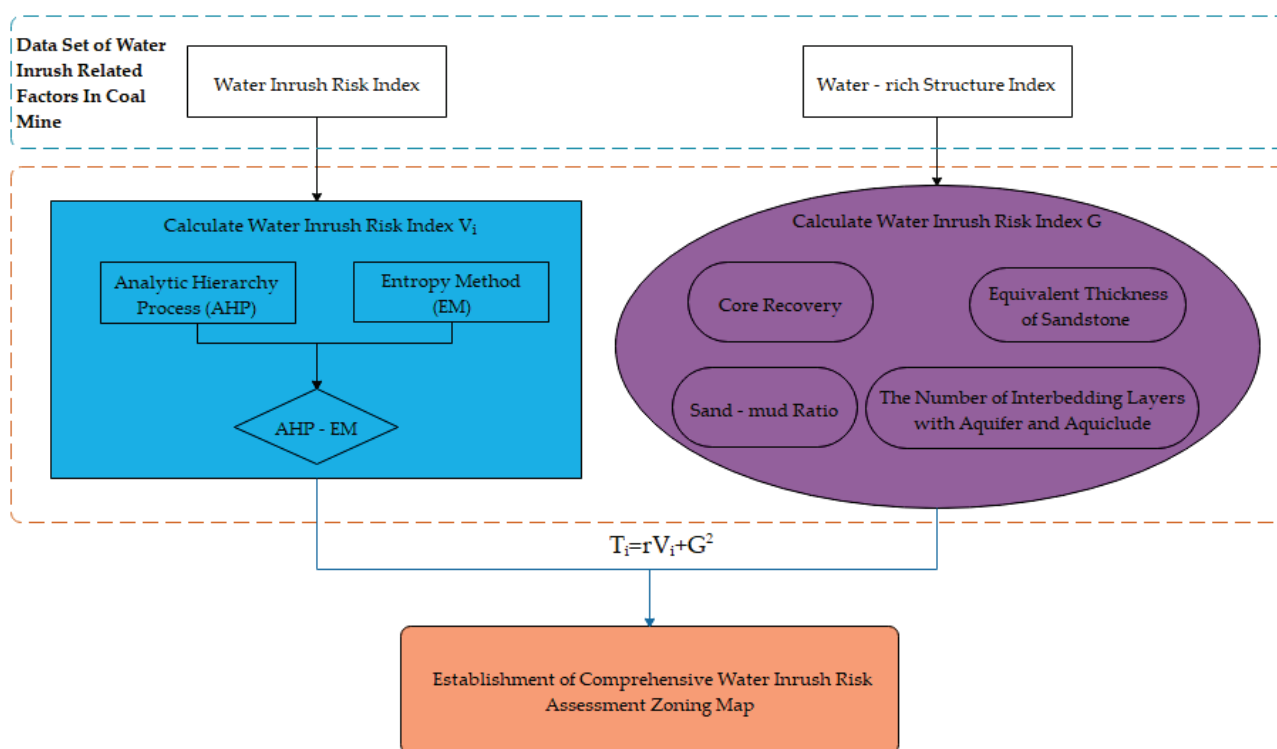


Figure 3. The working steps of the methodology.

By analyzing borehole exploration data (i.e., geophysical data, underground mining data) from the research area, and calculating the data of various control factors, the basic geological data of water inrush in the Dahaize mining area were obtained. The data used to support the results of the study are available from the corresponding researchers according to the requirements. It is important to normalize the data to avoid the effect of the different dimensions and units of the data on the analysis outcomes [24]. After the standardization, the source data were of the same scale, so it could be a good starting point for a full comparison. The basic data could be normalized by Equations (1) and (2).

$$r_{ji} = \frac{x_{ij} - \min_j(x_{ij})}{\max_j(x_{ij}) - \min_j(x_{ij})}, \quad (1)$$

$$r_{ji} = \frac{\min_j(x_{ij}) - x_{ij}}{\max_j(x_{ij}) - \min_j(x_{ij})} \quad (2)$$

where r_{ji} represents the normalized value of the j th evaluation object in the i th evaluation index, and x_{ij} represents the initial value.

2.2.1. Water Inrush Risk Index

Analytic Hierarchy Process (AHP)

The AHP is more suitable for decision-making problems where the target system has hierarchical and interlaced evaluation indicators, and the target value is difficult to quantitatively describe [25,26]. The judgment matrix was created by analyzing the influence of related factors on the target and determining the weight of each element on the target. At present, a great number of studies on AHP and its application in diverse domains have been undertaken by many professionals and scholars [27,28]. The following steps are usually taken when using the AHP to calculate the weight of indicators:

On the basis of the scale type, the relative importance of indicators at each level was quantified by using expert experience, and the judgment matrix $T = (a_{ij})_{n \times n}$ was constructed, as shown in matrix (3).

$$T = \begin{bmatrix} a_{11} & a_{12} & \cdots & a_{1n} \\ a_{21} & a_{22} & \cdots & a_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ a_{n1} & a_{n2} & \cdots & a_{nn} \end{bmatrix} \quad (3)$$

where $a_{ij} > 0$, and $i, j = 1, 2, \dots, n$.

x_i is obtained by multiplying each row vector of the judgment matrix; the root of x_i is calculated n times to obtain $\bar{q}_i$; finally, the relevant weight q_i is obtained by normalization. The calculation steps are as in Equation (4).

$$x_i = \prod_{j=1}^n a_{ij}, \bar{q}_i = \sqrt[n]{x_i}, q_i = \frac{\bar{q}_i}{\sum_{i=1}^n \bar{q}_i}, (i = 1, 2, \dots, n) \quad (4)$$

In the end, $q = (q_1, q_2, \dots, q_n)^T$ is the required weight vector.

Entropy Method (EM)

The EM is an objective weighting approach to determine the weight of indicators according to the correlation degree between indicators or the amount of information contained in them [29,30]. In general, the greater the difference between indicators are, the lower the formation entropy weight is, indicating that the more effective information indicators provided, the higher the weight of effective information was to indicators [14]. The specific steps of evaluating weight by EM are as follows:

If there are n evaluation indicators and m evaluation objects, the original data matrix was standardized by Equation (1) to obtain the normalized data matrix R (5).

$$R = (r_{ij})_{m \times n} = \begin{bmatrix} r_{11} & r_{12} & \cdots & r_{1n} \\ r_{21} & r_{22} & \cdots & r_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ r_{m1} & r_{m2} & \cdots & r_{mn} \end{bmatrix} \quad (5)$$

The entropy of the i th index was defined by Equations (6) and (7) when evaluating m objects with n indexes.

$$e_i = -k \sum_{j=1}^n f_{ij} \ln f_{ij}, \quad (6)$$

$$f_{ij} = \frac{r_{ij}}{\sum_{j=1}^n r_{ij}}, k = \frac{1}{\ln n} \quad (7)$$

where $i = 1, 2, \dots, m$; $j = 1, 2, \dots, n$. and if $f_{ij} = 0$, $f_{ij} \ln f_{ij} = 0$

After the entropy value of i th index, the entropy weight of the i th index was defined by using Equation (8).

$$u_i = \frac{1 - e_i}{n - \sum_{i=1}^n e_i} \quad (8)$$

where $0 \leq u_i \leq 1$, $\sum_{i=1}^n u_i = 1$.

AHP-EM

When it comes to calculating index weights, the AHP is more subjective because it relies purely on expert opinion and ignores actual geological data. Furthermore, the EM is more objective to calculate the index weights as it considers the geological data [13]. Therefore, the AHP and EM were combined based on the moment estimation method in

this research, and the AHP-EM weight of each factor was derived as a water intrush risk index. The optimization model is as follows:

For each index K_i ($1 \leq i \leq n$), there is:

$$\alpha_i = \frac{q_i}{q_i + u_i}; \beta_i = \frac{u_i}{q_i + u_i}, \quad (9)$$

$$\alpha = \frac{\sum_{i=1}^n \alpha_i}{\sum_{i=1}^n \alpha_i + \sum_{i=1}^n \beta_i}; \beta = \frac{\sum_{i=1}^n \beta_i}{\sum_{i=1}^n \alpha_i + \sum_{i=1}^n \beta_i} \quad (10)$$

Then, AHP-EM index combination weight is:

$$\omega_i = \alpha \times q_i + \beta \times u_i \quad (11)$$

where $\alpha, \beta > 0$ and $\alpha + \beta = 1$, $i = 1, 2, \dots, n$.

The normalized factors were evaluated using Equation (12) to calculate the mine water intrush risk model.

$$V_i = \sum_{j=1}^n \omega_j f_j(x, y) \quad (12)$$

2.2.2. Water-Rich Structure Index

Considering the factors such as aquifer hosting space, lithology, and hydraulic characteristics, the method of water-rich structure index was introduced to evaluate the water-rich of aquifer in the study area. The water-rich index method could fuse the selected index data and make a quantitative evaluation. The key point was to convert the thickness of aquifer and the properties of rock particles into a unified clastic thickness by mathematical expressions, at the same time considering the layers of sand-mudstone interaction, core recovery, and sand-mudstone ratio. The water abundance was evaluated after standardized data processing. The specific steps are as follows:

Different sandstones have different water-bearing capacities. The calculation formula of sandstone equivalent thickness is as Equation (13).

$$L = s_1 \times 1 + s_2 \times 0.8 + s_3 \times 0.6 \quad (13)$$

where L is the equivalent thickness of sandstone, s_1 is the coarse sandstone, s_2 the medium sandstone, and s_3 the fine sandstone.

Through the analysis and establishment of the evaluation index structure of aquifer water abundance, the indexes are sandstone and sand-mud ratio, which are positively correlated with aquifer water abundance, while the negative correlation indexes are sand-mudstone interaction layers and core recovery. As a result of the above analysis, the expression of the water-rich structure index model was constructed as follow Equation (14).

$$G = \frac{L \times r}{K_1 \times N} \quad (14)$$

where G is the water-rich structure index, L is the equivalent thickness of sandstone, r is the sand-mud ratio, K_1 is the core recovery, and N number of sand-mudstone interbeds.

2.2.3. Comprehensive Water Intrush Risk Assessment

The AHP-EM weight method not only comprehensively considered the experience evaluation of experts but also made full use of the actual geological data; the water-rich structure index calculated the water-filled strength of aquifer by using the formula. The basic geological data were used by the two methods, which are related to each other. In this paper, a new approach was proposed to comprehensively assess the risk of water intrush. From the approach, the risk of water intrush with a strong water-rich index from AHP-EM weight method was not as great as the region from AHP methods.

After normalizing T_i , the water inrush comprehensive risk assessment index was obtained rT_i . The comprehensive assessment model weighted by the two models improves the accuracy and fault tolerance of the model.

$$T_i = rV_i + G_i^2 \quad (15)$$

where T_i is the comprehensive water inrush risk assessment index; rV_i is the normalized water inrush risk index; and G_i is the water-rich structure index.

2.3. Determination of Risk Control Factors of Comprehensive Water Inrush in Dahaize Coal Mine

The water inrush of the coal roof occurs mainly in the process of mining, with the aquifer at the top of coal seam gushing down from the weak part of the stratum and flooding the mine roadway. The reasonable selection of risk control variables determined the accuracy of the risk analysis and evaluation of water inrush of coal seam roof. Therefore, on the basis of the hydrogeological and geological conditions of Dahaize coal mine, eight comprehensive evaluation factors were selected considering the drilling and pumping test results.

The control factors of water inrush risk index include core recovery, aquifer thickness, distance between indirect aquifer and coal seam, aquiclude thickness, and height of water-conducting fractured zone.

The control factors of the water-rich structure index include core recovery, the number of interbedding layers with aquifer and aquiclude, sand-mud ratio, and equivalent thickness of sandstone.

The factors, which are positively correlated with coal seam water inrush, are aquifer thickness, sand-mud ratio, and equivalent thickness of sandstone. The factors, which are negatively correlated with coal seam water inrush, are core recovery, distance from the indirect aquifer to coal seam, aquiclude thickness, the height of water-conducting fracture zone, and the number of interbedding layers with aquifer and aquiclude. In other words, the larger the positive correlation value, the greater the chance of separate bed water inrush; the lower the correlation coefficients value, the lower the risk of water inrush.

2.3.1. Control Factors of Water Inrush Risk Index

Core Recovery (K1)

The core recovery is the percentage of the total length of the core and the current footage, which reflects the integrity of the rock mass. When the core recovery is larger, the fracture development is worse and the rock layer is more intact. When the core recovery is smaller, the rock is more fractured, the water-bearing performance is better, and the risk of water inrush is higher. The core recovery could be as an index to evaluate water inrush, and the normalized thematic map is shown in Figure 4a.

Aquifer Thickness (K2)

The aquifers are mainly composed of (gravel) coarse grained sandstone, medium-grained sandstone, and fine sandstone. The thickness of various sandstones is an important index for determining aquifer capacity. The thicker sandstones formation, the larger the space to accommodate water and the higher water inrush risk of the mine. Therefore, the aquifer thickness was selected as the evaluation index, and the normalized thematic map is shown in Figure 4b.

Distance from Indirect Aquifer to Coal Seam (K3)

An indirect aquifer has hydraulic links to a directly connected aquifer. As a result of coal mining, water in the indirect aquifer may supplement the direct aquifer and eventually flow into the working face, posing a threat to coal mine workers' safety. The distance between the coal seam and the indirect aquifer can be used to assess the indirect aquifer's impact on mine water inrush. The closer the coal seam is to the indirect aquifer, the greater the risk of mine water inrush. The normalized thematic map is shown in Figure 4c.

Aquiclude Thickness (K4)

Mudstone and siltstone have poor permeability and can be regarded as effective water-repellent media. In the mining process, mudstone in roof overburden can be used as a good waterproof layer, which could effectively block the channel between the recharge source and the working face space, preventing aquifer water from flowing into the working face. Therefore, it affects mine water inrush, and the normalized thematic map is shown in Figure 4d.

Height of Water-Conducting Fracture Zone (K5)

Coal seam mining can form water-conducting fissure, which is the main channel for mine water to enter the working face. The most important parameter reflecting the influence range of mining disturbance is the height of the water-conducting fractured zone. Furthermore, it has a direct effect on mine water inrush. Many researchers and academics have now obtained the empirical formula for calculating the development height of a coal seam water-conducting fracture zone under various combination conditions through measurement and data analysis of the "upper three zones". According to empirical Formula (16) [31,32], the maximum height of the water-conducting fractured zone of the coal seam can be determined according to the thickness of the coal seam. The maximum height of the water-conducting fractured zone of the coal seam can be calculated using the coal seam thickness.

$$H = \frac{20\sqrt{M}}{n} + 10 \quad (16)$$

where M is the thickness of the coal seam (m), H is the height of water-conducting fractured zone (m), and n is the number of mining layers. The normalized thematic map is shown in Figure 4e.

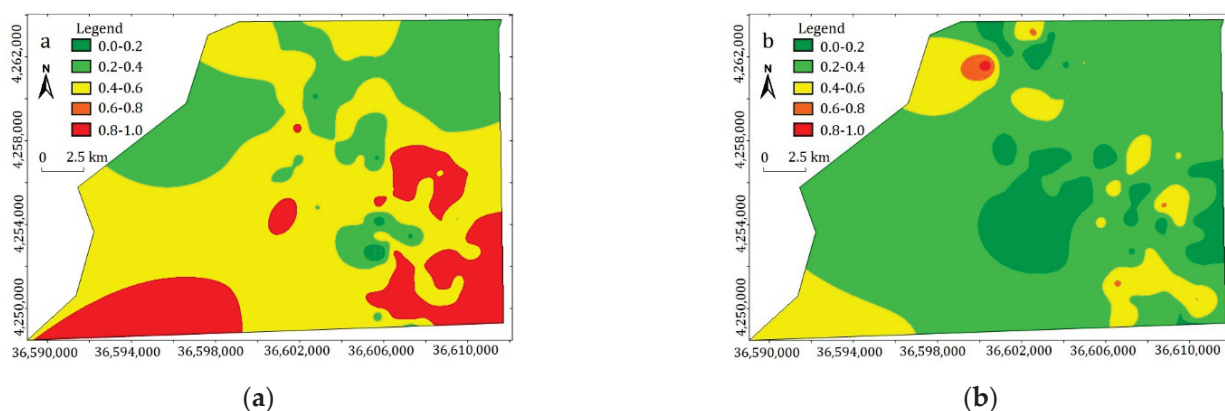


Figure 4. Cont.

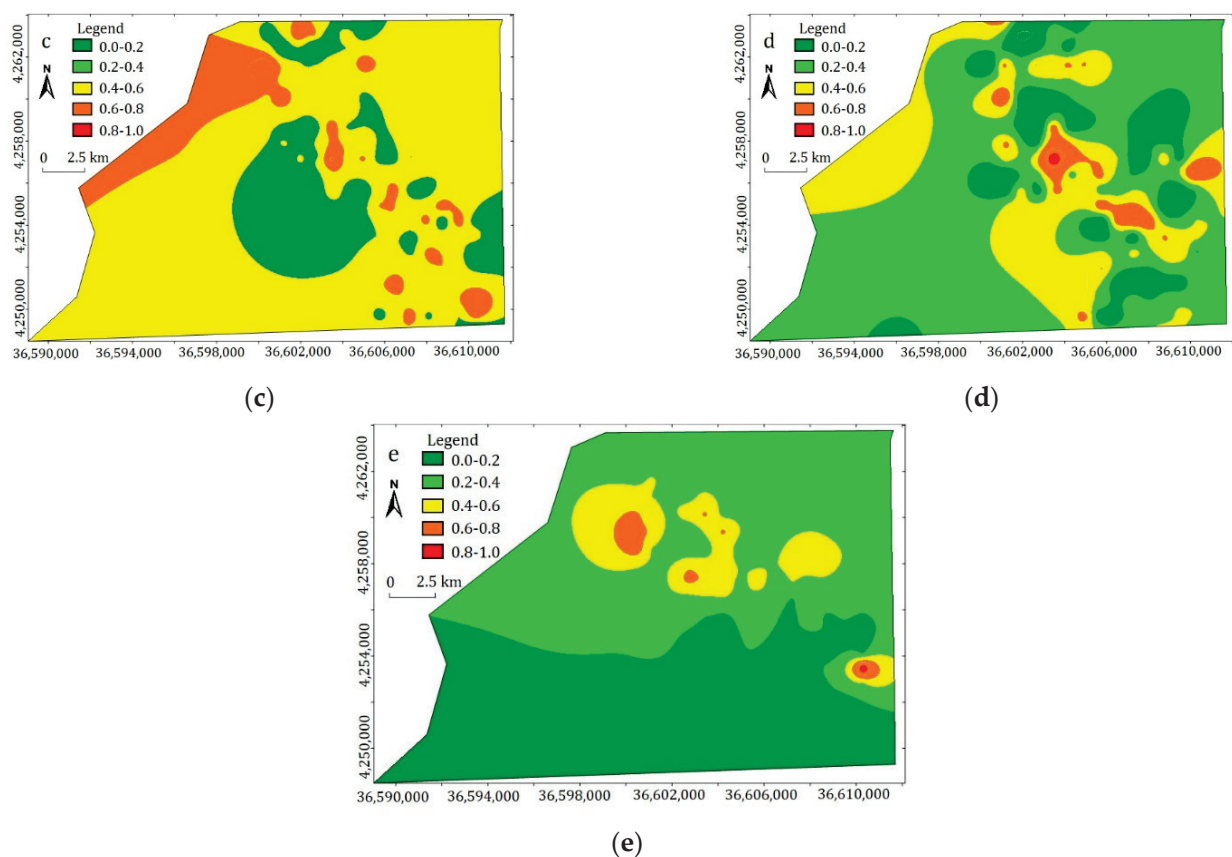


Figure 4. Water inrush risk control factors are depicted on a thematic map. (a) Core Monitor Rate; (b) Aquifer Thickness; (c) Distance from Indirect Aquifer to Coal Seam; (d) Aquiclude Thickness; (e) Height of Water-conducting Fracture Zone.

2.3.2. Control Factors of Water-Rich Structure Index

Core Recovery (K1)

It is consistent with the analysis of risk control factor K1 of water inrush.

The Number of Interbedding Layers with Aquifer and Aquiclude (N)

Whether it is a loose or hard aquifers, the aquifer is not integral, but segmented. There is a layer or multi-layer-resisting section within the aquifer. Although these isolation sections are not distributed regionally, they can also isolate water in aquifers. When there is less interaction between aquifer and aquiclude, the water abundance is stronger and there is higher risk of water inrush.

Sand-Mud Ratio (s)

Sand-mud ratio is the ratio of sandstone thickness to mudstone (siltstone) thickness in the target strata group, which is an indirect reflection of the sandstone thickness and the interaction of layers interaction between aquifer and aquiclude. That means the greater the sand-mud ratio, the more the sandstone content, the thinner the mudstone aquifer, the weaker the water insulation, and the higher the risk of mine water inrush. Therefore, the proportion of sandstone in aquifer group was chosen as one of the factors to control roof water hazards.

Equivalent Thickness of Sandstone (L)

The water abundance capacity of sandstone with different grain sizes is also different, among which the water abundance capacity of coarse sandstone is the strongest, medium sandstone is the second, and fine sandstone is the weakest. The equivalent thickness of sandstone is the total thickness of coarse sandstone after multiplying the thickness of sandstone with different particle sizes by a certain proportion coefficient. The equivalent thickness of sandstone is the important index to determine the aquifer water abundance capacity. When its thickness is larger, its space holding water is relatively larger, and the water content would be higher. Therefore, the equivalent thickness of sandstone was selected as an evaluation index of water abundance.

3. Results and Discussion

3.1. Calculation Results of Water Inrush Risk Index

3.1.1. Weight Calculation Based on AHP

Researchers have examined the elements influencing water inrush and used AHP to calculate the index weight q . The evaluation criteria were based on a 1–9 level. Saaty created the judging matrix T [24], as shown in Table 1. The judgment matrix suggests that the weight vector of every factor was calculated as shown in Table 2. The maximum eigenvalue $\lambda_{\max} = 5.207$ was calculated. To determine the random consistency ratio of matrix T , perform the following operations.

$$C.R = \frac{C.I}{R.I}, \quad (17)$$

$$C.I = \frac{\lambda_{\max} - m}{m - 1} \quad (18)$$

where $\lambda_{\max}$ is the maximum eigenvalue of T ; m is the number of evaluation indicators. The eigenvalues of random judgment matrix were repeatedly calculated more than 500 times, and then the arithmetic average of eigenvalues, namely the random consistency index was calculated. When $m = 5$, $R.I = 1.12$, then $C.R = 0.0462 < 0.1$, so the judgment matrix met the consistency test. After normalized the row vector of judgment matrix, the weight value q_i of each indicator was obtained using Equation (4), as shown in Table 2.

Table 1. Single-level AHP judgment matrix of water inrush factors.

Factor	K1	K2	K3	K4	K5
K1	1	1/4	2	1/3	1/4
K2	4	1	3	2	1/2
K3	1/2	1/3	1	1/2	1/3
K4	3	1/2	2	1	1/2
K5	4	2	3	2	1

3.1.2. Weight Calculation Based on EM

According to the 129 drilling wells exploration data of Dahaize coal mine, the data of influencing factors on the roof of the third member of Jurassic Yan'an Formation were sorted out from the No.2 coal mine. After normalization, the data matrix R was constructed, and Equations (7) and (8) were solved to obtain entropy and entropy weight of the matrix. The entropies were: $e_1 = 0.993$, $e_2 = 0.978$, $e_3 = 0.986$, $e_4 = 0.978$, $e_5 = 0.968$. The entropy weights u_i are shown in Table 2.

Table 2. The influencing factors weight of Dahaize coal mine.

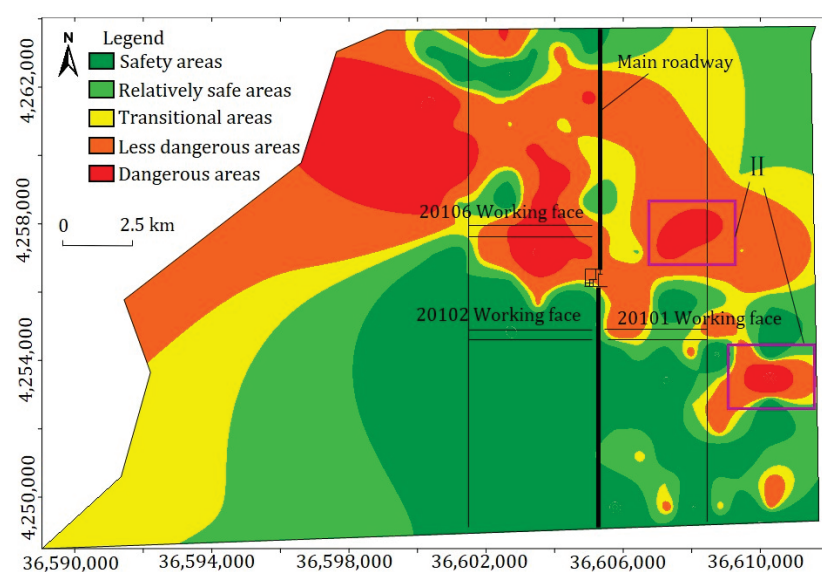
Weight	K1	K2	K3	K4	K5
AHP (q_i)	0.0923	0.2770	0.0835	0.1818	0.3654
EM (u_i)	0.0725	0.2244	0.1477	0.2235	0.3319
AHP-EM (ω_i)	0.0821	0.2502	0.1163	0.2031	0.3483

3.1.3. Weight Calculation Based on AHP-EM

The moment estimation method was used to couple AHP and EM with Equations (9)–(11), and the AHP-EM comprehensive weight ω_i of each factor was obtained as shown in Table 2. The importance of each evaluation index is determined by its weight. In Table 2, normalized factors could be evaluated using Equation (12). The risk index model of water intrusion was established in the study area (Equation (19)).

$$V_i = 0.0821f_1(x, y) + 0.2502f_2(x, y) + 0.1163f_3(x, y) + 0.2031f_4(x, y) + 0.3483f_5(x, y) \quad (19)$$

According to the final calculation results of the risk index model, the relative linear equilibrium classification method was used to classify regions as $0 < V_i \leq 0.334$ safety areas, $0.334 < V_i \leq 0.366$ relatively safe areas, $0.366 < V_i \leq 0.384$ transitional areas, $0.384 < V_i \leq 0.435$ less dangerous areas, and $0.435 < V_i \leq 1$ dangerous areas were prepared, as shown in Figure 5.

**Figure 5.** Water intrusion risk index zoning map.

3.2. Calculation Results of Water-Rich Structure Index

According to Equations (13)–(15), the normalized water-rich structure index value G_i could be obtained. Using the relative linear equalization classification method, the top aquifer from the No.2 coal seam of Dahaize coal mine to the third member of the Jurassic Yan'an Formation could be divided into various water-rich zones. The classification thresholds of G_i were 0.0021, 0.0368, and 0.32. The areas were classified as weak water-rich areas, relatively weak water-rich areas, transitional areas, medium water-rich areas, and relatively strong water-rich areas. The water-rich zoning is shown in Figure 6.

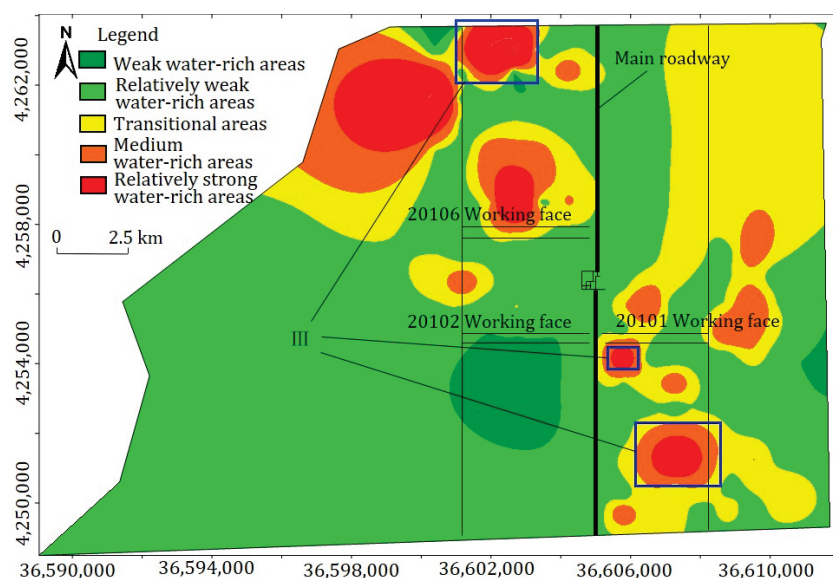


Figure 6. Water-rich structure index zoning map.

3.3. The Result of Comprehensive Water Inrush Assessment

In this paper, the comprehensive risk assessment model was constructed by weighting water inrush risk assessment index and water-rich structure index. The specific steps are as follows Equation (16):

1. Normalize water inrush risk index model V_i to obtain rV_i ;
2. Squared the index (G_i^2), because the overall value of the water-rich structure index is too small;
3. Add the normalized value rV_i to G_i^2 , and establish the comprehensive risk assessment model of water inrush.

The final comprehensive water inrush risk assessment index was normalized to get rT_i . The relative linear equilibrium grading method was used to distinguish the studies $0 < rT_i \leq 0.2068$ as safety areas, $0.2068 \leq rT_i \leq 0.2545$ as relatively safe areas, $0.2545 \leq rT_i \leq 0.3006$ as transition areas, $0.3006 \leq rT_i \leq 0.3760$ as less dangerous areas, $0.3760 \leq rT_i \leq 1$ as dangerous areas (Figure 7).

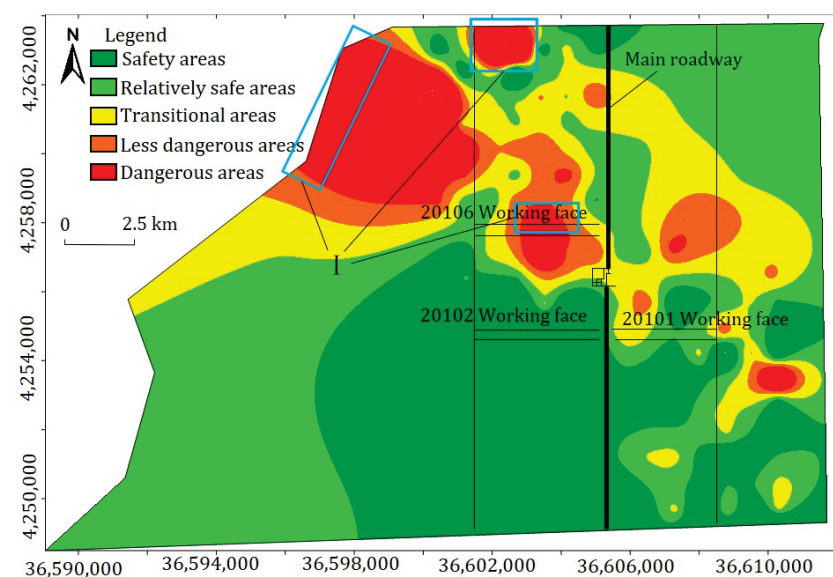


Figure 7. Comprehensive water inrush assessment zoning map.

Dahaize coal mine's first mining area is primarily divided into 20106 working face, 20102 working face, and 20101 working face. According to the water inrush risk assessment diagram, the conclusion is that the water inrush risk area is mainly located in the middle of 20106 working face, and the water abundance of the aquifer in the third member of Jurassic Yan'an Formation gradually increases from southeast to northwest. The actual mining situation of the 20106 working face showed that the water inflow in the middle section of the working face was relatively large, but other unmined areas cannot be verified in a short time. For the working face of the first mining area, for the areas with high risk of water inrush, under the two conditions of the high-water season and the flat-water season, the drainage should be started one month in advance according to the predicted water inflow. These results verify that the model has reliable practical application significance and provides a basis for the prevention and control of water hazards in the Dahaize coal mine.

3.4. Discussion

The method of water inrush risk of coal seam roof based on AHP has been verified and widely used. The water inrush risk assessment model established only by the AHP is shown in Figure 8. After comparing this method with the comprehensive water inrush risk assessment proposed (CWIRA) in this paper, the water inrush risk areas in the middle section of the 20106 working face and the northwest are basically similar, but the CWIRA has more dangerous areas (I in Figure 7) than the single AHP method. In addition, after comparing the results of the water inrush risk index (WIRI) which was calculated by AHP-EM (Figure 5) and the water-rich structure index (WRSI) (Figure 6), it was found that the two indexes were basically the same in the middle section of the 20106 working face and in the northwest of the study area. However, the II in Figure 5 areas in the WIRI were different from the WRSI, and the III in Figure 6 areas in the WRSI were different from the WIRI.

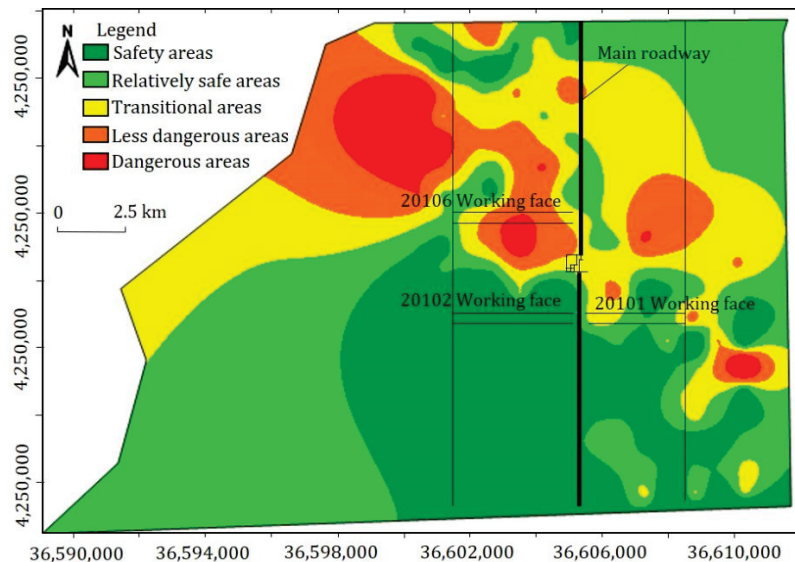


Figure 8. Water inrush assessment zoning map based on AHP.

In addition, the Zhangjiamao coal mine was selected as the validation area in this paper. By analyzing the geological data of the mining area, the control factors of water inrush risk index include specific capacity (Z1), hydraulic conductivity (Z2), aquifer thickness (Z3), core recovery (Z4), and ratio of brittle and tenacity terrane (Z5). The weights required for each control factor are shown in Table 3. Finally, the water inrush risk index model (Figure 9a), water-rich structure index model (Figure 9b), and comprehensive assessment model (Figure 9c) were established.

Table 3. The influencing factors weight of Zhangjiamao coal mine.

Weight	Z1	Z2	Z3	Z4	Z5
AHP (q_i)	0.375	0.125	0.250	0.125	0.125
EM (u_i)	0.299	0.298	0.141	0.010	0.252
AHP-EM (ω_i)	0.341	0.203	0.201	0.073	0.182

The dangerous area of water inrush from the weathered bedrock fissure aquifer in Zhangjiamao coal mine is shown in the red area in Figure 9c. The central area of the 15207 working face has a large amount of water inflow. The evaluation model is consistent with the actual mining, which shows that this method can be used for the coal mine hydrological assessment with similar geological conditions.

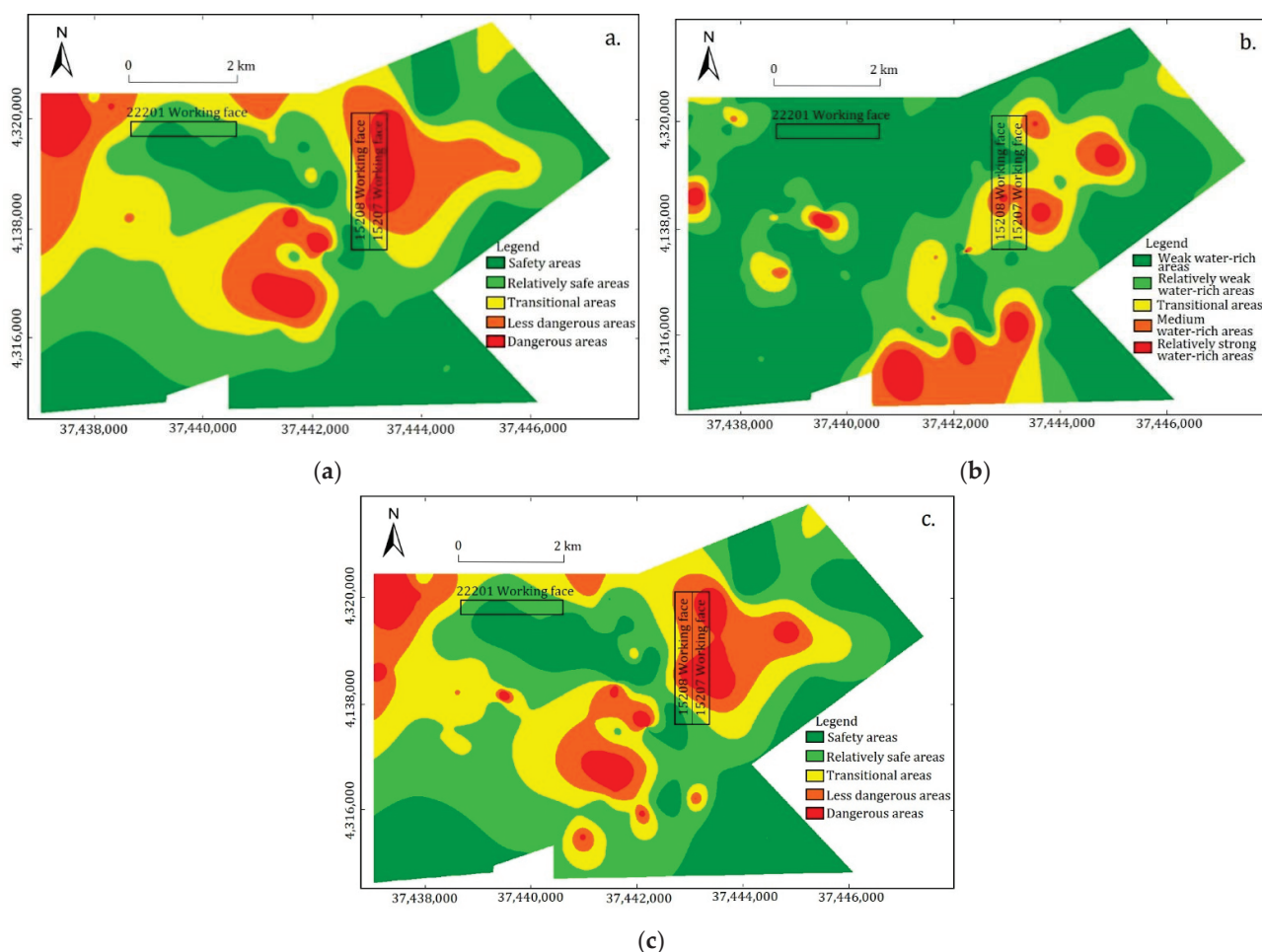


Figure 9. Water inrush assessment zoning map of the Zhangjiamao coal mine. (a) Water inrush risk index zoning map; (b) Water-rich structure index zoning map; (c) Comprehensive water inrush assessment zoning map.

4. Conclusions

In this study, a new method for comprehensive assessment of water inrush from coal seam roof was proposed, which is weighted by water inrush risk index and water rich index. The conclusions are summarized as follows:

1. The water inrush risk index was calculated by combining AHP with EM by the moment estimation method, and the water inrush risk index zoning map of the study area was compiled. In addition, the water content of coal seam roof was calculated by

the water rich structure index. Finally, the two indexes were weighted to established comprehensive assessment model.

2. On the basis of fully studying the geological and hydrogeological conditions of the coal mine, combined with the drilling data, the control factors of the water inrush risk indexes of Dahaize coal mine and Zhangjiamao coal mine were determined.
3. According to the analysis of the comprehensive water inrush assessment model, the high-risk areas of water inrush were clearly identified. The investigation of the actual mining situation in the mining area showed: the water inflow in the middle section of the 20106 working face of Dahaize coal mine was relatively large, and the water inflow in the central area of 15207 working face of Zhangjiamao coal mine was large.
4. The comprehensive assessment method is consistent with the actual mining status, which shows that this method can be used for the coal mine hydrogeological assessment with similar geological conditions and demonstrated its reliability and versatility. This method is of great significance for formulating corresponding measures to prevent water inrush in mining areas and ensuring safe production of coal mines.

Author Contributions: Z.S., W.B. and M.L. conceived and designed the experiment idea; W.B. analyzed the data and conducted the validation; Z.S. and W.B. focused on writing—original draft preparation; M.L. managed the project administration and supervision; Z.S. and M.L. did the writing—review and editing. All authors have read and agreed to the published version of the manuscript.

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Data Availability Statement: The data presented in this study are available on request from the corresponding author. The data are not publicly available due to this paper including a large number of mine geological exploration data.

Conflicts of Interest: The authors declare no conflict of interest.

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Article

Research on the Development Law of Pre-Mining Microseisms and Risk Assessment of Floor Water Inrush: A Case Study of the Wutongzhuang Coal Mine in China

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Abstract: Coal-mining areas are widely distributed in Northern China, but are under threat from confined water in the mining operation, resulting in a series of floor water-inrush hazards. Therefore, it is significant to effectively evaluate the floor water inrush to ensure safe and efficient coal mining. The 182602 working face of the Wutongzhuang coal mine served as the background for our research. The concept of “pre-mining microseisms” was proposed, and based on this, microseismic monitoring equipment was arranged on site. The correlation between microseismic events and the water abundance of an aquifer was analyzed, and a floor water inrush evaluation method was constructed based on the three elements of an aquifer and pre-mining microseisms. The main results are as follows: first, the microseismic events were excited by artificial disturbances before the mining of the working face including slurry diffusion and neighboring mining, which had the characteristics of sporadicity, clustering, and periodicity. Second, the regional distribution of water abundance was determined by taking the water inflow, water pressure, and grouting volume as the outward performance characteristics of water abundance in the Shanvuqing aquifer. Furthermore, the correlation coefficient between the pre-mining microseisms and the three elements of the aquifer (water inflow, water pressure, and grouting volume) was larger than 0.7. On this basis, an evaluation method associated with the water inrush risk along the strike of the working face was established based on pre-mining microseisms, dividing the working face into dangerous zones, suspected dangerous zones, and safe zones. Furthermore, pre-mining microseisms, water abundance, and structures were introduced as risk-evaluation indices, and the complete weight was calculated using an analytic hierarchy process and entropy-weight technique, before a vulnerability index model of floor water inrush was built. Finally, targeted treatment procedures were efficiently implemented to ensure the safe mining of working face 182602 due to the successful prediction of potential water risk zones. The research results provide scientific and technological support for pre-mining microseisms combined with water abundance as a technical method to prevent floor water inrush.

Keywords: pre-mining microseisms; floor water inrush; water abundance; forecast

1. Introduction

Coal is the main basic energy source in China and plays a leading role in the primary energy structure. China’s coal consumption in 2020 was 4.04 billion tons, and this is expected to reach as high as approximately 3.6 billion by 2030 [1,2]. At present, with the large-scale mining of coal resources, the current mining conditions have changed from easy to difficult, especially due to the prominent problem of floor water inrush. It is estimated that more than 55% of coal mines in China are limited by floor-confined water [3]. In recent years, most scholars have focused their attention on the study of the

mechanism and evaluation method of floor water inrush including “in situ fissures” and the “original destruction” theory [4], the “five figures plus double coefficients” method [5], the key layer “KS” theory of floor water inrush [6], the mechanism of delayed water inrush from the floor under the combined action of confined water [7], the Fisher’s discriminant model [8], and the height prediction of the water-flowing fracture zone [9]. In addition, many numerical simulation methods for the coupling of engineering-scale mining-water invasions have been obtained to reveal the progressive failure mechanisms of floor rock or coal pillars under mining-water immersion [10–12]. However, there are still deficiencies in the equipment for monitoring floor water inrush, and the development of the technology lags seriously behind the field’s urgent needs. Conventional monitoring technology has the disadvantages of poor monitoring timeliness, a small monitoring range, poor continuity, and a poor monitoring effect. As a result, an effective technology is urgently needed to compensate for the deficiencies of water inrush monitoring and early warning in coal mines [13].

A microseismic monitoring system was adopted to monitor the stability of engineering rock prior to deformation and damage. This technology was previously used primarily to monitor high-energy dynamic disasters in coal mines such as rock bursts and rock weighting [14]. Nearly half of the coal-mine disasters in China have been found to occur in clusters or be accompanied by earthquakes nearby, in which all the disaster types are involved [15]. As a matter of fact, the microseismic monitoring system could also detect the tiny vibration signals generated by rock failure during the formation of water-conducting channels in coal mines. It was also used to evaluate the risk of water inrush by introducing specific parameters such as the time and location of the water-conducting channel. Microseismic monitoring was first undertaken in a deep mine in China where water inrush was a significant issue for the progressive failure of the geological structures under investigation. With the results of the microseismic events and 3D illustration techniques, the activities of the geological structure and the fracture depth of the roof and floor were obtained [16]. Based on in situ experiments and numerical simulations, a new academic idea was proposed: “The rock microseismicity induced by mining processes and water pressure disturbance is in essence the index of groundwater inrush”. Groundwater inrush models have been calibrated and could be used to clarify the precursory characteristics and to locate the inrush pathway [17]. The microseismic monitoring technique was applied to the floor-mining fault, and a floor-fault water inrush prediction model was proposed based on the catastrophe analysis of microseismic signals [18]. Based on these academic results, the key layer of the roof was divided into two boundaries; taking the Dongjiahe coal mine in China as an example, the space–time and energy distribution of microseismic events were analyzed and this model of a coal mine roof was studied along the vertical direction [19]. This microseismic monitoring technique has been used successfully to evaluate the risk of water inrush in several mines such as the Tashan coal mine (Datong, China), Jiulishan coal mine (Jiaozuo, China), Fucun coal mine (Zaozhuang, China), and Halagu coal mine (Erdos China).

The findings of the aforementioned research have aided in the development and use of microseismic monitoring systems in areas such as groundwater inrush early warning. Nevertheless, previous research on microseismic characteristics and their engineering applications in coal mines has primarily focused on the distribution of microseismic events on the roof and floor during the mining process. Less is known about the characteristics of microseismic events before mining, which can be called ‘pre-mining microseisms.’ In this paper, taking the 182602 working face of the Wutongzhuang coal mine in China as an example, the authors statistically analyzed the distribution patterns of the floor pre-mining microseismic events. The hydrogeological characteristics on the working face caused by pre-mining microseismic could be determined, then the spatial location with high water abundance was judged. The microseismic activity information in the working face and surrounding area was interpreted in detail, and the characteristics of pre-mining microseismic were summarized. The pre-mining microseismic data were turned into valuable hydrogeological characteristic response data, which was normally deemed useless

in the grouting process. The research results further provide the basis and reference for microseismic data analysis and water inrush risk evaluation under similar conditions.

2. The Study Area and Its Engineering Background

2.1. Geological Conditions of 182602 Working Face

As shown in Figure 1, the Wutongzhuang coal mine, belonging to the mining area of Handan, is situated southeast of Handan City, Hebei Province, China, which is approximately 35.0 km from the center of Handan City. The 182602 working face with an average depth of 850 m, is located in coal seam 2 of the sixth mining area in the Wutongzhuang coal mine. The structure of the coal seam is simple, with an average thickness of 3.20 m, a hardness of 0.6, and an average dip angle of 14° . An inclined comprehensive long-wall stratified mechanized mining method was applied at the 182602 working face, that is, the width of the working face is 256 m, the length of the working face is 834 m, the average mining height is 3.5 m, and the average daily mining speed is 2.9 m/d. Normal faults are relatively developed inside the working face, and six faults and one syncline in total were found in the haulage gateway and air-return gateway. Through exploration, the fault does not conduct water, but the syncline conducts water. The layout of the 182602 working face is shown in Figure 2. In addition, the 182602 working face is adjacent to the 182101 working face with similar geological conditions.

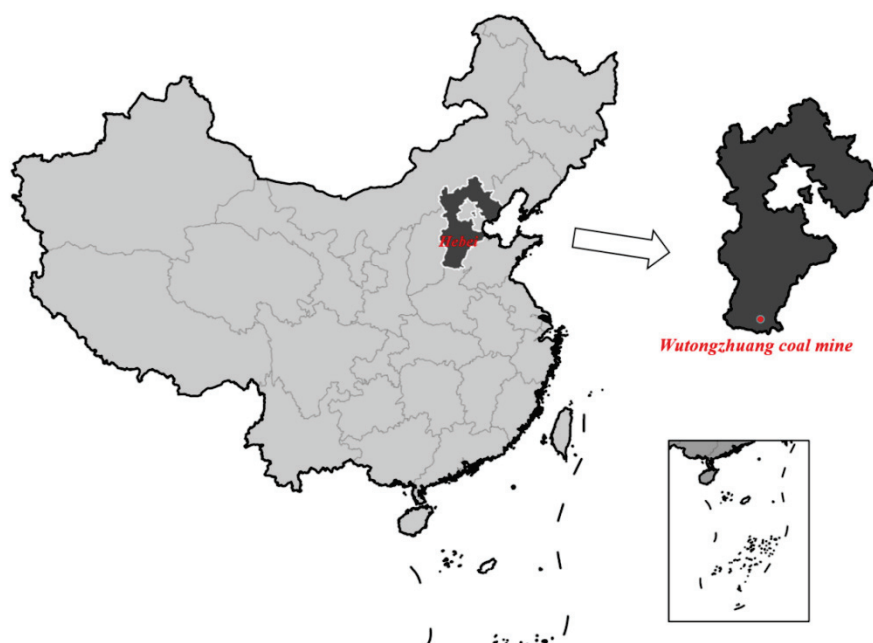


Figure 1. The location of the Wutongzhuang coal mine.

The immediate roof is sandy shale, with an average thickness of 3.21 m, whereas the main roof is fine-grained sandstone, with an average thickness of 6.13 m. The seam's immediate floor is sandy shale, with an average thickness of 3.54 m, whereas the seam's basic floor is fine sandstone, with an average thickness of 7.10 m. The lithological description of the coal seam roof and floor is listed in Figure 3.

The direct water-filled aquifer of the floor is the Yeqing limestone aquifer with an average thickness of 2.5 m and a hydraulic pressure of 0.35–7.2 MPa, which is approximately 38 m away from the coal seam. Another direct water-filled aquifer of the floor is the Shanvuqing limestone aquifer, with an average thickness of 5.5 m and a hydraulic pressure of 1.3–8.8 MPa, which is approximately 58.0 m away from the coal seam. The water bursting coefficient of the Shanvuqing aquifer is 0.07 MPa/m, which is greater than the critical value of 0.06 MPa/m. The Ordovician limestone aquifer has a wide distribution area, large thickness, and high water pressure, which is a major threat to mine safety production. The geological description of the aquifers is listed in Table 1.

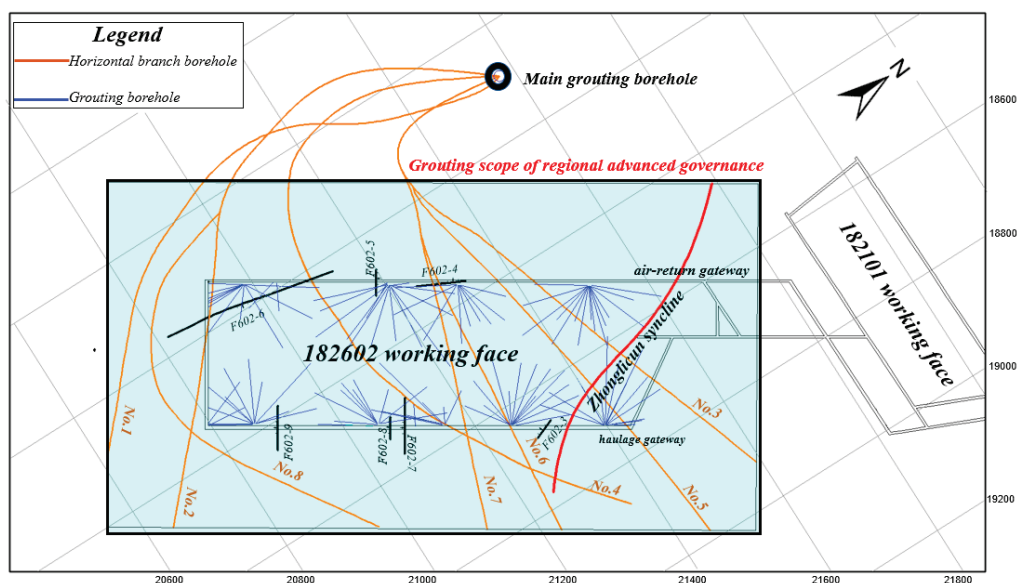


Figure 2. The layout of the 182602 working face in the Wutongzhuang coal mine.

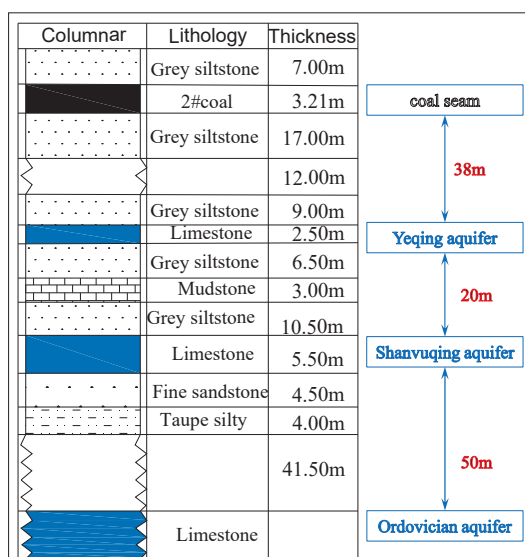


Figure 3. The lithological description of the coal seam roof and floor.

Table 1. The geological description of the aquifers.

Aquifer	Average Thickness (m)	Distance from Coal Seam(m)	Water Pressure (MPa)	Permeability Coefficient (m/d)	Specific Capacity (L/s·m)	Water Quality Type	Mineralization (g/L)
Yeqing	2.5	38	0.35~7.2	0.04	0.02	Cl·SO ₄ -Na. Ca	4.3
Shanvuqing	5.5	58	1.3~8.8	2.35	0.078	Cl·SO ₄ -Na. Ca	5.5
Ordovician	-	108	5.3~9.6	5.89	1.258	Cl·SO ₄ -Ca. Na	>5.24

2.2. Overview of the Regional Advanced Governance Technology on the 182602 Working Face

In Chinese mines, two different types of water prevention and control countermeasures are typically employed. One is the method of hydrophobic depressurization, which helps to lessen the risk of floor water inrush. However, when the water abundance of the aquifer is strong and the recharge is large, it will run into issues with reducing the hydraulic head difficulty, the high water pressure or drainage, and high cost. Therefore, few mines are currently using this method. Another method is the regional advanced governance

technology, which can increase the thickness of the aquifuge, and enhance the water insulation and rock strength of the floor rock mass [20].

The regional advanced governance technology was used to inject slurry into the grouting hole, which can be solidified according to the designed concentration. It would press and convey the slurry to the prescriptive rock and soil, and then fill in the soil cracks or pores, forming a similar curtain of the concrete cut-off wall [21,22].

The Wutongzhuang coal mine is a typical coalfield with a high risk of water leakage in Northeast China, which has adapted the regional advanced governance technology to pre-dispose the water leakage in the Shanvuqing and Ordovician limestone. As shown in Figure 2, the key to achieving regional advanced governance technology is ground multi-branch horizontal directional grouting and floor grouting reinforcement.

The ground multi-branch horizontal directional grouting project was carried out on the 182602 working face. One main grouting borehole and eight horizontal branch grouting boreholes were constructed on the ground face, and the grouting horizon of the horizontal branch boreholes was the Shanvuqing aquifer. The information on the ground grouting boreholes is shown in Table 2. Furthermore, in the floor grouting reinforcement project, a total of 174 boreholes were constructed to expose the Shanvuqing limestone aquifer, and the cumulative grouting volume was 998.77 tons.

Table 2. The information on ground horizontal branch boreholes.

Borehole Number	Horizontal Length (m)	Grouting Time	Grouting Volume (t)	Circulation Loss (t)
No. 1	739	December 2017	159	-
No. 2	811	January 2018	295	-
No. 3	760	March 2018	16	-
No. 4	652.5	June 2018	376	40
No. 5	866	July 2018	437	28
No. 6	717	August 2018	82	13
No. 7	700	September 2018	124	-
No. 8	764	September 2018	52	-

2.3. Overview of the Microseismic Monitoring System on the 182602 Working Face

The KJ1073 microseismic monitoring system, which was created independently by the Key Laboratory of Mining Microseismic in Hebei Province, China, may be utilized for daily monitoring on the 182602 working face. The system is made up of both ground and underground components. The ground equipment consists mostly of the ‘data processing computer’, ‘mainframe for monitoring’, and underground equipment.

To create an ‘encircled’ geophone array structure and a high-precision microseismic monitoring system, we attempted to install geophones at various locations and optimizing the placement algorithm, ultimately determining the following based on the installation and debugging results. As shown in Figure 4, five single-axis geophones and one triaxial geophone were placed on the haulage gateway of the 182602 working face, and six single-axis geophones and one triaxial geophone were installed in the air-return gateway, with a distance of 130 m between each geophone.

The 182602 working face was mined on 7 November 2018. The microseismic monitoring system was set up and monitored on 30 May 2018. The total time of the pre-mining microseismic monitor was five months. Microseismic events occurred between the No. 2 coal seam floor and the Ordovician limestone karst aquifer before the mining of the 182602 working face, which were characterized by “large frequency, large scope, many aggregation zones, and dispersion” due to the regional advanced governance project and the mining disturbance of the adjacent 182101 working face. These are referred to as “pre-mining microseisms” as a whole.

The pre-mining microseismic monitoring process was divided into two periods based on the stoppage time of the adjacent 182101 working face (29 July 2018). The first period

was from June to July 2018. The second period involved August, September, and October 2018.

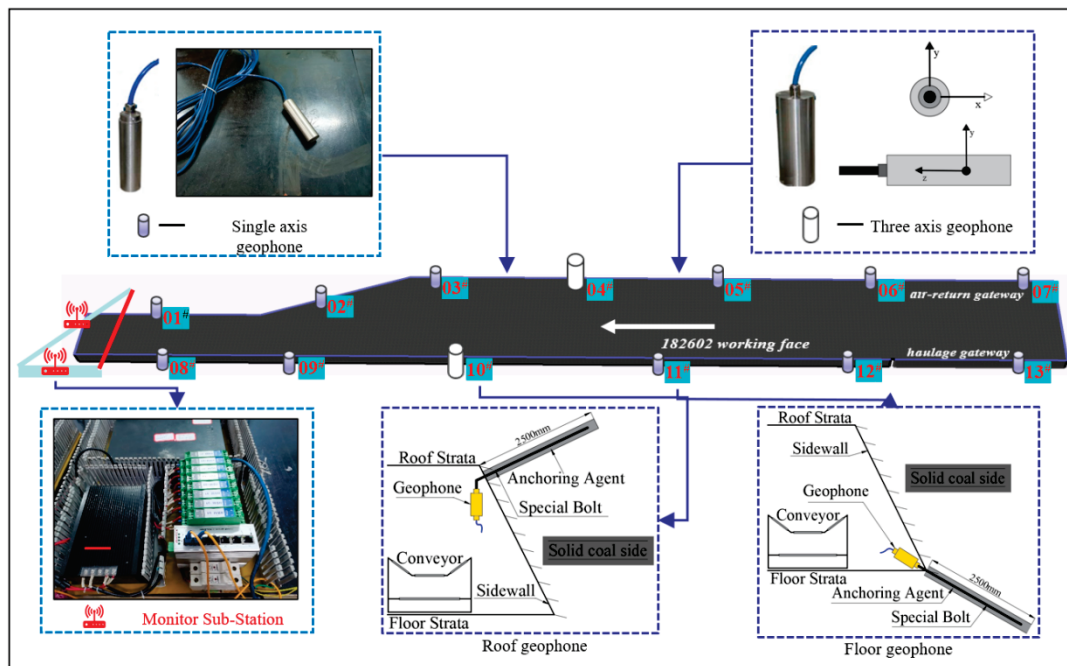


Figure 4. The microseismic monitoring system arrangement on the 182602 working face.

3. Dynamic Development Pattern and Distribution Characteristics of Pre-Mining Microseisms

3.1. Plane Distribution Characteristic of Microseismic Events within the Range of Shanvuqing Aquifer

The kernel density estimation was used to analyze the plane distribution characteristic of the microseismic events. The kernel density cloud chart of microseismic events within the range of the Shanvuqing aquifer during the first period is shown in Figure 5, and the kernel density cloud chart of the microseismic events during the second period is shown in Figure 6. Different degrees of blue in the figures represent the different density values of the microseismic events. Referring to the results of other studies and the empirical analysis, the zone with a frequency kernel density greater than $0.00006/\text{m}^2$ was the fracture developing zone or potential zone of the water-conducting channel under the action of stress disturbance.

During the first period, the kernel density of microseismic events was distributed in a V-shaped continuous two-center type with the Zhonglicun syncline as its axis. The high-frequency kernel density center 'A' was located in the zone near the main grouting borehole, and the other kernel density center 'B' was located in the zone near the 182101 working face. The maximum kernel density value during June and July was $0.000339/\text{m}^2$, and the kernel density zone extended in a belt from the center to the 182602 working face. The maximum kernel density value within the 182602 working face was $0.00024/\text{m}^2$, indicating that the fractures within the working face had not been effectively blocked, especially in the zone near the Zhonglicun syncline.

During the second period, the kernel density distribution of microseismic events showed a phase change. The kernel density zone during August 2018 showed a bicentric discontinuous distribution in a northwest–southeast direction. When compared to August 2018, the kernel density zone during September 2018 showed a one-center discontinuous distribution in the northwest–southeast direction, with a 19.2% increase in the center 'A' kernel density value and a 41.3% decrease in the center 'B' kernel density value. When compared to September 2018, the kernel density area during October 2018 showed a non-center discontinuous distribution in the northwest–southeast direction, with a 45.1%

decrease in the center 'A' kernel density value and a 41% decrease in the center 'B' kernel density value. There was no microseismic event response within the 182602 working face during the second period, indicating that the fracture within the 182602 working face had been effectively sealed. The kernel density value of center 'A' increased or decreased due to the influence of the grouting time and grouting volume of the horizontal branch grouting boreholes, and the kernel density value of center 'B' gradually decreased due to the stoppage of the adjacent 182101 working face.

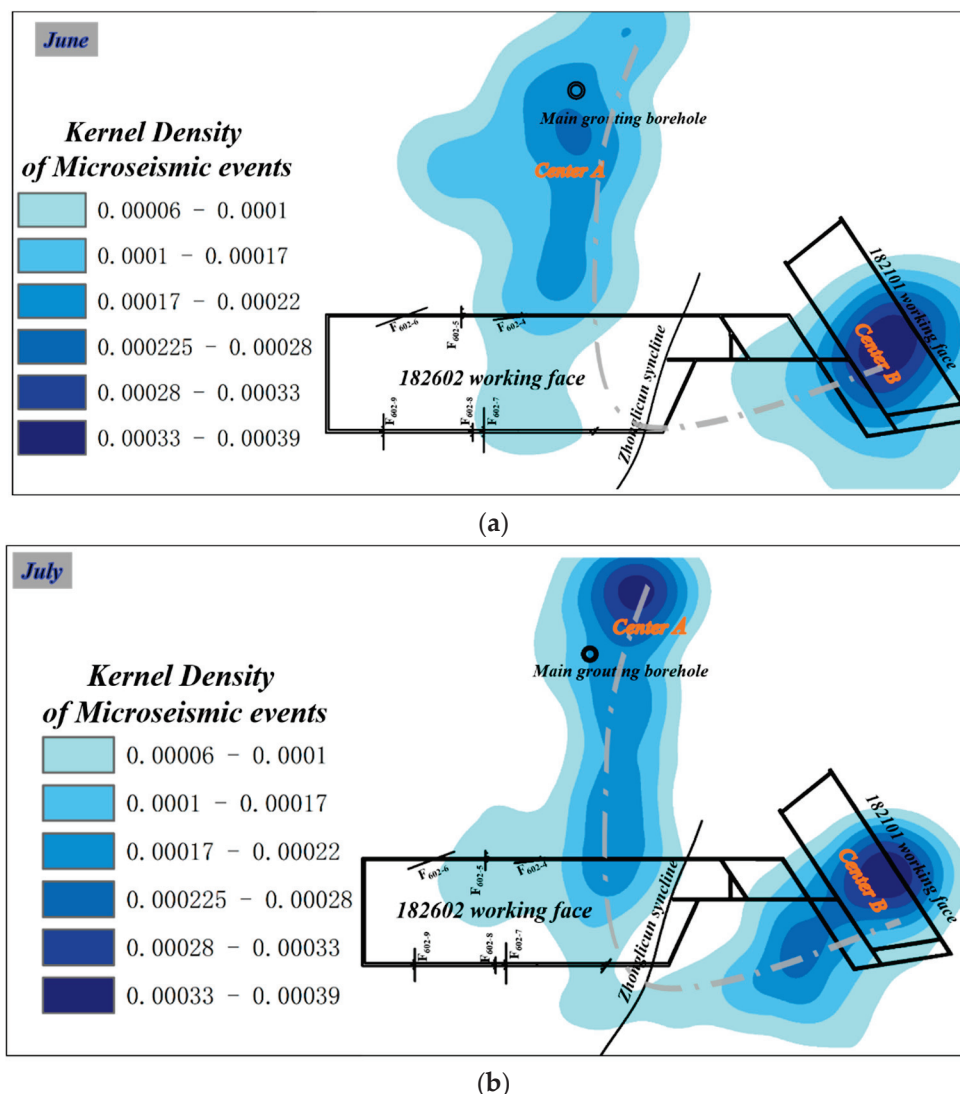


Figure 5. Kernel density distribution characteristic of microseismic events in the first period. (a) Kernel density value during June. (b) Kernel density value during July.

The aggregation of microseismic events in center 'A' reflected the expansion process of fractures caused by the high-pressure slurry, and the aggregation of microseismic events in center 'B' reflected the expansion process of fractures caused by mining stress disturbance in the adjacent 182101 working face. During the monitoring process, the kernel density value of center 'B' showed a decreasing trend. It was determined that the decrease in the kernel density values of the microseismic event was related to the progressive decrease in the residual mining stress following the stoppage of the 182101 working face, resulting in the gradual closure of deep water-conducting fractures. During the monitoring process, the kernel density value of center 'A' increased first and then decreased. The high-pressure slurry was determined by expanding the hydraulic fractures and cavities, closing them, releasing significant strain energy, and causing microseismic events. However, the fractures

were filled and strengthened with the progress of grouting, some vertical and horizontal water-conducting channels were blocked, and the groundwater flow field was rebalanced, leading to a greater decrease in the nuclear density of microseismic events, especially in October 2018.

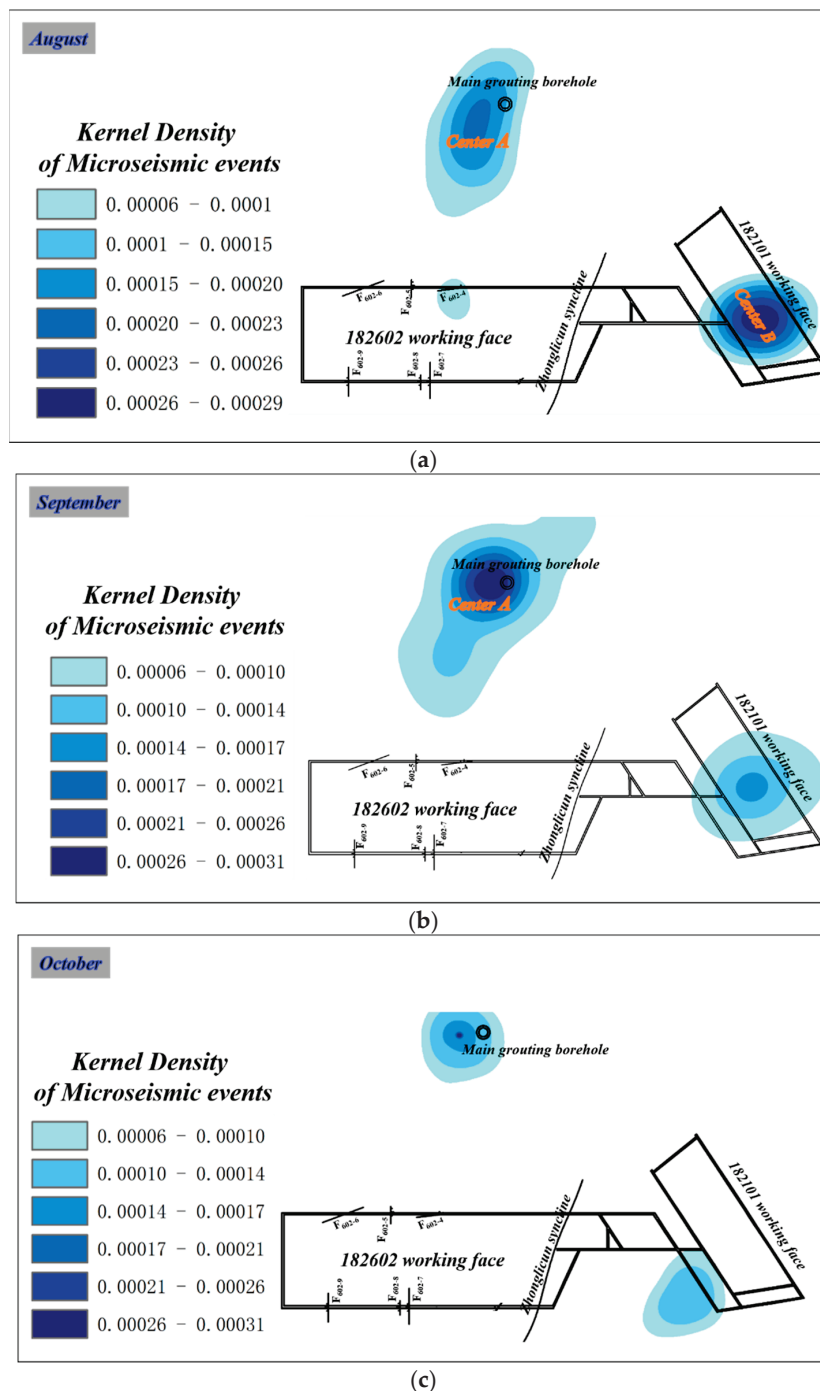


Figure 6. Kernel density distribution characteristic of microseismic events in the second period. (a) Kernel density value during August. (b) Kernel density value during September. (c) Kernel density value during October.

Therefore, comparing the changes in the distribution of the microseismic kernel density values between the first and second periods, it can be seen that pre-mining microseisms would be excited only after the equilibrium of the rock–fluid system was disrupted under artificially disturbed conditions (high-pressure slurry diffusion, neighboring mining stress)

in the working face's pre-mining phase. When the influence of external disturbances weakens, the aggregation degree of microseismic events would weaken until it disappears.

3.2. Spatial Distribution Characteristic of Microseismic Events within the Range of Shanvuqing Aquifer

The microseismic events within the Shanvuqing aquifer were spatially located and the spatial distribution chart of the microseismic events was drawn. The red sphere represented microseismic events and the gray surface represented a coal seam. In addition, the XZ-axis and YZ-axis directions of the space were projected, respectively. Taking June 2018 as an example, the spatial distribution of microseismic events in the Shanvuqing aquifer is shown in Figure 7.

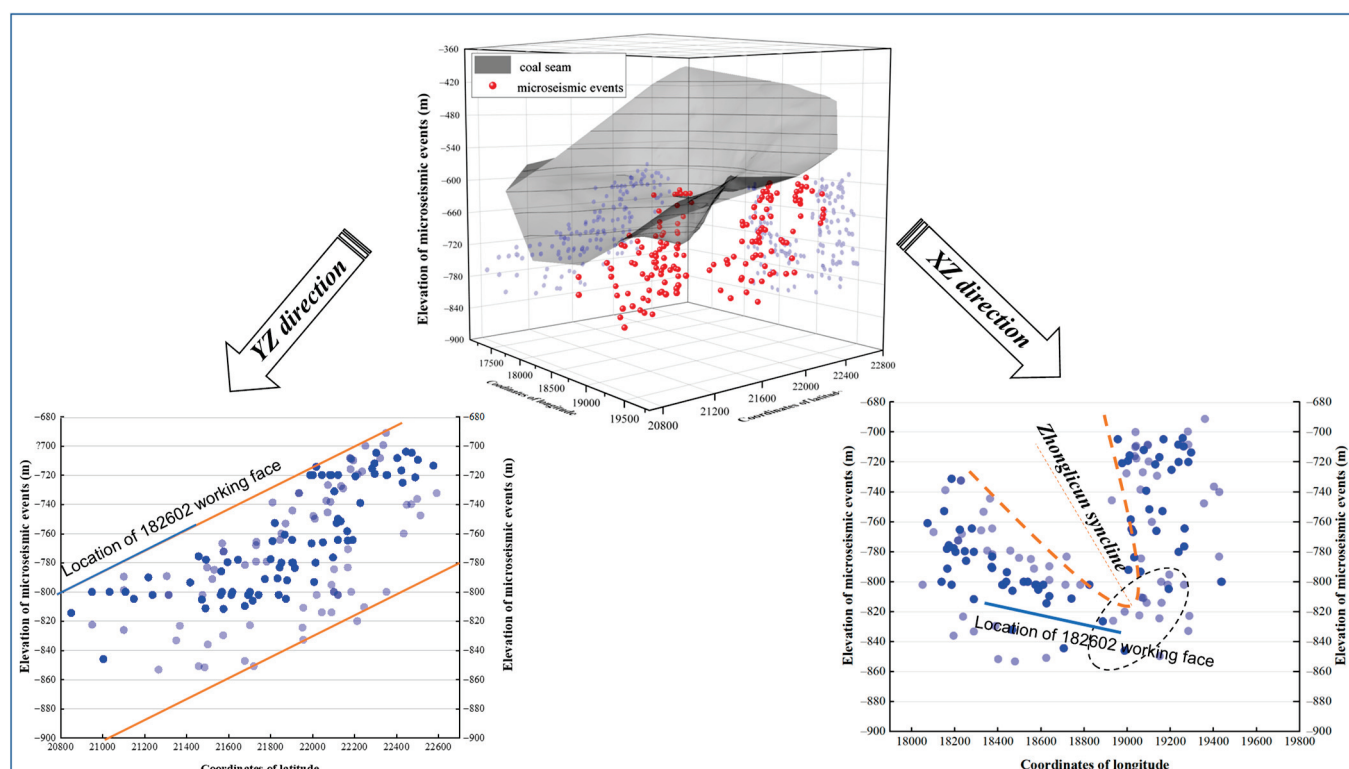


Figure 7. The spatial distribution of microseismic events during June 2018.

The coal seam elevation in the monitoring area ranged from -950 m to -360 m, and there were many microseismic events within the Shanvuqing aquifer and spatially distributed continuously. The microseismic events that were projected took the Zhonglicun syncline as the axis in the XZ-axis direction, showing a “V” continuous distribution. There were massive microseismic events at the endpoint. These were distributed in a belt along the two wings of the Zhonglicun syncline, with a left wing width of 338.8 m and a right wing width of 348.3 m. The microseismic event projection was distributed in an inclined strip along the axial plane of the Zhonglicun syncline in the YZ-axis direction, with a width of 313.2 m.

The spatial distribution characteristics of the pre-mining microseisms during July, August, September, and October 2018 were similar. The microseismic event projection was distributed in an inclined strip along the axial plane of the Zhonglicun syncline in the YZ-axis direction. The spatial distribution of microseismic events showed that the axial part and the two wings of the Zhonglicun syncline were more fractured, and more strain energy had accumulated inside due to the influence of high-level tectonic stress. Under the action of tectonic stress, high ground stress, and high water pressure, strain energy was released, which made fractures and cavities with water storage space develop and expand,

leading to the destruction of the groundwater flow field equilibrium, thus causing rock damage and exciting microseismic events.

As a result, the higher the possibility of the existence of a potential water-conducting channel in zones with a high degree of fissure development (the two wings of Zhonglicun syncline), the rock system's equilibrium was more likely to be disrupted, and the pre-mining microseismic events often aggregated in these types of zones.

3.3. Energy Evolution Characteristics of Microseismic Events within the Range of Shanvuqing Aquifer

According to the previous dynamic phenomena, when the energy release of a single microseismic event reaches more than 10^4 J, the rock mass may be macroscopically damaged [23]. A large microseism was defined as when the energy release of a single microseismic event was greater than 10^4 J, and a destructive micro-seismic as when the energy release of a single microseismic event was greater than 10^5 J, based on the pertinent literature and the microseismic energy release in the Wutongzhuang coal mine. The destructive microseisms reflected the instantaneous rupture of the coal and rock mass, accompanied by the instantaneous release of accumulated elastic energy.

The construction of the No. 4 grouting borehole was on 9 June 2018, and the construction of the No. 5 grouting borehole started on 26 July 2018. To analyze the effect of grouting on the excitation of microseismic events, we counted the representative microseismic monitoring data from 10 June to 20 August 2018 before mining and quantified the energy of the microseismic events. A total of 900 microseismic events were collected. As shown in Figure 8, the grouting impact period was divided into three phases regarding the energy variation characteristics of the microseismic events.

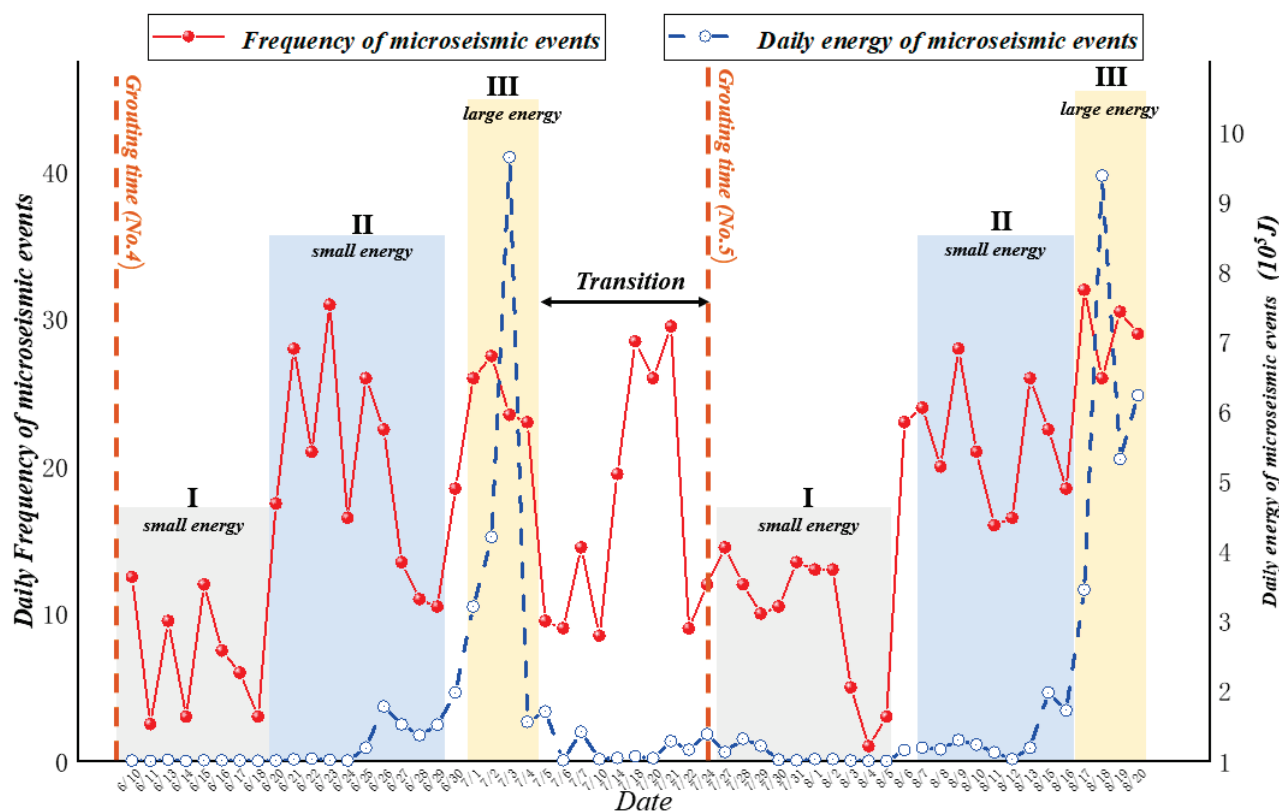


Figure 8. The energy evolution characteristics of the microseismic events.

The first phase lasted nine days, from 10 June to 18 June, and the microseismic events were characterized by ‘low frequency and small energy’. The daily frequency of microseismic events was less than 15, the daily energy was less than 10^5 J, and the energy of a single microseismic event was less than 10^3 J. The second phase lasted ten days, from

20 June to 29 June, and the microseismic events were characterized by ‘high frequency and small energy’. The daily frequency of microseismic events was not less than 15, but the daily energy was not more than 5×10^5 J, and 95.8% of the energy of microseismic events was less than 10^3 J. The third phase lasted four days, from 1 July to 4 July, and the microseismic events were characterized by ‘high frequency and large energy’. The duration of this phase was short. The daily frequency of microseismic events was generally greater than 20, and the daily energy exceeded 5×10^5 J. Then, from 5 July to 25 July was the transition, which usually lasted for a long time, and there were no grouting boreholes during this period. The daily frequency of microseismic events varied greatly, but the daily energy was small, with the maximum not exceeding 5×10^5 J. The energy of 87.4% of microseismic events was less than 10^3 J. Until the construction of the No. 5 grouting borehole (26 July 2018), the distribution of microseismic events repeated the above phases in turn, and the frequency and energy changed periodically.

The characteristics of the pre-mining microseisms varied with the progression of grouting, which was obviously different from the large-scale microseisms induced by periodic weighting after mining of the working face. The pre-mining microseisms showed a phased change, and the inducement was mainly a small-scale microfracture. In the first phase, the slurry was naturally filled into large fractures or rock defects, the grouting time was long, there was no need for grouting pump pressure, and the stress concentration caused by filling grouting was small, resulting in the microseismic events characterized by ‘low frequency and small energy’. In the second phase, the grouting time was long, the high-pressure slurry diffused radially, destabilizing the groundwater flow field, and the penetrative grouting induced stress concentration and strain energy release, resulting in microseismic events characterized by ‘high frequency and small energy’. In the third phase, the grouting time was short, the high-pressure slurry caused further expansion of the fracture opening, fractures of the grouting injured the rock mass, and a substantial amount of strain energy was released, resulting in the microseismic events characterized by ‘high frequency and large energy’. When the energy was concentrated and released, it reentered a new round of energy accumulation and release process. The microseismic events were excited again until the next grouting, and their characteristics changed in phase.

3.4. Dynamic Development and Distribution Characteristics of Pre-Mining Microseisms

Combined with the analysis of plane distribution, spatial distribution, energy distribution, and the disturbance factors of microseismic events before mining, we found the following three interesting performance characteristics of the pre-mining microseisms.

One of the characteristics of pre-mining microseisms was sporadicity. There were periods of high microseismic activity followed by periods of relative calm. The sporadicity of microseismic activity reflected the regional stress or local rock mass stress anomaly. For example, the frequent microseismic events at center A in June and July 2018 were closely related to the branch hole grouting activity at this location. However, the substantial decrease in microseismic events at center A during October 2018 was associated with the decrease in the strain energy release triggered by slurry diffusion near the end of grouting. Therefore, the sporadic analysis of pre-mining microseisms can be used for real-time monitoring and evaluation of the grouting engineering effect, and guide the next grouting work.

One of the characteristics of pre-mining microseisms is clustering. The dense zone of microseismic activity always formed in the area with relatively developed fractures, relatively broken rock mass, strong water abundance, and a large slurry consumption. The clustering of pre-mining microseisms could be used to reflect the zones where fractures are relatively developed and a water-conducting channel is easily formed.

One of the characteristics of pre-mining microseisms was periodicity. The energy of pre-mining microseisms was in a phased change from ‘low frequency and small energy’ to ‘high frequency and small energy’ to ‘high frequency and large energy’, so as to complete

the release of energy for rock failure. This change process corresponded exactly to the multiple phases of the grouting process (filling grouting, penetrative grouting, fracturing grouting). The periodicity of the pre-mining microseisms applied to the monitoring of a single branch hole grouting and regional grouting process, and the evaluation of the grouting effect.

4. Correlation Analysis between Characteristic of Pre-Mining Microseismic and Water Abundance of Floor Aquifer

The Shanvuqing aquifer is a direct water-filled aquifer of coal seam 2 on the 182602 working face. The water abundance of the Shanvuqing aquifer is strong and heterogeneous. It is laterally recharged by Ordovician limestone water, which might still pose a threat of water inrush to coal seam mining after grouting reinforcement. The accuracy of the exploration results of the aquifer by geophysical methods before mining on the working face remains to be discussed [24]. Therefore, it is of great significance to understand the correlation between the characteristics of pre-mining microseisms and the water abundance of the Shanvuqing aquifer.

4.1. Analysis of Water Abundance of the Shanvuqing Aquifer

The average total thickness of the Shanvuqing aquifer is 5.5 m, with local water abundance. According to the actual exposure of the boreholes within the 182602 working face, the average distance from the aquifer to the bottom of the working face is 58 m, the maximum water inflow from a single hole is 80 m³/h, the highest water level of the aquifer is +113.5 m, and the water temperature is 36.5 °C~45.2 °C. The 182602 working face was separated into the inner and outer sections based on the distribution of the water abundance of the aquifer. The basic conditions of the Shanvuqing aquifer within the inner and outer sections of the 182602 working face are shown in Table 3.

Table 3. The basic conditions of the Shanvuqing aquifer within the inner and outer sections.

Zone	Location in Relation to the Zhonglicun Syncline	Number of Boreholes	Water Inflow (m ³ /h)	Water Pressure (MPa)	Water Temperature (°C)	Water Level (m)
Inner section	Away from the side	53	0.1~80	1.3~8.1	36.5~44.1	−606.5~113.5
Outer section	Near the side	121	0.2~80	1.7~8.9	38.2~46.5	−568.5~102.5

We commonly assumed that the water inflow, water pressure, and grouting volume were the outward performance characteristics of the water abundance of the Shanvuqing aquifer [25].

(1) The water inflow of boreholes

The fracture development degree of the aquifer is closely related to its water storage capacity, and also determines the recharge and discharge conditions of the aquifer. Since the fracture development degree of the aquifer is difficult to measure by direct means, the water inflow of the boreholes can reflect the fracture development degree and water abundance of the Shanvuqing aquifer.

According to engineering practice, a water inflow higher than 10 m³/h is called high water inflow. As shown in Figure 9c, there were 12 boreholes with a water inflow not less than 10 m³/h within the inner section, accounting for 25% of the total boreholes, and the maximum water inflow reached 50 m³/h. There were 55 boreholes with a water inflow not less than 10 m³/h in the outer section, accounting for 46.2% of the total boreholes, and the water inflow of several boreholes reached 60 m³/h, with the maximum water inflow reaching 80 m³/h. Therefore, the boreholes with higher water inflow in the outer section were significantly more than those in the inner section.

(2) The water pressure of boreholes

The water pressure value of the aquifer is one of the main elements reflecting the water abundance. When the water pressure value is higher, it indicates that the water abundance of the aquifer is stronger.

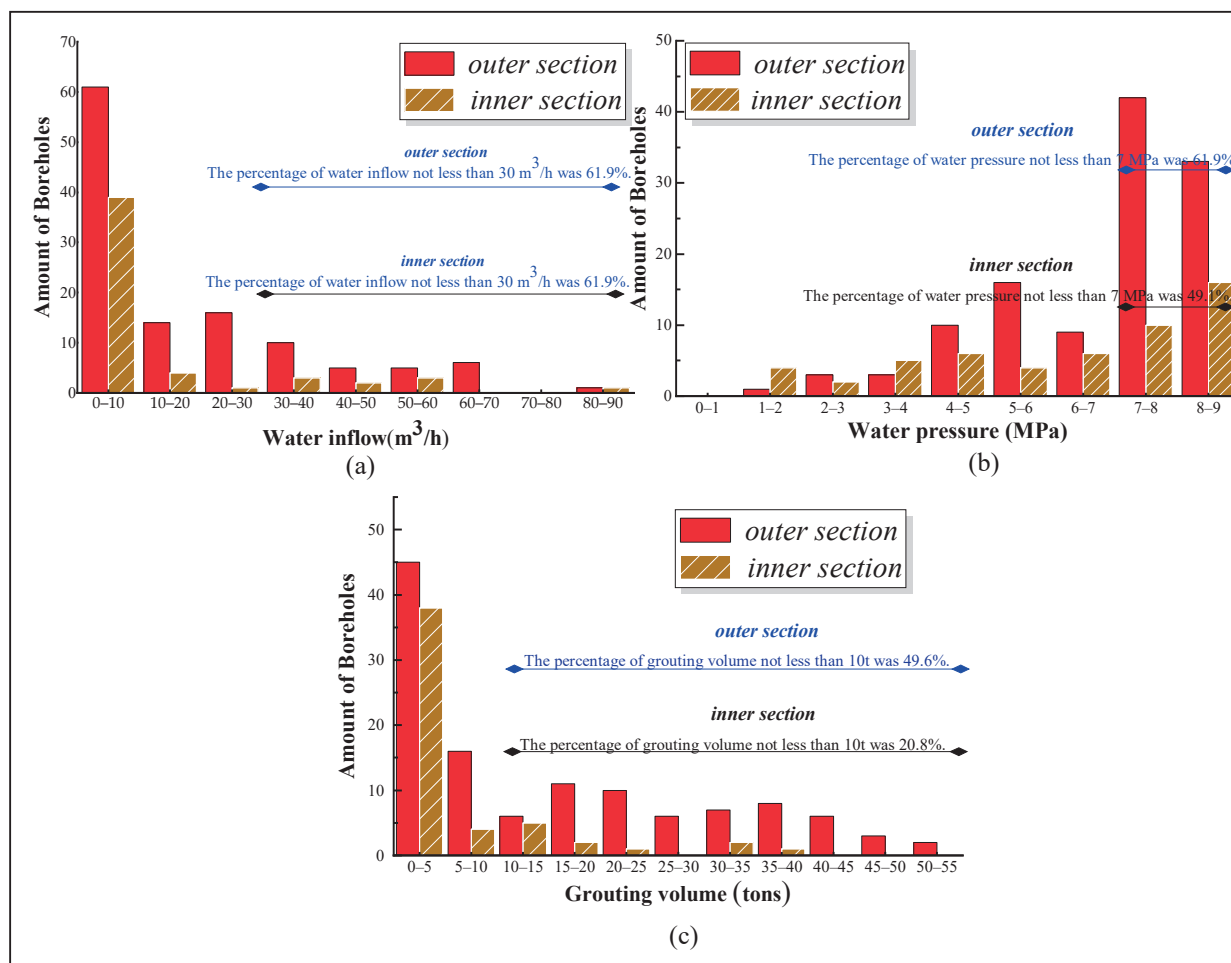


Figure 9. Variation in the water inflow, water pressure, and grouting volume in the inner and outer section.

According to engineering practice, a water pressure higher than 7 MPa is called high water pressure. As shown in Figure 9b, there were 25 boreholes with a water pressure not less than 7 MPa within the inner section, accounting for 49.1% of the total boreholes, and the maximum water pressure reached 8.1 MPa. There were 75 boreholes with a water pressure of not less than 7 MPa in the outer section, accounting for 61.9% of the total boreholes, and the maximum water pressure reached 8.9 MPa. Therefore, the boreholes with higher water pressure in the outer section numbered significantly more than those in the inner section.

(3) The grouting volume of boreholes

The grouting volume of boreholes can reflect the degree of fracture development of the aquifer to a certain extent. When the grouting volume is greater, the fractures are developed and the connectivity is good.

According to engineering practice, a grouting volume higher than 10 tons is called a high grouting volume. As shown in Figure 9c, there were 10 boreholes with grouting volume not less than 10 tons within the inner section, accounting for 20.8% of the total boreholes, and the grouting volume reached 23.52 tons. There were 60 boreholes with a grouting volume not less than 10 tons in the outer section, accounting for 49.6% of the total boreholes, and the maximum grouting volume reached 56.7 tons. Therefore, the boreholes

with a higher grouting volume in the outer section numbered significantly more than those in the inner section.

Therefore, the boreholes with a high water inflow, high water pressure, and high grouting volume (hereinafter referred to as ‘3-high’ boreholes) in the outer section numbered significantly more than those in the inner section. Therefore, the water abundance of the outer section was significantly higher than that of the inner section, showing a gradually decreasing trend from the east to the west of the working face, from the syncline axis to the open-cut of the working face.

4.2. Correlation Analysis between Microseismic Events and ‘3-High’ Boreholes

The number of ‘3-high’ boreholes could indicate the water abundance of the Shanvuging aquifer in different zones. We investigated the relationship between the water abundance and microseismic events along the strike of the working face. As shown in Figure 10, the variation in the water inflow, water pressure, water temperature, and grouting volume was basically positively correlated, and the trend of microseismic events along the strike of the working face was consistent with that of the ‘3-high’ boreholes. Microseismic events and ‘3-high’ boreholes increased gradually along the strike of the working face. Microseismic events and ‘3-high’ boreholes in the outer section numbered significantly more than those in the inner section, and increased significantly in the fracture zone around the Zhonglicun syncline.

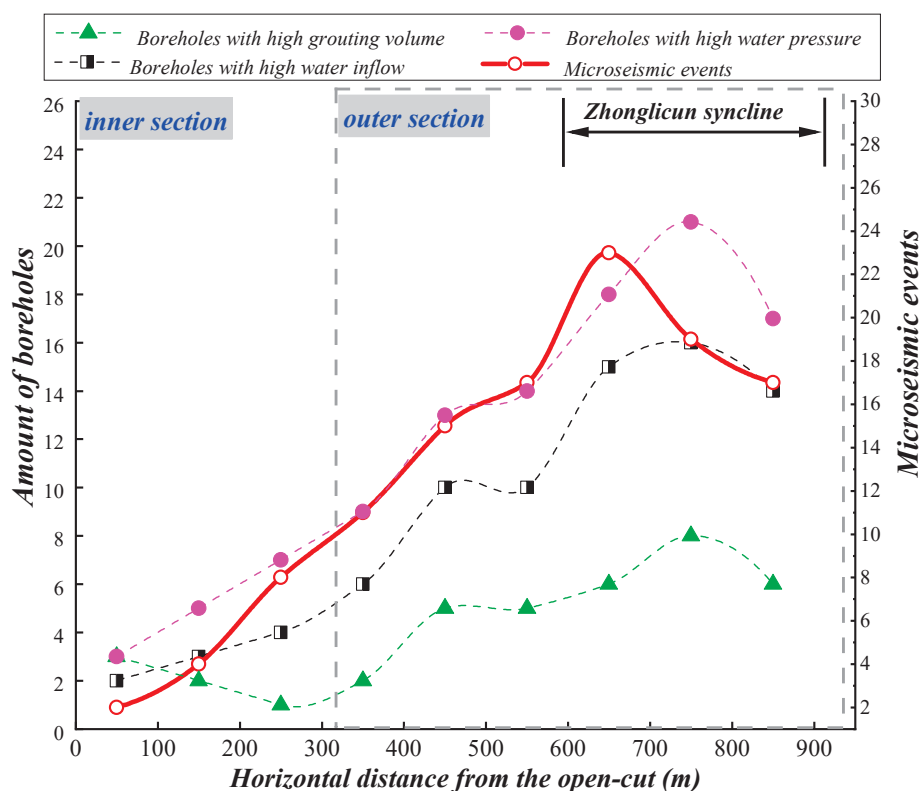


Figure 10. The variation trend of ‘3-high’ boreholes and microseismic events along the strike.

The correlation coefficient in statistics can be used to reflect the closeness of the correlation between variables.

$$\gamma(X, Y) = \frac{Cov(X, Y)}{\sqrt{Var[X] Var[Y]}}$$

where $Cov(X, Y)$ is the covariance of X and Y ; $Var[X]$ is the variance of X ; $Var[Y]$ is the variance of Y .

The correlation coefficients γ_1 , γ_2 , and γ_3 were calculated for the microseismic events and the boreholes with high water inflow, high water pressure, and high grouting volume, respectively, and the results can be obtained as $\gamma_1 = 0.86$, $\gamma_2 = 0.82$, and $\gamma_3 = 0.71$. Therefore, microseismic events were highly correlated with the distribution of '3-high' boreholes, that is, pre-mining microseisms were highly correlated with the water abundance of the Shanvuqing aquifer.

In conclusion, microseismic events within the Shanvuqing aquifer gradually decreased from the direction of the Zhonglicun syncline axis to the open-cut of the working face, which was essentially consistent with the distribution of the water abundance of the Shanvuqing aquifer. Microseismic events were more concentrated in zones with high water inflow, high water pressure, and high grouting volume, and microseismic events were significantly reduced in zones with weak water abundance. Therefore, pre-mining microseisms can identify the spatial water abundance differences of the main water-filled aquifers and characterize the water inrush risk zones.

4.3. Risk Assessment Method of Water Source along the Strike of the Working Face

Under the assumption that all other conditions were essentially the same, based on the elements of cumulative water inflow, average water pressure, and cumulative grouting volume of the grouting boreholes, a risk grading criteria of water source is presented as it was feasible to measure the risk level of the floor water inrush by judging the water source and its water abundance.

The risk of the water source was graded regarding the following criteria. The zone with a high water inflow, high water pressure, and high grouting volume was the dangerous zone. When only one or two of the three elements was high, the zone was suspected as being dangerous. Since pre-mining microseisms were highly correlated with the water abundance of the aquifer, the water inrush risk should be subsequently verified by the distribution of microseismic events. Furthermore, zones with low water inflow, low water pressure, and low grouting volume were the safe zones.

The information of 174 grouting boreholes was counted and cumulative water inflow, average water pressure, and cumulative grouting volume were examined along the strike, as shown in Figure 11. According to the risk grading criteria of the water source, there was no dangerous zone in the inner section, and there were two danger zones in the outer section, which were located in the fault F602-4 and Zhonglicun syncline, respectively. The zone at a distance of 0–50 m from the open-cut was expressed in the form of high cumulative water inflow, low average water pressure, and high cumulative grouting volume. In addition, there was only one element with a large value at a distance of 250–300 m, 350–400 m, 450–500 m, and 750–800 m from the open-cut, which were suspected as being dangerous zones. The rest were normal zones.

The microseismic distribution is shown in Figure 12. A total of 75.3% of the microseismic events were clustered within the dangerous zones of the outer section; 5.2% of the microseismic events were sporadically distributed in the suspected dangerous zones of the inner section, indicating that the risk of floor water inrush was at a low level; and 13.3% of the microseismic events were distributed in the suspected dangerous zones of the outer section, indicating that the risk of floor water inrush was at a high level. Therefore, when mining the outer section of the working face, exploration and water control measures should be conducted in advance.

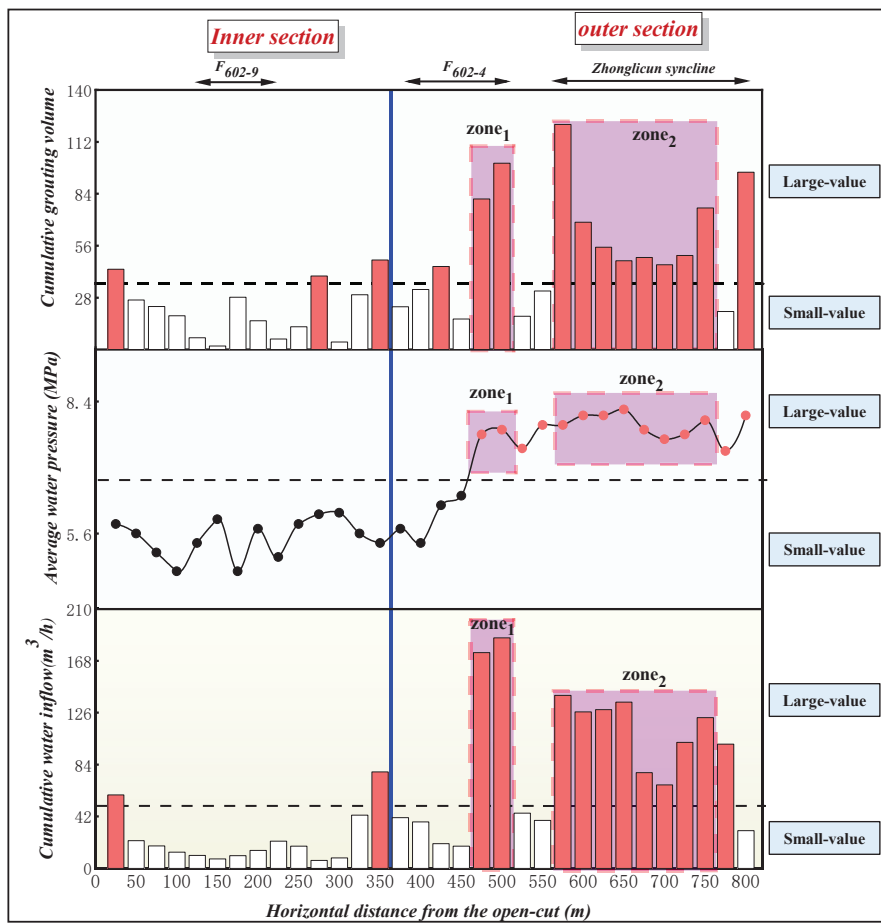


Figure 11. The variation trend of the three elements along the strike of the working face.

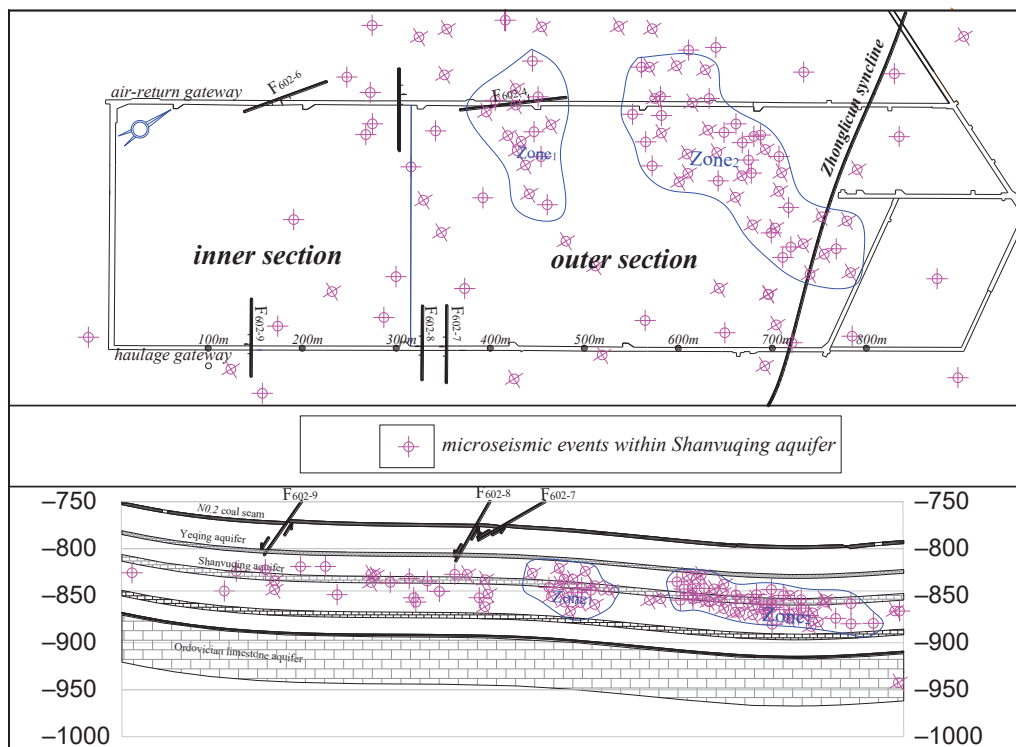


Figure 12. The distribution of microseismic events within Shanvuqing aquifer.

5. The Vulnerability Index Model of Floor Water Inrush Based on Level Analysis and Entropy Weight Method

The vulnerability index approach [26] is a new practical method for the risk evaluation of floor water inrush that takes into account numerous parameters (including their respective weights) and can reflect the nonlinear dynamic phenomenon of floor water inrush under the influence of multiple factors. We built the vulnerability index model for the risk of floor water inrush using the microseismic monitoring results and the water abundance of the aquifer as research indicators. The determination of weights in the modeling process was challenging, since it is erroneous to adopt a single objective or subjective weighting technique for the risk evaluation of floor water inrush. This might result in too much subjective arbitrariness in the evaluation results or too much concentration on the data itself. As a result, we first utilized the level analysis to determine the weights and subjectively analyzed the value of each index. Next, we applied the objective entropy weight method to estimate the relative relevance of the indicators and assigned weights, then we assigned the weights to establish the comprehensive weights, which improved the accuracy and reliability of the evaluation results [27,28]. The model building process is shown in Figure 13.

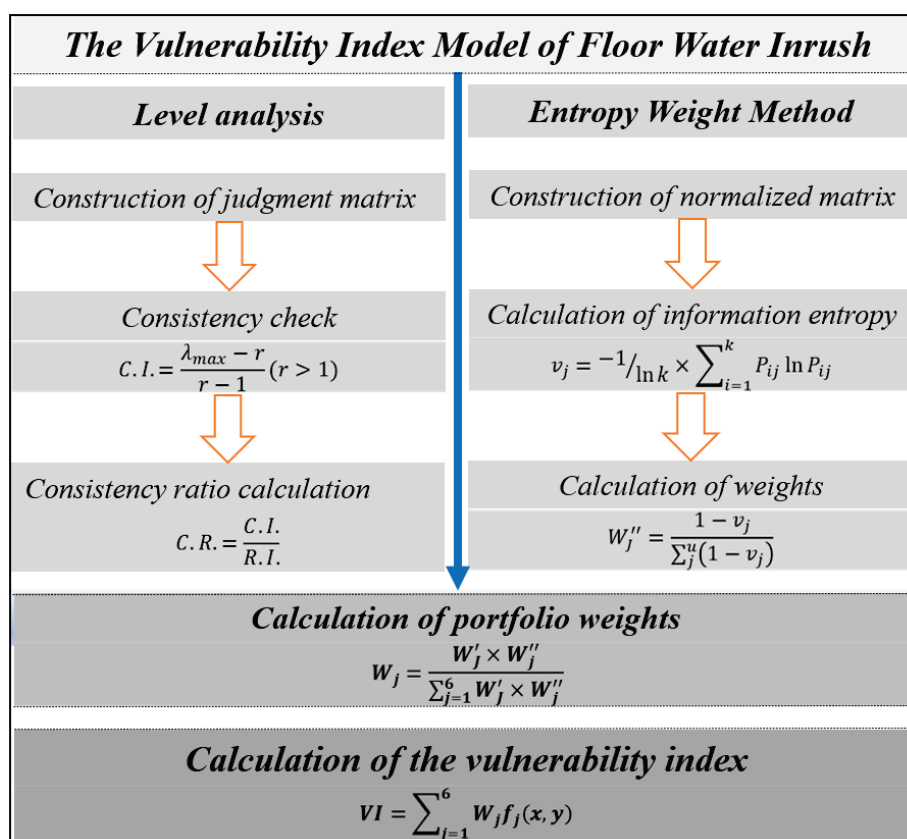


Figure 13. The vulnerability index model of the floor water inrush building process.

5.1. Analysis of the Risk of Floor Water Inrush within the Range of the 182602 Working Face

The microseismic kernel density, three elements of water abundance (water volume, water pressure, grouting volume), and two elements of tectonic (fault, syncline) explained the floor rock state and the risk of floor water inrush from diverse angles. According to the preceding paragraph, one of the characteristics of pre-mining microseismic events was clustering, the dense zone of microseismic activity was always formed in the zones with relatively developed fractures, relatively broken rock strata, and a high water abundance under the action of artificial disturbance. Therefore, the higher the value of the microseismic kernel density, the higher the fragility of the rock strata and the higher the risk of floor water inrush. In addition, zones with high water abundance had a high risk of floor water inrush.

In the tectonic zones, fractures were well-developed, which were easy to communicate with the Ordovician limestone aquifer below, and the risk of water inrush was high.

We examined the risk of floor water inrush within the 182602 working face. First, the above six elements were used as research indicators, and the weights W' and W'' were determined from the subjective and objective perspectives by using level analysis and the entropy weight method, respectively, and the comprehensive weight W was determined. After that, the vulnerability index model was used to evaluate the risk of floor water inrush. Finally, the risk zoning evaluation chart was drawn with the help of ArcGIS technology. The comprehensive weights were calculated as shown in Table 4.

Table 4. The comprehensive weights of the six research indicators.

Research Indicators	W'	W''	$W' \times W''$	W
Microseismic kernel density	0.202	0.213	0.0345	0.1866
Water inflow (m^3/h)	0.247	0.268	0.0637	0.3442
Water pressure (MPa)	0.213	0.175	0.0373	0.2014
Grouting volume (tons)	0.084	0.063	0.0097	0.0522
Fault (m)	0.072	0.057	0.0073	0.0397
Syncline (m)	0.182	0.224	0.0326	0.1759

The product of the comprehensive weights of the research indicators and the quantified normalized value were calculated, that is, the vulnerability index model of the floor water inrush was established. The VI values were calculated and counted, and the normalized thematic charts of the research indicators were drawn by using information processing technology, as shown in Figure 14.

$$VI = 0.1866f_1(x, z) + 0.3442f_2(x, z) + 0.2014f_3(x, z) + 0.0522f_4(x, z) + 0.0397f_5(x, z) + 0.1759f_6(x, z) \quad (1)$$

where VI is the risk degree indicator; $f_j(x, z)$ is the quantified normalized value of the j influencing elements.

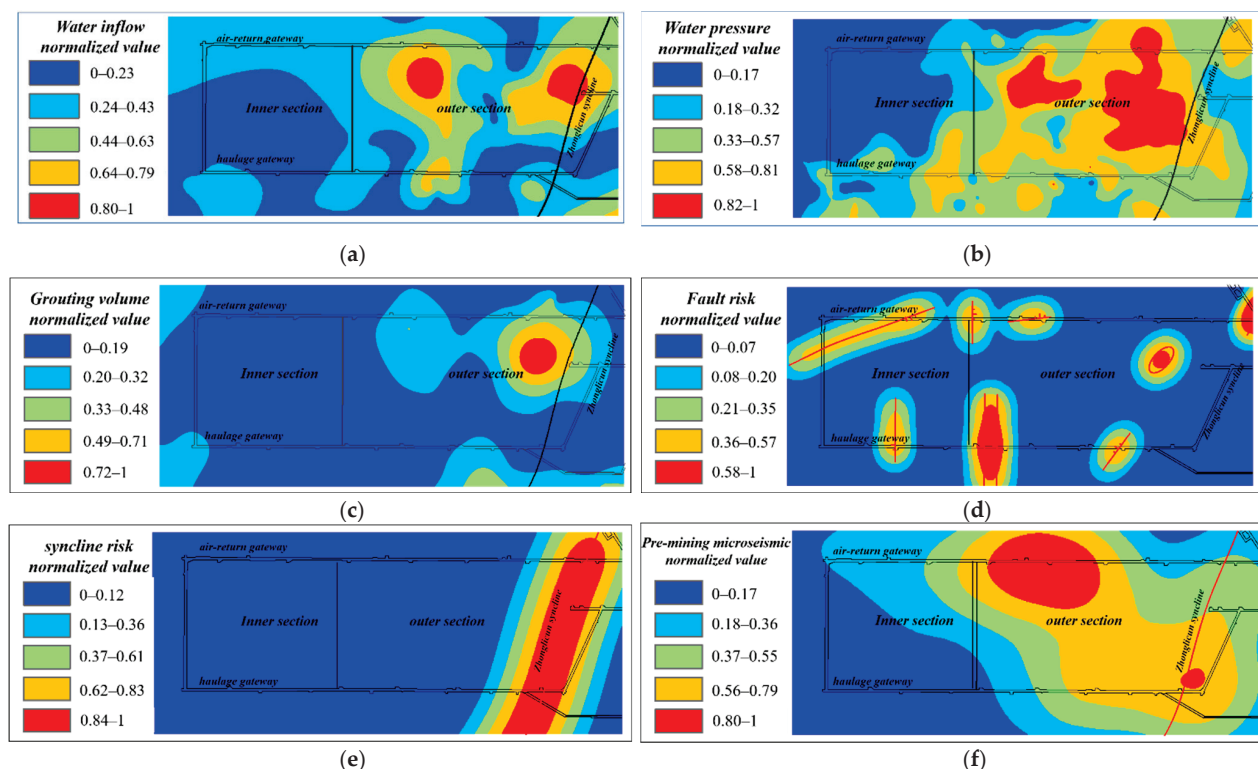


Figure 14. Normalized thematic charts of the research indicators. (a) Water inflow, (b) Water pressure, (c) Grouting volume, (d) Fault, (e) Syncline, (f) Pre-mining microseisms.

The consequence of the combined influence of several study indicators was the floor water inrush, which is a complex process governed by multiple elements. As a result, the thematic charts of each research index were superimposed to create the final comprehensive evaluation chart of floor water inrush, and natural breaks (Jenks) were used to categorize the comprehensive evaluation of floor water inrush into five levels: level I–level V, with level I being the most dangerous zone and level V being the relatively safe zone, as shown in Figure 15.

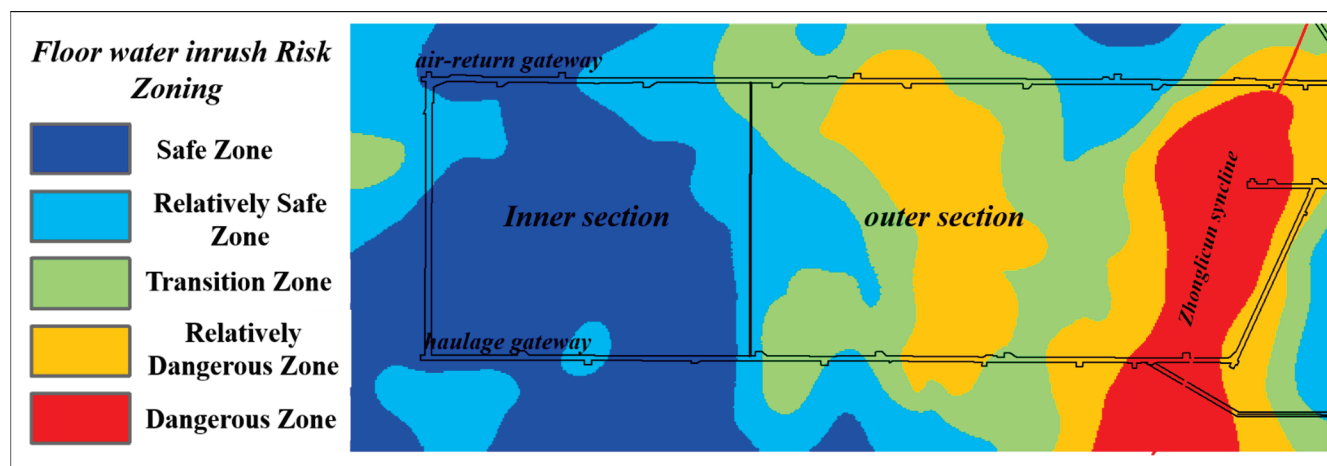


Figure 15. The comprehensive risk evaluation of the floor water inrush within the 182602 working face.

It can be seen from the figure that the water inrush dangerous zone and relatively dangerous zone were mainly distributed in the outer section near the Zhonglicun syncline side. Compared with the calculation results of the water inrush coefficient, dangerous and relatively dangerous zones were also the zones with the maximum water inrush coefficient (a water inrush coefficient greater than 0.06). The dangerous and relatively dangerous zones were also zones with high water abundance. At the same time, the dangerous and relatively dangerous zones were also zones with the most concentrated pre-mining microseisms. According to the analysis, the hydrogeological characteristic in the two flanks of the Zhonglicun syncline was complex, making it the most likely zone for floor water inrush. Therefore, when the working face was mined to this zone, we should pay close attention to the changing trend of water inflow in the goaf, make accurate judgments in combination with the actual situation of the site, and specify reasonable water inrush countermeasures.

5.2. Geophysical Exploration of the Risk of Floor Water Inrush

To make the vulnerability index model of floor water inrush based on pre-mining microseisms more trustworthy and to investigate the hydrogeological conditions in the working face and the surrounding area, we employed the network parallel electrical method to conduct the physical analysis of the floor rock layer [29]. The method was used to obtain the apparent resistivity of the rock mass according to the conductivity difference between the rocks and ore bodies. Low apparent resistivity represents the high degree of fracture and water abundance of the rock layer in the exploration zones.

We used the YBD11 mining network parallel electrical instrument to place electrodes and an electrical survey line in the haulage gateway, air-return gateway, and open-cut in turn, with a total of 13 exploration points and 64 electrodes per exploration point. According to previous exploration experience, the apparent resistivity of the abnormal zones of the Wutongzhuang mine was generally below 20 $\Omega \cdot m$. The results of the geophysical exploration are shown in Figure 16. In the vertical direction, the abnormal zones were mainly developed in the Shanvuqing layer. In the horizontal direction, there were two

abnormal zones DF1 and DF2 in the inner section, and three abnormal zones DF3, DF4, and DF5 in the outer section of the working face. However, the results of the borehole exploration and verification showed that DF1 and DF2 in the inner section were weakly water abundance zones and DF3, DF4, and DF5 in the outer section were high water abundance zones. Compared with the vulnerability index model, the spatial distribution of the abnormal zones DF3, DF4, and DF5 were found to be essentially the same as that of the water inrush dangerous zone and relatively dangerous zone. In conclusion, the microseismic-based water risk assessment method is accurate and dependable, and it may be used to guide and inform water management in coal mines.

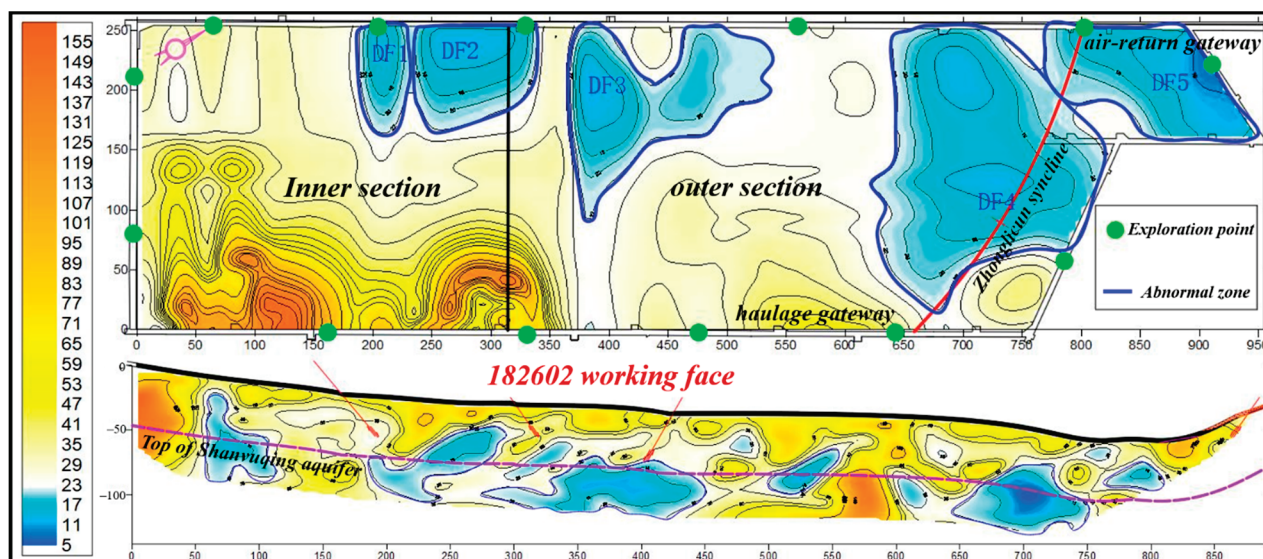


Figure 16. The geophysical exploration of the risk of floor water inrush.

6. Discussion

Pre-mining microseisms will be triggered in the fracture development zones under artificial disturbance such as grouting works. The high-pressure slurry caused local stress concentration in the fractures and cavities in the formation, so the cracks and cavities split and expanded. The elastic strain energy released during this process led to the occurrence of microseismic events. As the grouting work progressed, the fracture network was filled and the formation was strengthened. The water-conducting channel of groundwater was blocked and the groundwater flow field was rebalanced, greatly reducing the nuclear density of microseismic events. Based on this formation mechanism of the pre-mining microseisms, a vulnerability index model of floor water inrush was built using level analysis and the entropy weight method. The 182602 working face has had second floor grouting reinforcement carried out in the predicted dangerous zones. Twenty-six boreholes have been constructed, and the final hole depth exceeded the Shanvuging aquifer. Through the targeted grouting treatment measures, the efficient and safe mining of the working face was finally realized.

Unfortunately, the microseismic monitoring system is often installed just as the working face is about to be mined, which makes it very difficult to monitor the pre-mining microseismic events during the grouting period. Limited data were used, which reduces the accuracy and reliability of this study. In subsequent research, we will focus on new study areas such as the Jiaozuo mining area and Xingtai mining area, and further deepen the law and mechanism of pre-mining microseisms.

7. Conclusions

In this paper, taking the 182602 working face of the Wutongzhuang coal mine as the research background, combined with the plane, spatial, and energy characteristics of microseismic events before mining, the concept of pre-mining microseisms was proposed.

On this basis, the correlation between microseismic events and the water abundance of the aquifer was considered, and an evaluation method with regard to floor water inrush was constructed based on “three elements” of the aquifer and pre-mining microseisms. The main conclusions are as follows.

(1) Under the artificial disturbances including slurry diffusion and neighboring mining, there were relatively dense microseismic events in the zones where water-conduct channels are prone to occur. This type of microseismic event can be called pre-mining microseisms, which is characterized as being “sporadic, cluster and phased”.

(2) It was commonly assumed that the obvious performance characteristics were water inflow, water pressure, and grouting volume in the water abundance of the Shanvuqing aquifer. It was verified that the water abundance of the outer section was significantly higher than that of the inner section. The correlation between the microseismic events and three high drilling holes was noticeable with values of 0.86, 0.82, and 0.71, respectively. On this basis, a risk assessment method of water source along the strike of the working face was proposed. Two dangerous zones were determined.

(3) According to the generated vulnerability index model, the dangerous zone and relatively dangerous zone were determined comprehensively in the outer section of the working face, which were basically consistent with the observation result of the DC method. As such, the zones were at risk of forming water-conducting channels.

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Article

Experimental Study of Confined-Pressure Soaking on Sandstone Damage-Fracture Characteristics Using Acoustic Emission and Energy Dissipation

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Abstract: In order to explore the mechanical characteristics of confined-pressure soaking (CPS) sandstone during the damage-fracture process, the uniaxial compression and acoustic emission (AE) experiment of CPS sandstone was conducted. The mechanical parameters, fracture morphology, AE and energy characteristics under uniaxial stress were researched. The results show that the uniaxial compressive strength and elastic modulus decrease as a logarithmic function with CPS parameters, corresponding to the inflection point of modification. The fracture mode gradually changes from brittleness to toughness with the increase in moisture content. The difference between fracture fragmentation and mechanical behavior mainly depends on the degree of CPS. The evolution of AE characteristics and energy dissipation has a good correspondence with CPS specimens. The higher the CPS degree is, the less the AE cumulative number is and the longer the excitation time is. Based on the phase variation of AE events and energy dissipation, the CPS damage variables are proposed to quantify the weakening degree and reveal the damage evolution process. The results provide a useful foundation for evaluating rock failure and improving service life.

Keywords: confined-pressure soaking (CPS); mechanical characteristics; fracture morphology; acoustic emission; damage evolution

1. Introduction

At present, confined-pressure water blasting technology is one of the effective measures for pre-splitting hard roofs and preventing rockburst in coal mines in China [1]. Coupling blasting with confined pressure water is to fill the borehole with water instead of air and the surrounding rock could reach a specific soaking time in the state of pressure stabilization. Then the charge initiates to achieve a better fracturing effect. The drilling fractured rock mass forms a certain range of immersion zone, and meanwhile, the soaking range and degree will be further strengthened under the effect of confined-pressure increasing permeability, so as to obtain a large range of soaking modified rock mass [2–4]. The time and pressure change of confined-pressure water soaking (CPS) stimulates the response of rock mass to varying degrees (moisture content), inducing the microstructure modification of rock mass [5–7]. Therefore, it is of great significance for rock stability monitoring and disaster early warning in coal mine production to master the characteristics of deformation and failure precursory indicators of CPS rock [8].

After loading, the crack closure and secondary propagation of rock mass will be accompanied by the release of elastic wave, which propagates rapidly and produces acoustic emission (AE) events [9]. Therefore, understanding the AE characteristics of rock is helpful to master the law of crack propagation and damage evolution. There is an internal relationship between the mechanical characteristics and AE damage, and the confined-pressure water will weaken the rock samples, affecting the relationship between

the parameters [10]. As for the influence of water on rock weakening, scholars have done a lot of research and achieved some research results. Yao et al. [11] proposed the new Mohr-Coulomb model of coal samples with various moisture content and analyzed the effect of water level on the tensile failure planes as well as the percentage of tensile failure planes versus the total. The increasing moisture content promotes the possibility of shear failure. The weakening degree mainly depended on the clay mineral content [12]. Zhang et al. [13] pointed out AE activities could be divided into compaction and elastic deformation, non-elastic deformation and fracturing failure phase and proposed a new damaged model to describe the coal damage. Qin et al. [14] carried out the uniaxial compression mechanical and AE tests of rock samples with different moisture content and clarified the influence of moisture content on AE characteristics. Teng et al. [15] analyzed the mechanical and AE characteristics of dry and saturated sandstone under different loading rates. The constitutive model was proposed and confirmed based on the softening factors of elastic modulus (EM). Geng et al. [16] pointed out a gradual change of the sandstone deformation mode from brittle to ductile with moisture content increasing. Based on AE accumulative and dissipated energy theory, damage variables were proposed which could reflect the damage evolution during the compression. Cai et al. [17,18] analyzed the influence of water saturation on the mechanical behavior of different rock types, obtaining the influence characteristics of immersion on the strain, uniaxial compressive strength (UCS) and EM. The weakening degree depended on the porosity and content of the expansive clay. Shen et al. [19] found that water reduced the strength and brittleness of coal by destroying the microstructure and resulting in elastic energy accumulation. Zhu et al. [20] explored the crack activity of saturated rocks at key stress points by uniaxial compression and AE tests. The results showed due to the existence of water, the number of AE events decreased relatively, indicating that water enhanced the ductility. Qin et al. [21] analyzed the compression failure characteristics of coal samples and obtained the response of strength and AE characteristics to moisture content. The increase in moisture content resulted in a decrease in AE accumulation and an increase in excitation time lag. Xia et al. [22] carried out experimental research on AE characteristics during cyclic loading and unloading of dry-saturated rock.

To sum up, the stress-strain behavior, fracture mode, energy and AE characteristics of rock are sensitive to water immersion, and the weakening degree of for different types of rock mainly depends on the content of expansive clay. The literature research mainly focuses on the weakening damage effect on rock, with natural soaking time and moisture content as the influencing factors. Compared with natural soaking, the weakening effect of confined-pressure water is different in that confined-pressure water activates the propagation of micro-cracks, and further strengthens the modification of rock. There are few studies on the response of mechanical and acoustic characteristics of modified sandstone by confined-pressure water to the soaking time. In addition, the influence of confined-pressure variation on the damage characteristics of sandstone has not been reported, and the mechanism of confined-pressure water and rock interaction during loading is unable to verify. Therefore, it is the feature of the research to effectively correlate CPS parameters with sandstone damage (based on AE and energy dissipation).

Therefore, taking the sandstone roof of Datong Mining Area as the research object, uniaxial compression tests were carried out on the soaking specimens through MTS and AE test systems. The stress-strain curves, mechanical properties, fracture morphology, AE events and energy were closely linked together, and the geometric relationship with CPS parameters was established. Then, from the point of view of the phased characteristics of AE events and the evolution of energy dissipation, the damage evolution characteristics of CPS sandstone at different loading stages are quantified. The damage variables under different CPS degree are derived and established, and the applicability is obtained. Combined with the engineering environment of confined pressure blasting in a coal mine, the work comprehensively explored the evolution characteristics of the sandstone mechanical

damage under different CPS parameters and their quantitative relationship, so as to better apply it to coal mine safety production.

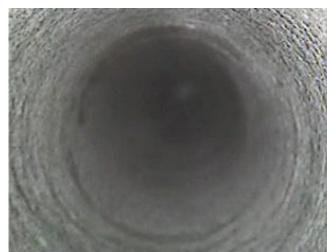
2. Test Scheme

2.1. Sandstone Specimens Preparation

The testing core was taken from the main roof of 8939 working face in Xinzhouyao mine of Datong Mining Area, China. The integrated core and drilling borescope were shown in Figure 1. The surface of sandstone is grey, and joint characteristics are not significant. According to GB/T23561.7-2009 “Methods for determining the physical and mechanical properties of coal and rock”, the rock was processed into cylinders perpendicular to stratification with dimensions of $\Phi 50 \times 100$ mm. The average mass of the rock sample is 494.5 g, and the average natural density is 2526 kg/m^3 . The irregularity error of ends was controlled within 0.05 mm to ensure good contact between the tester and specimen ends. The specimens with obvious appearance defects were removed, and the longitudinal wave velocity of the samples was measured (average value was 2641 m/s). Rock blocks with similar wave velocity were selected as the experimental reserves.



(a) Sandstone core and graph.



(b) Drilling borescope

Figure 1. Selection of sandstone core.

The sample was analyzed by XRD, and the diffraction pattern is shown in Figure 2.

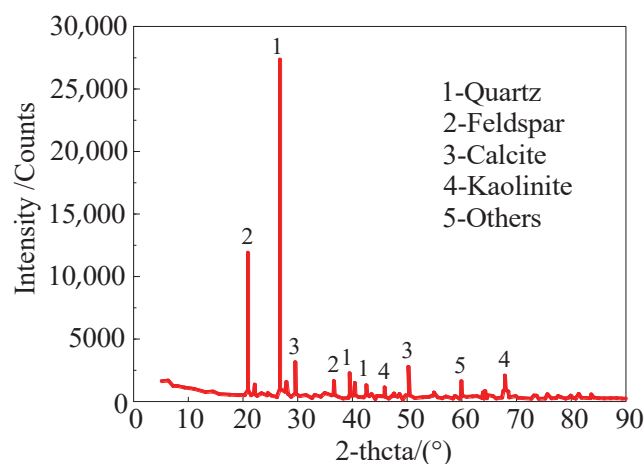


Figure 2. X-ray diffraction figure of mineral composition of sandstone.

The content of quartz and feldspar in sandstone is the highest, followed by calcite, and the least is kaolinite, among which kaolinite is greatly affected by water soaking. Kaolinite has strong water absorption and has expansion and plasticity after soaking. Because the content of kaolinite is relatively low, the soaking and modification effect of sandstone will be lower than that of mudstone or argillaceous siltstone.

2.2. Testing Instrument and Parameter Design

Before loading, the independent design and detachable CPS device were used to realize the different CPS degrees of specimens, as shown in Figure 3.

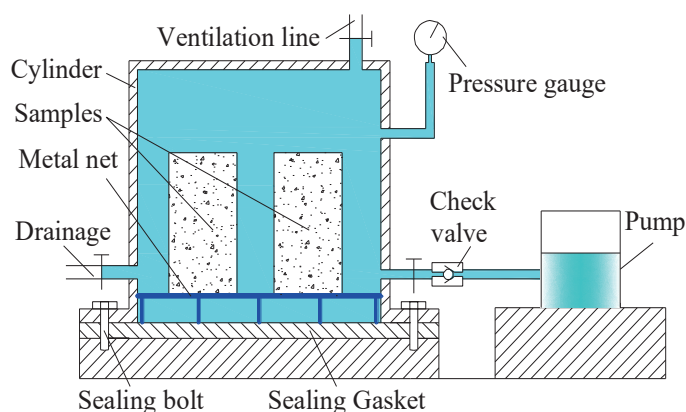


Figure 3. Confined-pressure soaking device for coal-rock.

Before conducting the experiment, the sealing chamber is opened to place the specimens on the loading platform, and then the device is reassembled and sealed. The water is injected into the cavity through the pump and water injection to soak the rock sample until the cavity is full. In the simulation of the CPS environment, the pressure relief valve and pressure gauge monitoring are used to achieve the specific soaking time of samples under different stable CPSPs.

AE monitoring adopts the PCI-2 acquisition system, and relevant parameter settings are shown in Table 1. Paste the strain gauge and AE sensors (piezoelectric AE sensor Nano30, bandwidth 125~750 KHz, resonant frequency 140 KHz) according to Figure 4. After connecting the corresponding equipment with the acquisition system, it is necessary to conduct several sets of broken-lead experiments to ensure that each sensor is in the best coupling state with the rock sample surface. Only in this way could the AE sensor stably receive high-quality signals during the experimental process.

Table 1. Parameter setting of AE acquisition system.

Threshold/dB	Sampling Rate/MHz	Pre-trigger/ μ s	Peak Definition Time (PDT)/ μ s	Hit Definition Time (HDT)/ μ s	Hit Locking Time (HLT)/ μ s
40	5	256	50	200	300

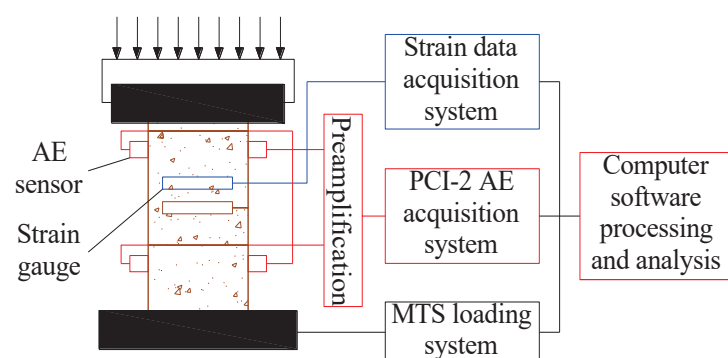


Figure 4. Test system of uniaxial compression and AE monitor for CPS sandstone.

CPS specimens were loaded by an electro-hydraulic servo universal tester with a displacement loading rate of 0.5 mm/min. The stress-strain data acquisition system used a static strain gauge, displacement sensor and stress sensor to monitor the deformation and loading of specimens. Combined with the actual production, it was determined that the confined-pressure soaking time (CPST) of rock specimens was 0~24.0 h and confined-pressure soaking pressure (CPSP) was 0~8.0 MPa.

3. Results and Analysis

3.1. Mechanical Characteristics of CPS Sandstone

The sandstone contains water-absorbing minerals such as clay, which makes the moisture content increase continuously and micro-structure modification. Especially for CPS, the degree of modification and weakening is enhanced [23]. In order to analyze the mechanical properties under different CPS degrees, the moisture content is measured, and the sensitive range of influencing factors on modification is analyzed. Meanwhile, the uniaxial compression test is carried out to explore the response characteristics of UCS and EM to CPS parameters.

3.1.1. Analysis of Characteristics of Soaking Degree for Sandstone

The variation curves of moisture content under different CPS degrees are shown in Figure 5.

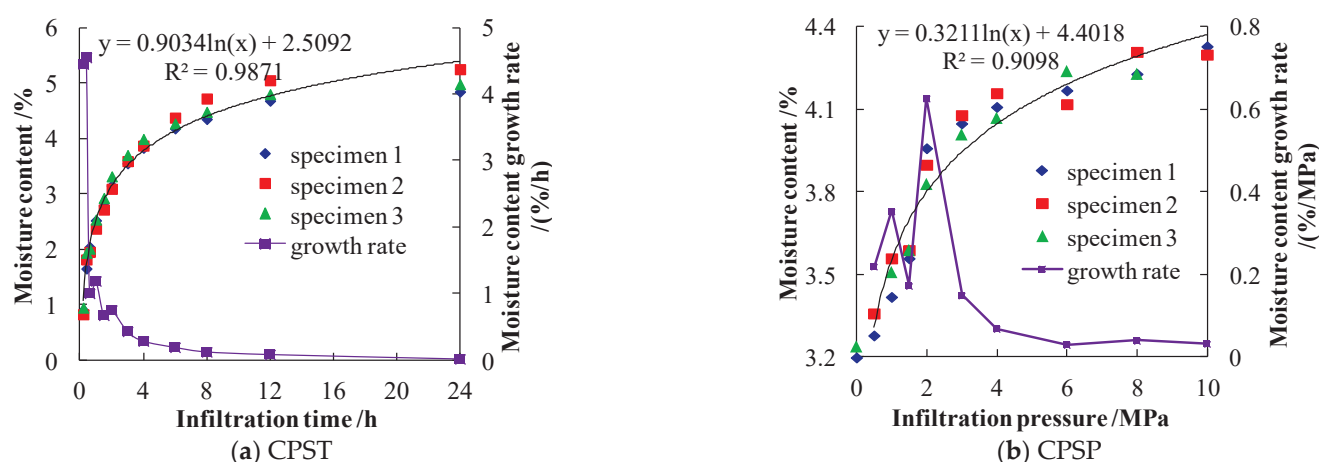


Figure 5. Moisture content variation under different CPS degrees.

The moisture content of CPS specimens shows the characteristics of stage growth. For CPST, the main growth stage is 0~12.0 h with the rate gradually decreasing, which indicates the water absorption-dissolution-modification process is basically completed within 12.0 h. For CPSP, the main growth stage is 1.0~6.0 MPa (main stage: 1.5~3.0 MPa), during which CPS has a significant effect on micro-modification. In other words, the permeability and mudding resistance are the most significant in 1.5~3.0 MPa. When the pressure exceeds 6.0 MPa, the response of moisture content to CPSP decreases obviously.

The variety of moisture content could indirectly reflect the response of modification intensity to CPS degree of sandstone. Therefore, the factor interval (CPST: 0~24.0 h, CPSP: 0~6.0 MPa) affecting the modification degree is determined.

Because the moisture content increase in sandstone caused by CPS is different from that of the natural soaking, in order to verify the reliability of the test conclusion and innovation, the paper compares the change of moisture content by CPS with that in the relevant literature. The natural moisture content satisfies a variety of positive correlation functions in terms of soaking time [24,25], while the moisture content induced by CPST in the paper satisfies a logarithmic function. Taking red sandstone as an example [26], the increase in moisture content obtained by comparing CPS is shown in Equation (1). However, the changing trend of the moisture content for natural and CPS is similar, which could be approximately divided into three stages and the growth rate of each stage is consistent [27,28].

$$\begin{cases} \text{CPST : } w_t = 0.9034 \ln(t) + 2.5092 \\ \text{Natural : } w_t = -4.26e^{-t/7.2} + 4.27 \end{cases} \quad (1)$$

where, t is the soaking time, including natural and CPS state; w_t is the moisture content. The formula could be used for forecasting the moisture content through the soaking time.

For the increase in CPSP, the moisture content increase in the soaking process is similar to that of the CPST state. Therefore, the conclusion obtained based on CPS parameters is consistent with the previous research, indicating the reliability of the experimental conclusion.

3.1.2. Mechanics Characteristics of Sandstone under Different CPS Degree

The stress-strain curves of specimens under different CPSTs are shown in Figure 6.

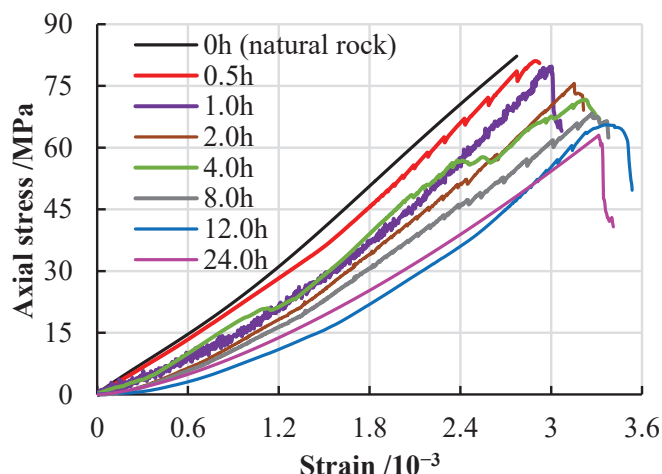


Figure 6. Stress-strain curves under different CPSTs.

CPST has a certain influence on the loading process of the specimens. For the natural sandstone, the stress-strain curve increases approximately linearly without an obvious yield stage, showing brittle fracture. With the increase in CPST, the crack initial-closure section of loading specimens grows, which indicates that CPST increase activates the development of primary cracks. The middle of the curve shows “lower convex” and the nonlinear deformation period of the natural samples is shorter than that of CPS samples, showing the transition from the elasticity of natural specimens to the non-linear loading of CPS specimens. After peak stress, the specimens show a sudden drop-transition decline, and some have the post-peak bearing capacity, which indicates that the failure of specimens gradually transit from brittle to non-brittle.

Variations of UCS and EM under different CPSTs are shown in Figure 7.

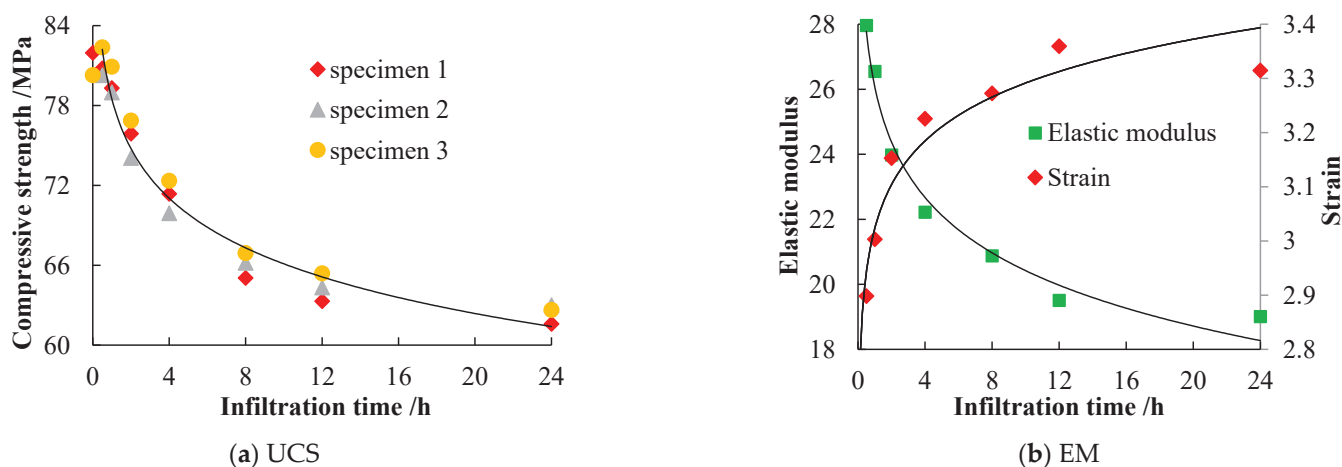


Figure 7. Mechanical parameters variation with different CPST.

With the increase in CPST, UCS and EM decrease in the form of logarithmic function and the various trend is similar, which is in good agreement with the change of moisture content. The rapid attenuation stage is 0~8.0 h, corresponding to UCS decreasing from

82.24 MPa to 66.04 MPa (EM: 29.66 GPa to 20.88 GPa). Compared with natural specimens, UCS and EM are reduced with 21.0% and 29.6%, respectively. The attenuation rate obviously decreases in 8.0~24.0 h, and UCS and EM are reduced with 27.0% (UCS) and 36.0% (EM), respectively. The failure strain corresponding to UCS increases, and the attenuation strength of EM is higher than that of UCS, which indicates that the failure of specimens transit from brittleness to non-brittleness. Based on the attenuation trend of UCS and EM, the sensitive CPST range of weakening is 2.0~8.0 h.

The stress-strain curves of specimens under different CPSP are shown in Figure 8.

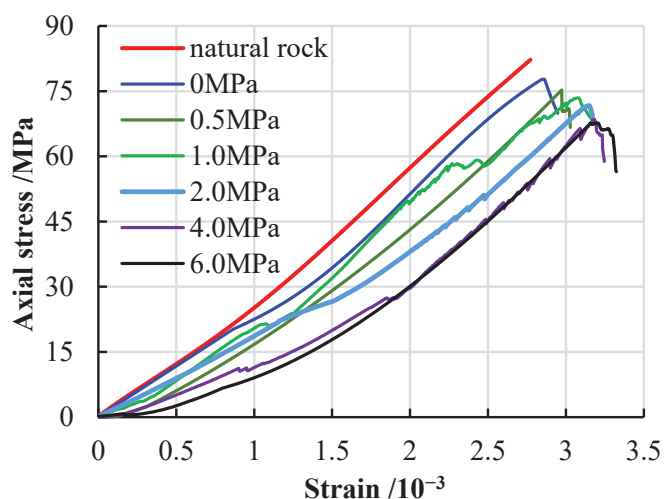


Figure 8. Stress-strain curves of specimens under different CPSP.

The variation of the stress-strain curves on the CPSP is similar to that of CPST, presenting “lower convex” growth as a whole. With the increase in CPSP, the crack closure process increases (the growth stage of CPSP is lower than that of CPST, that is, the crack activation and propagation degree decrease), and the convex bending amplitude increases. When CPSP is lower than 1.0 MPa, the stress peak has no obvious yield stage. When the CPSP exceeds 2.0 MPa, the curve has an obvious drop process, and there is no obvious post-peak bearing capacity.

Figure 9 shows the variation of UCS and EM with different CPSPs.

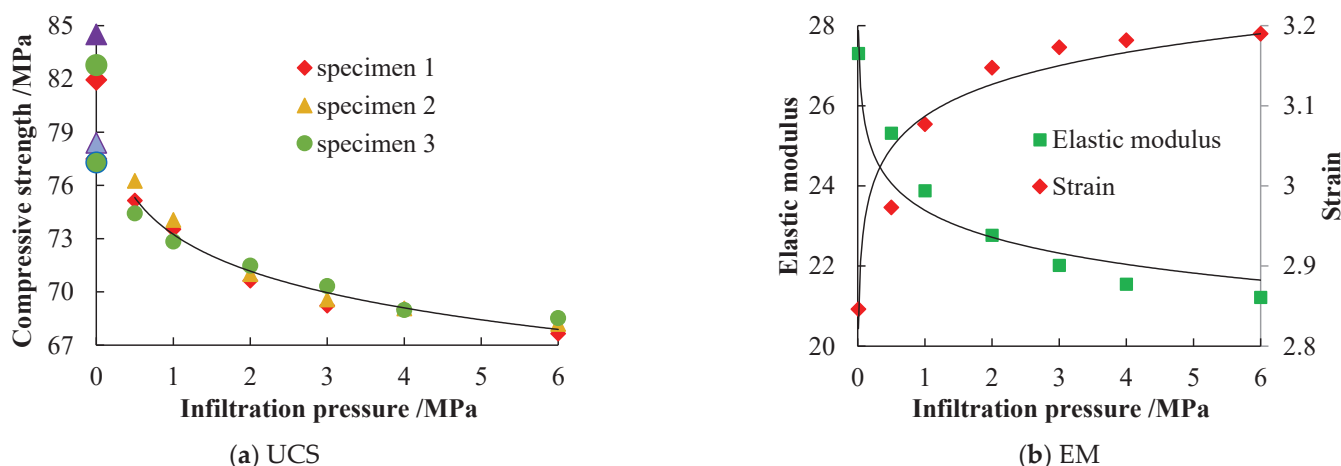


Figure 9. Mechanical parameters variation with different CPSP.

The attenuation law of UCS and EM on CPSP is similar to that of the CPST in logarithmic function, which is in good agreement with the moisture content. The attenuation could be divided into three stages: when CPSP is lower than 2.0 MPa, UCS decreases from 82.24 MPa (natural) to 71.04 MPa (29.66 GPa to 22.02 GPa for EM), which is 13.6% and

23.2% lower than that of the natural. The attenuation degree decreases at 2.0~4.0 MPa with 16.1% and 27.3% (UCS and EM) lower than that of the natural state. The UCS and EM of the specimen decreased by 17.2% and 28.5%, respectively, within 4.0~6.0 MPa. The peak strain corresponding to the peak strength shows an increasing trend, indicating that the failure of the specimen varies from brittle fracture to non-brittleness. Based on the attenuation trend of UCS and EM, the sensitive value of CPSP is determined at 2.0~4.0 MPa.

The decrease in UCS and EM is an important feature of sandstone softening under CPS. The quantitative relationship between UCS (EM) and CPST/CPSP is proposed by fitting:

$$\left\{ \begin{array}{l} t : \left\{ \begin{array}{l} \sigma_c(t) = -5.379 \cdot \ln(t) + 78.488 \\ E(t) = -2.457 \ln(t) + 26.077 \\ \varepsilon(t) = 0.1165 \ln(t) + 3.0232 \end{array} \right. \\ p : \left\{ \begin{array}{l} \sigma_c(p) = -2.99 \cdot \ln(p) + 73.243 \\ E(p) = -0.974 \ln(p) + 23.393 \\ \varepsilon(p) = 0.0575 \ln(p) + 3.0868 \end{array} \right. \end{array} \right. \quad (2)$$

where, $\sigma_{c(t)}/\sigma_{c(p)}$ and $E(t)/E(p)$ are UCS and EM under different CPSTs and CPSPs.

In order to illustrate softening effect of CPS on sandstone, the weakening characteristics of conventional soaking in related research are compared. In the paper, the test results are compared with the strength parameters of samples with different moisture contents, as shown in Figure 10. The curves fitted in the previous literatures are mainly negative exponential and quadratic functions [16,29–31]. Meanwhile, it is found that UCS and EM satisfy the negative logarithmic function for CPST and CPSP. For CPSP growth under a specific CPST, the mechanical parameters continue to decay, indicating that the CPS weakening is the coupling effect of soaking time and high-pressure water. The increase in CPST and CPSP are also accompanied by the continuous growth of moisture content. Therefore, the UCS and EM of the specimen decrease with the increase in moisture content [31], whether from CPS or natural soaking. The results show the reliability of the experimental data and also show the weakening effect of the confined-water and rock coupling.

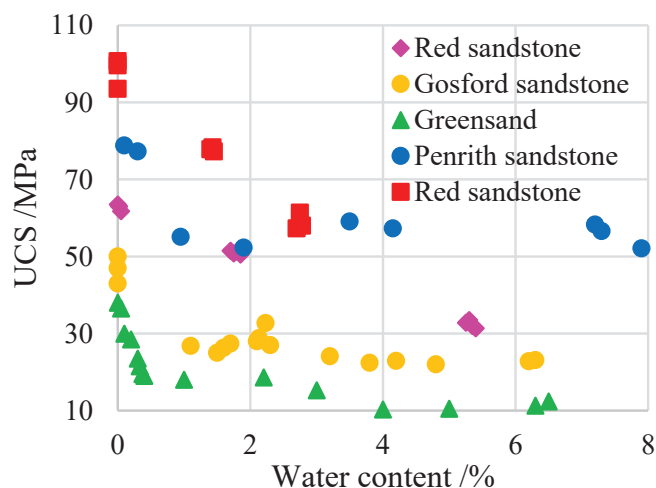


Figure 10. Variation of moisture content and uniaxial compressive strength of various sandstones.

3.1.3. Fracture Characteristics of CPS Sandstone

After CPS, minerals will be muddled, and internal micro-cracks will be activated. Meanwhile, the water absorption would cause the modification of micro-structure, and the fractures mainly propagate along the activated micro-cracks zone [32]. Failure pattern under different CPSTs is shown in Figure 11.

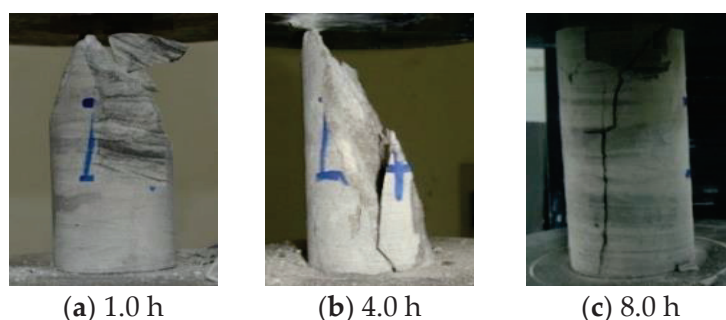


Figure 11. Failure patterns of specimens under uniaxial compression with different CPST.

With the increase in CPST, the specimens transit from local fracture to the overall. When soaking 1.0 h, the fracture is concentrated in the shallow layer of the middle and upper part, while the lower remains intact; it indicates that the short-term soaking causes the modification heterogeneity and results in the formation of local stress concentration during the loading, showing fracturing regional centralization and fragmentation irregularity. When soaking 4.0 h, the specimen shows the characteristics with the upper fracturing and the lower splitting, and the cracks penetrate along the axial direction, during which the fracture surface is irregular due to the change of fracture direction. At soaking 8.0 h, the fracture morphology of specimens is similar to that of the natural: a single main crack penetrates along the axial direction, and the fracture surface is regular, showing a homogenized strength weakening effect. The failure presents a transition to non-brittle fracture in CPST.

Figure 12 shows the fracture patterns of specimens with different CPSPs. Under low pressure (1.0 MPa), the fracture trajectory propagates irregularly, accompanied by spalling. The main crack propagates to the middle of the specimen, produces lateral rotation, and then develops stably until the specimen penetrates and ruptures, during which the main-wing cracks propagate alternately. Under 2.0 MPa, the upper part of the specimen propagates along two cracks and converges into a main crack at 3/4 height. Meanwhile, a lateral wing crack is developed in the middle of the specimen and the right side of the loading failure specimen is relatively intact, which still has high axial bearing capacity. Under the pressure of 6.0 MPa, the fracture trajectory of the specimen will show irregular development: the crack propagation has multi-directionality, presenting multiple-cracks and multiple-pieces; it indicates that when CPSP exceeds the sensitive weakening value, the fracture dimension increases; this is due to the increase in CPSP, the weakening degree of specimen increases, which is different from the homogenization weakening characteristics under CPSTs. Under a high CPSP, the internal weakening of the specimen presents multi-direction-irregularity, and the deterioration degree is more significant.

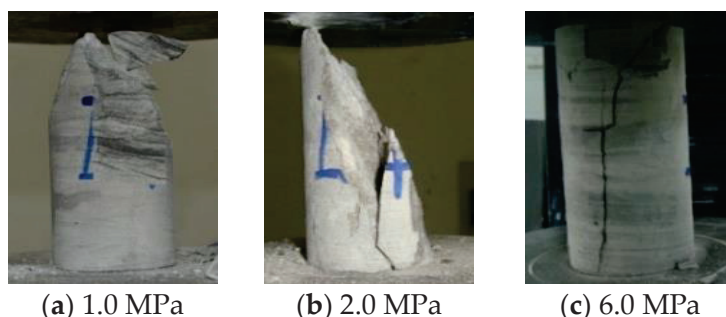


Figure 12. Fracture patterns of specimens under uniaxial compression for different CPSP.

The failure and fracture modes of CPS and natural soaking rock samples are similar with the increase in moisture content. Under three soaking conditions, the modification effect of low-medium moisture content on rock samples is mainly shear failure [33,34]. For high moisture content, the specimens subjected to CPS and natural soaking tend to be

homogeneous in modification; their fracture modes are consistent, showing longitudinal tensile cracks and local fragments [35,36]; this shows the reliability of the experimental results of the CPST effect. For high confined pressure, the modification and weakening degree of specimens increases. The specimens have a higher level of fragmentation and more small fragments are produced when a fracture occurs, which further reveals the weakening effect of confined-pressure water on sandstone; this is one of the characteristics of CPS modification and weakening.

3.2. AE Characteristics of CPS Sandstone during Loading

3.2.1. Comparison of AE Characteristics of Sandstone under Different CPS Degree

The loading specimen fracture is the combined action of the internal micro-crack closure, energy accumulation, transformation and release [37,38]. AE monitoring system could receive the signal of a damage-failure process for loading specimen, reflecting the crack propagation [39,40]. The CPS modification would inevitably lead to the change of fracture characteristics. The accumulation number of AE event is selected as a quantitative parameter for analyzing the relationship between AE signal and strain and its response to CPS degree. AE event number-strain curves with different CPSTs and CPSPs are shown in Figure 13.

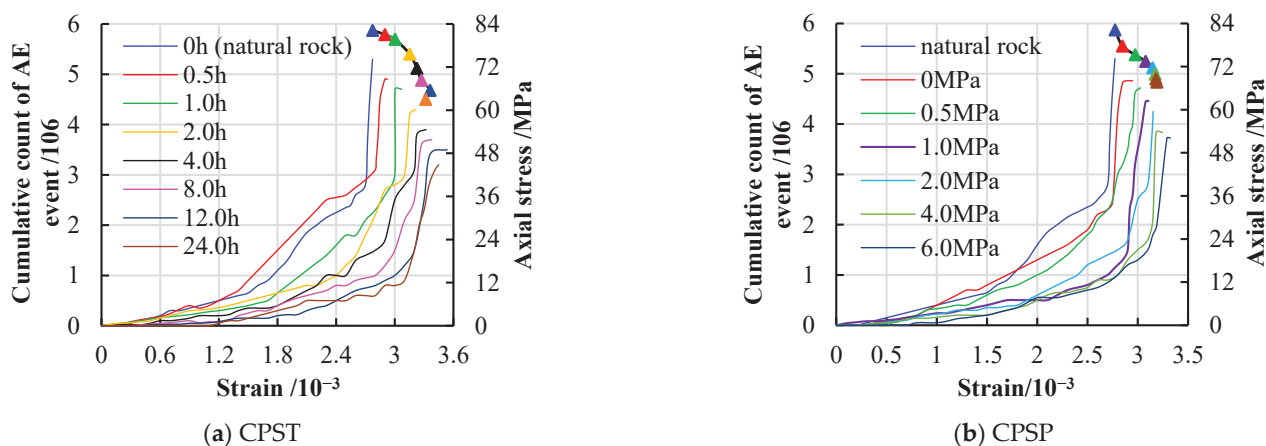


Figure 13. Accumulative number of AE events-strain curve under different CPS degree.

Combined with the stress-strain curves, it can be seen that the evolution of AE cumulative events with different CPS parameters in the loading is consistent. The variation of AE events could be divided into four stages: the first is the primary micro-fracture closure stage, in which the AE signal is very little. The second is elastic deformation, in which the cumulative number of AE events increases slowly, but the growth rate and loading duration is different under different CPS degree. The third is the rapid damage accumulative stage, and the fourth is the fracture-instability stage. With the increase in CPS degree, AE cumulative events vary from step-up to rapid growth in the stage.

In the primary fracture closure stage, AE events of all specimens is very few, but the loading strain of cracks closure stage increases. The reason is that the stress level of the specimen is low at the initial loading stage, and the primary crack is mainly compacted and closed without crack propagation. With the increase in CPS degree, the intensity of mudding and micro-structure modification increases, and the corresponding number of micro-cracks propagation decreases, resulting in the corresponding strain increasing.

The second stage is the elastic deformation, in which AE event accumulation number shows a stable increase. CPS sandstone is mainly elastic deformation without obvious damage-failure characteristics. With the increase in CPS degree, AE accumulation number shows a lower trend. Especially when CPST is more than 8.0 h and CPSP exceeds 2.0 MPa, the attenuation trend decreases obviously. The response of AE accumulation number to CPST is more sensitive in the stage.

The third stage is damage accumulation stage, including crack initiation-propagation and damage accumulation precursor before fracture. For crack initiation, AE event rate increases significantly, and the number fluctuates significantly, which corresponds to the initiation, propagation and even penetration of internal cracks. With the increase in CPS degree, AE event rate decreases gradually, indicating that confined water softens sandstone and indirectly inhibits AE event generation. For the damage accumulation precursor stage before fracture, with the increase in CPS degree, the weakening effect of sandstone is strengthened. At the stage, AE event rate increases sharply with the strain inflection point of 85–95%, but the peak value gradually decreases with local abrupt changes.

The fourth stage is the fracturing instability. For natural samples, the failure presents brittleness characteristics without obvious post-peak characteristics. With the increase in CPS degree, the strain increases continuously after fracturing, but the AE accumulation number no longer increases, that is, the softening of CPS sandstone makes no crack propagation in the instability stage, and almost no AE signal occurs.

The increase in CPS degree has obvious inhibitory action on the accumulative number of AE events, which decreased in logarithmic function, as shown in Figure 14. The cumulative number of AE events of natural specimens reached the maximum with 5.4×10^6 times. In CPST, the cumulative number of AE events show piecewise attenuation characteristics: the first stage is CPST with 0–4.0 h, corresponding to the rapid attenuation stage of AE events. The number of AE events is 3.9×10^6 times at soaking 4.0 h, which is 72.2% of that of natural specimens. The second stage is 4.0–12.0 h, and the number of AE events is 3.3×10^6 times at soaking 12.0 h, which is 61.1% of that of natural specimens. After soaking 24.0 h, the cumulative number of AE events is 3.0×10^6 times, which is 55.6% of that of natural specimens. For different CPSPs, the cumulative number of AE events has similar characteristics with CPST, but on the whole, the decline rate is relatively slow. For the natural water soaking, the AE event number is 4.86×10^6 times and when CPSP is 2.0 MPa, the AE event number is 4.25×10^6 times, which is 78.7% of that of the natural specimen. When CPSP is 4.0 and 6.0 MPa, the AE event number are 70.9% and 68.9% of that of the natural specimen, respectively. Therefore, the reasonable duration of CPS weakening is 2.0–8.0 h and CPSP is 2.0–4.0 MPa.

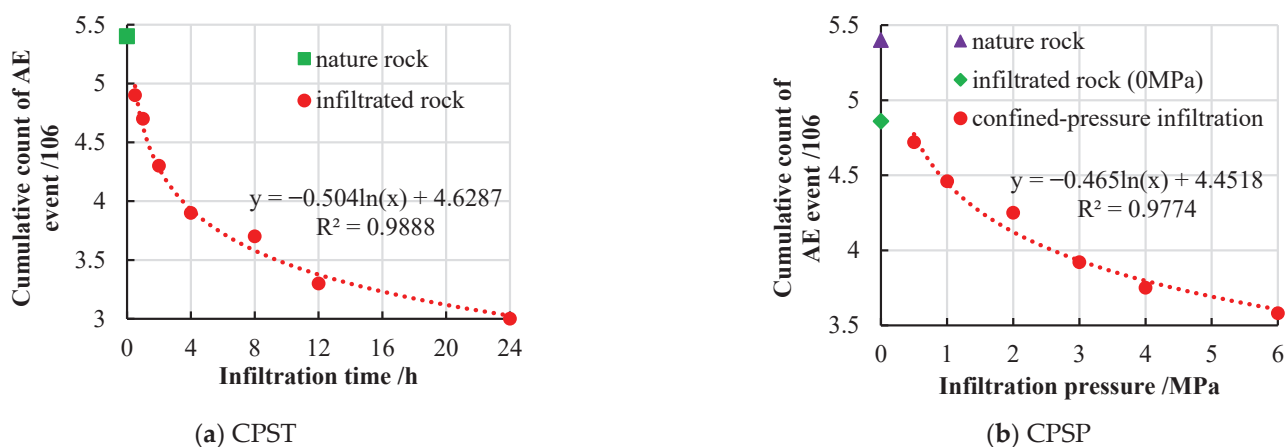


Figure 14. Variation curve of accumulative AE events number under different CPS degrees.

To sum up, the variation of AE accumulation number is consistent with the damage evolution of crack initiation and propagation in loading specimens. Due to the influence of primary fracture and natural damage to a certain extent, the AE event characteristics of sandstone specimens are still different under the same loading conditions. However, the influence trend of CPS degree on the AE event of sandstone is consistent: with the increase in CPS degree, the AE accumulation number decreases greatly, which is due to the clay and other minerals, and the confined water causes micro-structure modification. That is, the greater degree of soaking, the greater damage degree of confined water to the specimen.

According to the description of related literatures, the evolution of AE events in the whole loading process for conventional soaking samples could be divided into quiet period, stable growth, significant growth and sudden change stages [33,41], which is consistent with the change trend of CPS. The growth of moisture contents inhibits the generation of AE signals to a certain extent, and meanwhile delays AE excitation time [19,42]. Compared with conventional soaking, the suppression process of CPS on AE is strengthened, but the weakening effect does not affect AE events and energy concentrated explosion phenomenon when the sample is destroyed [31,43]. Under the CPS with long-term and high-pressure, the starting time of AE signal excitation shows a delayed growth in the increase in CPS parameters. Under high CPS parameters, AE event rate in the stable stage and significant growth stage decreases significantly, and the attenuation intensity of events accumulation is significantly higher than that of conventional soaking. That is, the suppression effect of CPS on AE is stronger than that of conventional conditions, which is one of the research features and innovations of the paper.

3.2.2. Damage Characteristics for Different CPS Degree Based on AE

AE event is the result of fracture development, and the damage increase caused by CPS will lead to the change of constitutive relationship. Therefore, the sandstone damage model under different CPS degrees can be established through damage theory [44,45].

Assuming that the natural specimen is isotropic without initial damage, if the AE events number of complete failure for the specimen section A_m is N_m , the AE event rate n of per-unit area failure is:

$$n = N_m / A_m, \quad (3)$$

The quantitative relationship between the accumulative number of AE events and CPS degree is obtained by fitting:

$$N_d = -A \cdot \ln(t, p) + B, \quad (4)$$

where, A and B are the test fitting parameters. The initial damage variable D of natural sandstone is defined as $D = 0$ and the complete failure $D = 1$. The damage variable D_{AE} related to the degree of CPS based on AE is obtained as follows:

$$D_{AE} = 1 + \frac{[A \cdot \ln(t, p) - B]}{N_m} = \frac{A}{N_m} \cdot \ln(t, p) + (1 - \frac{B}{N_m}), \quad (5)$$

Variation of the damage coefficient with strain normalization under different CPST and CPSP are shown in Figure 15.

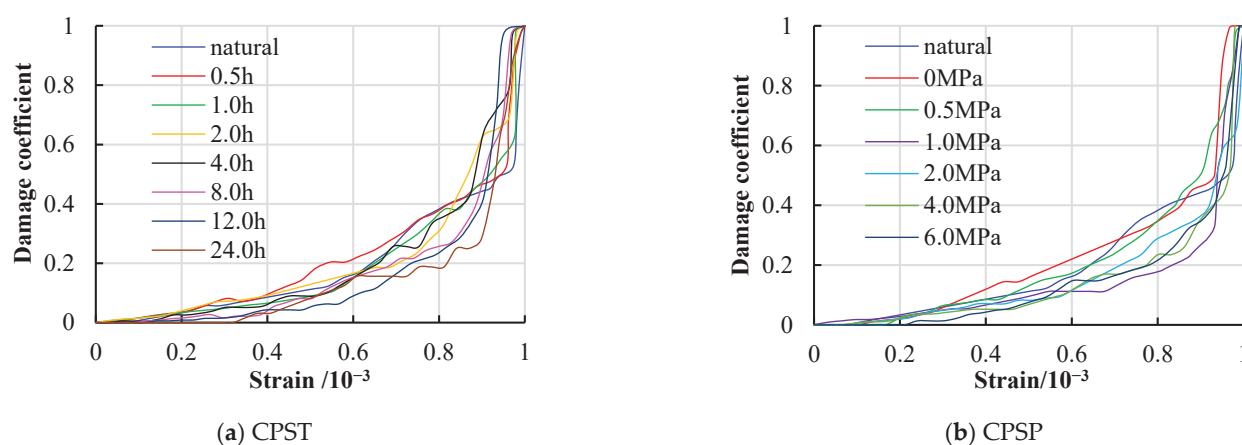


Figure 15. Variation of the damage coefficient and normalized strain of sandstone.

The damage coefficient D_{AE} correlates well with strain normalization and presents three stages, including slow, approximate linear and rapid-abrupt growth on the increasing

of the strain. The inflection point of D_{AE} growth is 25~30% and 85~90% of the failure strain, which is consistent with the strain inflection point of 85~95% reported by Zhou and Wasantha [18,46]. The rapid-abrupt growth of D_{AE} means CPS sandstone has entered the plastic deformation and failure stage. When reaching the inflection point of D_{AE} (85~90%), the range of D_{AE} values is 0.25~0.45 for the CPS sandstone samples. The inflection point in the curve corresponds to more than 90% of the peak strain, but before entering the mutation stage, the damage variable shows a local rapid growth. The stage is determined by the joint action of CPS modification and loading plastic deformation of rock samples. D_{AE} of sandstone under the uniaxial loading differs according to CPST and CPSP. Although CPS has a certain influence on the deformation and damage process of sandstone, the overall variation trend is similar; this is because the loading effect in this change is higher than the CPS modification effect.

Based on the weakening characteristics, the damage variable D_E defined by EM is established [47]:

$$D_E = 1 - \frac{E_s}{E_m}, \quad (6)$$

where E_s is the real-time EM of CPS sandstone; E_m is EM of the natural sandstone.

According to attenuation characteristics of AE event and EM for natural and different CPS degree sandstone, the fitting variation curve of damage quantity with the increase in CPS degree could be obtained:

$$\left\{ \begin{array}{l} \left\{ \begin{array}{l} D_{AE}(t) = 0.0933 \ln(t) + 0.1428 \\ D_{AE}(p) = 0.0776 \ln(p) + 0.1753 \end{array} \right. \\ \left\{ \begin{array}{l} D_E(t) = 0.0828 \ln(t) + 0.1208 \\ D_E(p) = 0.0568 \ln(p) + 0.191 \end{array} \right. \end{array} \right. , \quad (7)$$

With the increase in CPS degree, the damage value of the specimens increased in sections. In CPST, 4.0 h is the inflection point of the growth rate. With soaking 0.5~4.0 h, D increases from 0.093 to 0.278 under AE condition, and D in EM increases from 0.0573 to 0.251, which is the rapid accumulation stage of damage value. With soaking 4.0~24.0 h, D increases from 0.278 to 0.444 under AE condition and D in EM increases from 0.251 to 0.360, in which D growth rate is significantly lower than that of the former with a small increasing rate.

Under the specific CPST, the inflection point of CPSP is 2.0~3.0 MPa. In 0~3.0 MPa, D increases from 0.1 to 0.269 under the AE condition, and D in EM increases from 0.079 to 0.258, in which the growth rate of damage value is larger and more stable. Under the condition of 3.0~6.0 MPa, D increases from 0.269 to 0.311 under the AE condition, and D in EM increases from 0.258 to 0.285, in which the growth rate of damage value is obviously lower than that of the former. The above shows that with the increase in CPSP, the damage degree of sandstone increases correspondingly, but the deterioration degree is lower than that of CPST.

With the increase in CPS degree, D defined by AE is higher than that defined by EM in terms of CPST and CPSP; this is due to the AE event belongs to the crack propagation on the three-dimensional volume of the specimen, while the failure of specimens generally propagates with the single main crack at full height from the aspect of EM. However, before the failure of CPS rock mass, multiple wing cracks will occur near the fracture line (plane) which leads to the number increase in non-penetrating fracture AE events.

Therefore, based on the angle of multiple crack propagation, D defined by the AE event is more suitable to describe the deterioration of CPS rock mass.

3.2.3. Damage Evolution for Different CPS Degree Based on Energy Dissipation

During rock deformation and failure with loading, the sample energy is released and dissipated according with the law of energy conservation. The energy dissipation degree could therefore be employed for analyzing the degree of sandstone damage [48,49].

Assuming that the sample is a closed system, the total energy U varies owing to the response of the external loading, as follows:

$$U = U_E + U_D, \quad (8)$$

where, U_E and U_D are the elastic and dissipation energy, respectively.

U and U_{Ei} could be calculated with Equation (9):

$$\begin{cases} U = \sum_{i=0}^n \int_{\varepsilon_i}^{\varepsilon_{i+1}} \sigma_i d\varepsilon = \sum_{i=0}^n (\varepsilon_{i+1} - \varepsilon_i) \cdot \frac{\sigma_i + \sigma_{i+1}}{2} \\ U_{Ei} = \frac{1}{2} \sigma_{i+1} (\varepsilon_{i+1} - \varepsilon_{(i)E}) \approx \frac{\sigma_{i+1}^2}{2E_i} \end{cases}, \quad (9)$$

where ε_i is the strain corresponding stress σ_i , E_i is the elastic modulus at the elastic stage and D_{Ui} is the damage coefficient based on the dissipation energy.

Figure 16 shows the energy distribution relationship under different loading stages. The sandstone damage based on the energy dissipation could therefore be calculated:

$$D_{Ui} = \frac{U_{Di}}{U_D} = \frac{U_i - U_{Ei}}{U_D}, \quad (10)$$

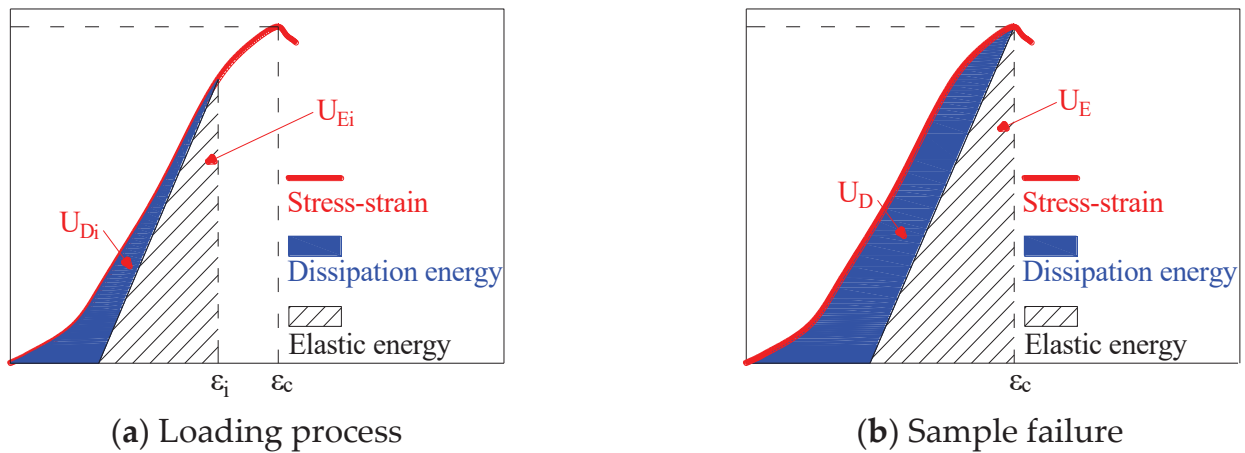


Figure 16. Energy dissipation characteristics during rock deformation.

Variation of the damage coefficient of sandstone based on dissipative energy under different CPS degrees are shown in Figures 17 and 18.

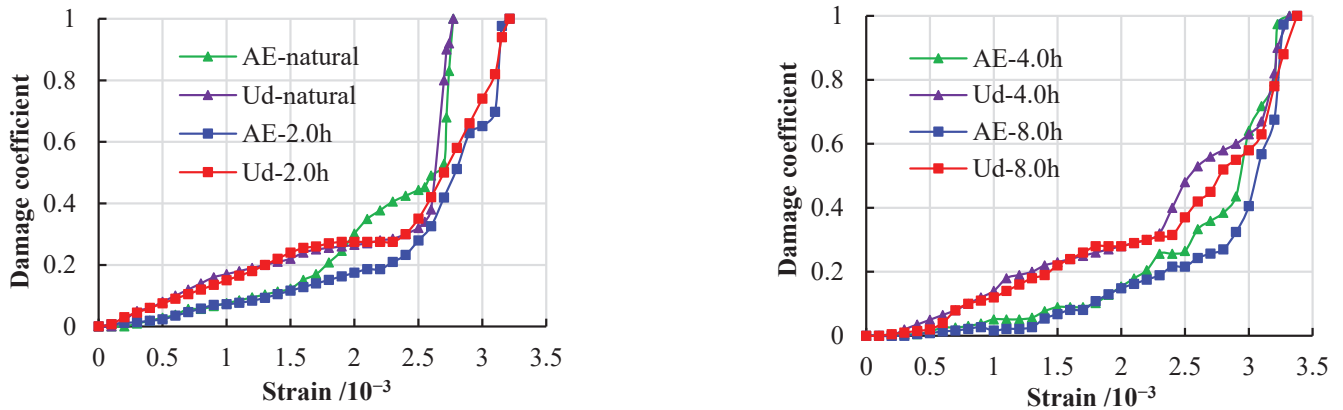


Figure 17. Damage variable of sandstone based on dissipative energy under different CPST (D_{Ud} are damage variables calculated by dissipation energy).

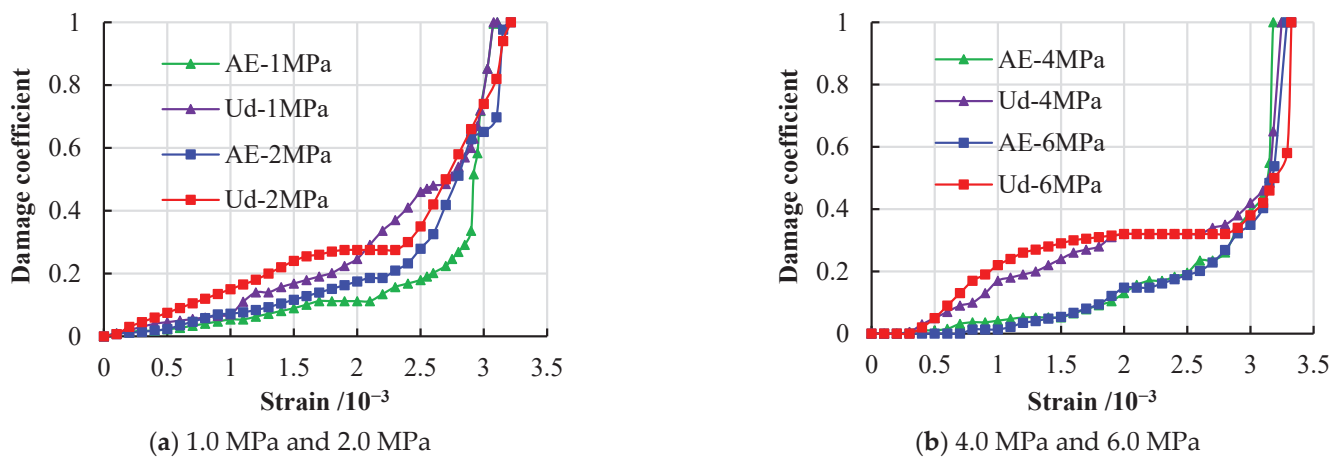


Figure 18. Damage variable of sandstone based on dissipative energy under different CPSP.

The fluctuation growth characteristics of D_{Ud} are consistent with the change trend of the stress-strain curve. In the initial stage of loading (slow and approximate linear growth stage of D_{AE}), D_{Ud} increased significantly and is larger than D_{AE} as a whole, for the dissipation energy is produced due to the closure of internal primary-microcracks accompanied by a small number of AE events. However, the damage degree (D_{Ud} and D_{AE}) develops slowly without substantial change.

During the middle of loading, the closed primary-cracks experienced secondary propagation, and the number of AE events (D_{AE}) increased linearly. However, there is a smooth transition stage of D_{Ud} change in the strain with different span intervals. The slow growth of the stress-strain curve in the initial stage and the approximate linear growth in the middle and later stages will lead to a decrease in D_{Ud} to some extent. Due to the energy dissipation with an irreversible process, the damage coefficient will not decrease in the actual loading process. The difference between D_{Ud} and D_{AE} tends to decrease with the increase in strain. When the loading deformation of the specimen reaches the inflection point of damage change, the difference and discreteness of the damage coefficients D_{Ud} and D_{AE} (dissipation energy) increased under different CPSTs.

When the strain reach 85~95% of the sandstone failure strain (Abrupt inflection-point of rock mass damage growth), the dispersion of the damage coefficients D_{Ud} and D_{AE} (dissipation energy) increases under different CPSTs and a large number of cracks are quickly initiated and propagated, which indicates that the sample is in critical instability state. The damage coefficients calculated with energy dissipation are in accordance with those based on AE. The abrupt growth of D_{Ud} and D_{AE} is an obvious transition point corresponding to 85~95% of the peak strain, which means that the failure of the sample is about to occur.

The related literature describes that the damage of rock mass in the conventional soaking state has gone through the stages of initial compaction closure, stable damage increase and damage acceleration [24,50], and proposes that the quantification of damage variables based on energy dissipation and AE is consistent. The view is consistent with the conclusion of the paper. However, the damage degree of the conventional soaking specimen increases slowly in the stable stage, but it enters the damage acceleration stage ahead of the natural state (before the peak stress), which indicates the critical transition point of damage [16,51]. As for CPS, with the increase in moisture content, the influence point of damage variable growth gradually moves forward, and the corresponding damage growth strain interval gradually expands [52], indicating that the softening effect of CPS is enhanced. For the increase in moisture content caused by CPSP, the damage variable accelerates to accumulate in the stable stage, and the final value in the stable stage is higher than that of the conventional condition. However, under a high CPSP, due to the confined-pressure water activating crack development, a certain degree of internal damage will

be generated, resulting in faster growth of dissipated energy during the loading process. Therefore, the damage variable calculated based on dissipated energy is slightly higher than that of AE. The comparative analysis reflects the unique modification weakening effect of sandstone under the condition of CPS, which is also one of the innovations of the paper.

4. Conclusions

- (1) CPS has an obvious influence on the moisture content and mechanical characteristics of sandstone: with the increase in CPS degree, UCS and EM of sandstone show logarithmic attenuation, and the sandstone varies from brittle to non-brittle failure. The main controlling factor affecting the strength of CPS sandstone is CPST.
- (2) The fracture of specimens with short CPSTs shows concentration and irregularity in the middle-upper regions. The specimens with longer CPSTs penetrate the block axially with a single main crack, showing a homogenized strength weakening effect of CPST. The fracture trajectory of low CPSP specimen is irregular, and the fracture of the specimen with high CPSP is characterized by multi-dimension and small size, which indicates the deterioration characteristics of CPSP.
- (3) The increase in CPS parameters has inhibition effect on AE events in different loading stages, which is consistent with the evolution of internal fracture initiation and propagation. In the damage accumulation stage, AE events show a step-type growth, which is the major growth stage. The cumulative number of AE events attenuates in the form of logarithmic function on the CPST and CPSP, but its response to CPST is more sensitive.
- (4) With the increase in CPS degree, the excitation time of AE event is increasing, which is consistent with the increase in internal fracturing-damage degree. However, the deterioration effect of CPST is better than that of CPSP. Based on the damage theory, the damage variable with different CPS degree based on AE events and energy dissipation of the specimens is established, which have a good consistency to quantify the damage evolution.

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