

Article

Field and Numerical Study on Deformation and Failure Characteristics of Deep High-Stress Main Roadway in Dongpang Coal Mine

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Abstract: Deep horizontal high stress and high permeability geological factors appear when coal mines are converted to deep horizontal mining. When the roadway is damaged by the mining face, and the supporting components are mismatched, the deep roadways necessitate extensive repair work, which has a negative impact on the coal mining economy and sustainability. This paper carried out a series of field tests on the roadways deformation, crack distribution, and loose rock zone of the deep roadways. Furthermore, a numerical calculation model was established using the discrete element method (DEM) and calibrated with laboratory tests and RQD methods. Both the stress and crack distribution in the surrounding rock of the deep roadway were simulated. The field test and the corrected numerical model showed consistency. A FISH function was used to document the propagation of shear and tensile cracks around the roadway in three periods, and a damage parameter was adopted to evaluate the failure mechanism of the deep roadways under the dynamic stress disturbance. The matching of specifications of anchor cables, rock bolts, and anchoring agent is the primary point in the control of deep roadways, and revealing the stress evolution, crack propagation, and damage distribution caused by mining effects is another key point in deep roadway controlling. The field test and DEM in this paper provide a reference for the design of surrounding rock control of deep roadways and the sustainable development of coal mines.

Keywords: deep mine; main roadway; deformation and failure characteristics; crack and damage analysis; filed and numerical simulation study

1. Introduction

With the progressive increase in the intensity of coal resources, shallow coal resources are becoming rare, and most of the mines are mined in deep areas in order to improve the sustainability of the coal industry [1–5]. The surrounding rock is shattered and loose, with extensive damage and cracked joints, as a result of numerous variables such as geo-stress, dynamic pressure, and geological structure [6]. Fractured roadway excavation, the complicated stress environment with asymmetrical distribution, and an evident increase in in-situ ground stress would produce strength deterioration and secondary stress concentration distribution on the surrounding rock [7]. Deep roadways are plagued by a variety of issues, such as asymmetrically high ground stress and various lithologies, making it difficult to pinpoint the sources of structural instability and adopt effective management techniques [8–10]. Figure 1 depicts geological data from asymmetrical structural fractures along roadways, including sandstone and mudstone [11].

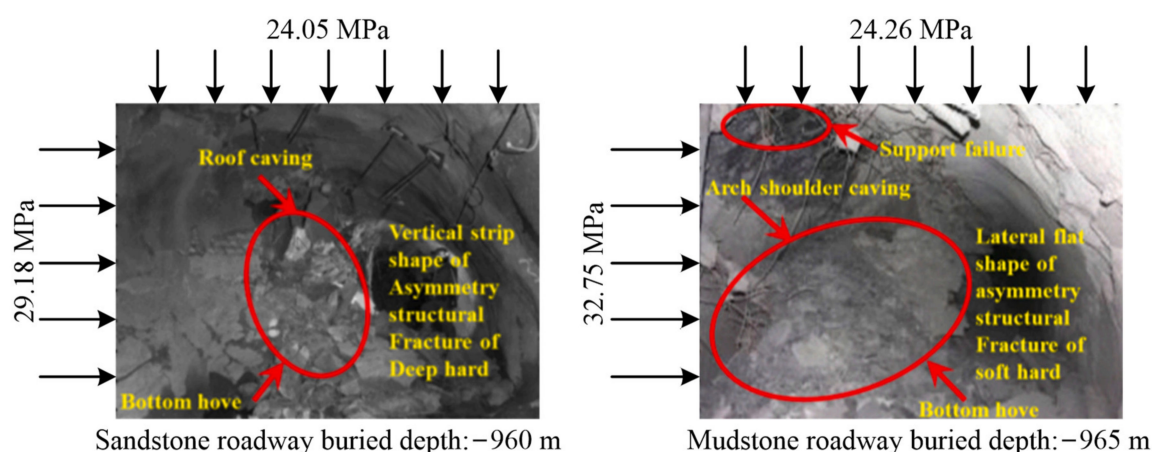


Figure 1. The in-site deformation photograph of deep roadways [11].

Scholars conducted extensive research on the deformation and failure mechanisms of soft rock roadways, as well as proposed and built roadway support technologies [3,5,6,12]. Yu et al. [2] systematically studied the stability of deep-buried rocks through field investigation, laboratory analysis, theoretical derivation, and engineering applications. He et al. [3] proposed that core scientific issues arising in deep underground projects are encountered with the conditions of “three-high and one-disturbance”, i.e., high in-situ stress, high temperature, high seepage pressure, and a strong mining disturbance, which form a complex geomechanical environment for deep engineering. Li et al. [5] carried out a large-scale geomechanical model test to explore the surrounding rock deformation and failure mechanisms of such deep roadways. Yang et al. [12] showed that shallow rock has a significant scale of tensile failure, which causes swelling and fracture surrounding the roadway. Significant floor heaving, side shrinkage, and roof sinking occur as a result of the primary support being weak and when there is no support on the floor. To support the ventilation roadway, a novel “bolt-cable-mesh-shotcrete + shell” combination support is suggested. Wang et al. [13] demonstrated that roadway deformation is extensive with a wide damage range. The anchor bolts are frequently found in the severely fractured surrounding rock, the support potential is not utilized, the arch’s support strength is insufficient, and the post-bearing capacity is low; all of these mainly lead to the failure of the roadway bearing capacity and a concept of “high-strength, integrity, and pressure-relief” is proposed. Common supporting methods for shallow roadways are unlikely to extend to deep geological settings; thus, Kang et al. [14] are looking at developing a new type of combined supporting system for weak floors in difficult geological environments. Li et al. [15] provided a case study of the deformation failure mechanism and support technology for a deep roadway with soft rock mass and evaluates the modes, influencing factors, laws, and processes of deformation failure in the roadway based on comprehensive field research and numerical model analysis. Zhao et al. [16] presented a technique that involves analyzing the failure features of roadways, as well as the microscopic fracture properties of the surrounding rock, using a digital drilling televiewer and three-dimensional laser scanning devices. Zuo et al. [17] systematically investigated the Macro/Meso dynamic behavior of deep rock or coal-rock combined institutions under various loading conditions and developed a coupled grouting control technology for the surrounding rock, standard strength assistance in the deep roadway, and the corresponding velocity vector movement model of overlying strata. Shi [18] proposed a systematic model of deformable block systems that represents an important solution for large displacement, large deformation, and failure computational methods, assuming that the forces acting on each block, whether from applied load or interaction with other blocks, assuage the equilibrium conditions. Chen et al. [19] used insight into mechanics to evaluate the fracture and develop principles of floor mining fractures to reveal the coupling relationship between certain mining-induced fractures and

theoretical stress and hydraulic pressure, and defined a model of fracture progression and connection structure for the deep floor discoloration based on a self-developed simulation test, especially for high floor water transients.

The DEM represents rock masses as a collection of blocks that can be stiff or deformable, and an explicit solution technique is used [20]. The blocks are permitted to behave as if they were continuum media, and the block and joint interactions are represented using Newton's equations of motion. Unlike the finite difference method, this avoids the requirement for a huge stiffness matrix. Fairhurst et al. [21] understood the advantages of the distinct element method for modeling non-continuous rock masses by comparative analysis of an excavation in a jointed rock mass developed by a finite difference method and a universal distinct element code model. Bai et al. [22] utilized the discrete element method to simulate the failure of a laminated roof, focusing on the formation and stabilization of micro cracks and macroscopic cracks, including the growth of control mechanism stress and deformation in the laminated roof. Hamdi et al. [23] utilized a mixed finite-discrete element method to model the entire 3D cracking procedure during traditional laboratory testing, including Brazilian tension and uniaxial compression strength. Srisharan et al. [24] presented stability studies on two tunnels, a horseshoe-shaped and an inverted arch-shaped tunnel, in a deep coal mine in China using the DEM simulation model. The calibrated models were analyzed for different supported and unsupported cases to estimate the significance and adequacy of the current supports being used in the mine and to suggest possible optimization. Zang et al. [8] presented a case study on the deformation failure behavior and support design of a deep roadway in the Tangyang mine by field tests and DEM simulations. Finally, a DEM simulation and a field experiment were conducted to evaluate the rationality of the proposed support scheme, and the results showed that the new support method could effectively control the surrounding rock. In a study by Li et al. [25], DEM was carried out to further explore the deformation failure characteristics and factors influencing deep roadways with different engineering geological conditions.

Dongpang Mine (shown in Figure 2a) in Xingtai City, Hebei Province, is currently mining at a level of +480. The #2 coal seam has a ground burial depth of approximately 580 m. The coal and rock masses are impacted by high ground stress, high ground temperatures, and high permeability after deep mining. The mine track roadway in the 11 mining area (TR-11MA) was severely distorted throughout the service era and has undergone numerous repairs, but the control effect is not discernible. A large amount of the previous literature on the mechanism of deformation and failure of deep roadways and the research methods of control technology mainly focuses on in-site testing, theoretical models, and numerical simulations [1,19,26]. However, few studies have focused on matching support strength and support structure (rock bolts and anchor cables axial force, length of anchorage agent, bolt pre-stress, and thickness of pallets). Furthermore, few systematic investigations on the failure characteristics of the surrounding rock have been conducted (thickness of the loose rock zone and fracture distribution characteristics of the surrounding rock), which is the motivation of this paper. Combining the above-mentioned site's geological conditions, the method of field and numerical simulation is used to systematically study the failure characteristics of deep high-stress, analyze the stress field, crack field, and damage change characteristics of the deep high-stress main roadways affected by mining disturbances. Finally, corresponding control technologies were proposed for the repair and extension section of the main roadways, which reduces roadways repair times, improves the sustainability of development, and serves as a model for deformation control of similar deep high-stress roadways.

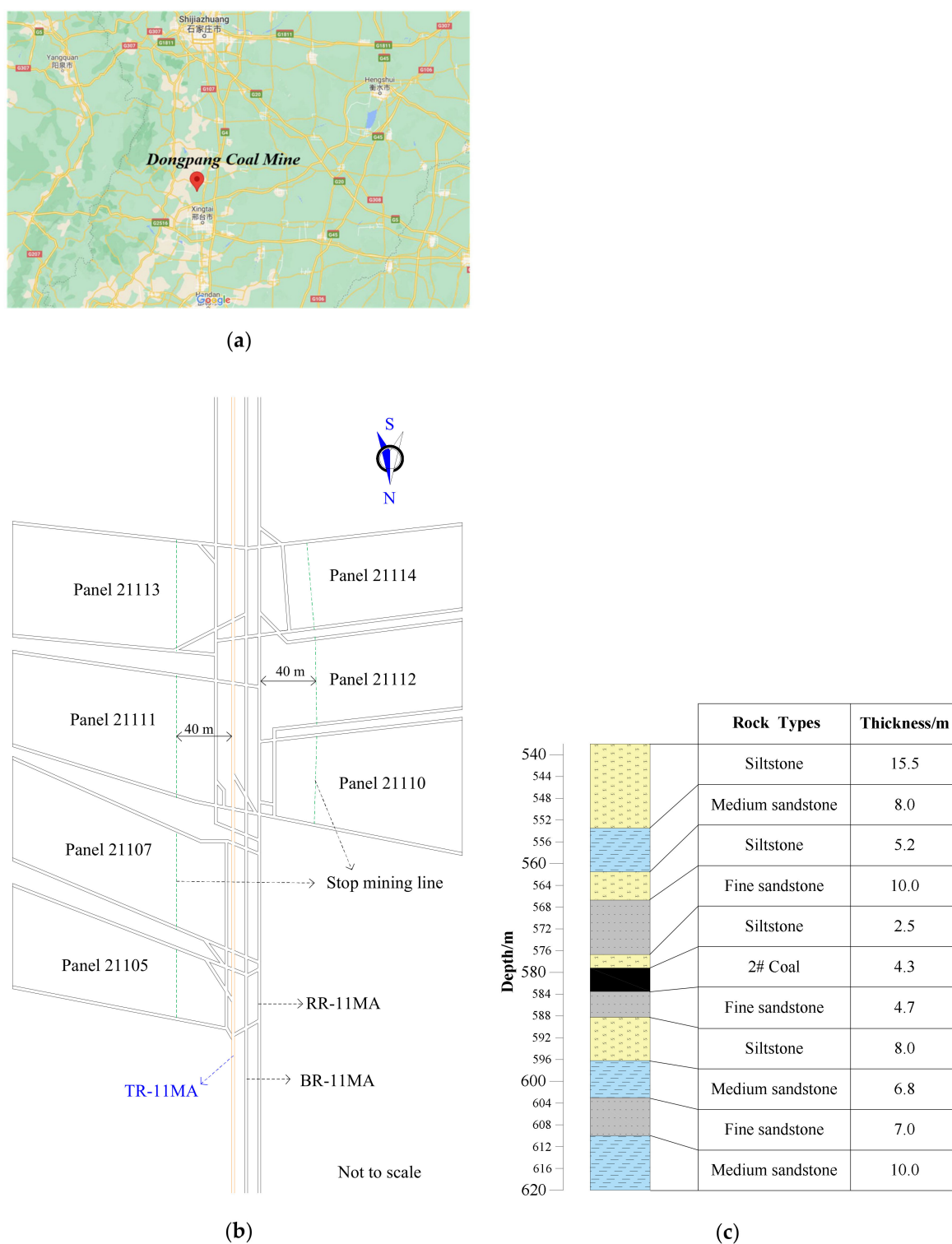


Figure 2. Geological Conditions of Dongpang Coal Mine. (a) Study site location; (b) Panel map and main road layout of the 11 # mining area at Dongpang coal mine; (c) Columnar diagram of rock layers, including rock types, sequence, and buried depth.

Therefore, the main objectives of this paper are:

- Geological conditions and field study on the deformation and failure characteristics of the deep high-stress main roadways;
- Numerical simulation study on deformation and failure mechanism of deep main roadways;

- To develop a basic control strategy of the reinforcement and repair technology of the deep main roadway in the Dongpang coal mine.

2. Geological Conditions of the Deep High-Stress Main Roadways

2.1. Geological Conditions of the Main Roadways

The main mining area of Dongpang Mine is the 11 mining area (11MA). This mining area is a two-wing arrangement. There are three main roadways in the 11MA. The TR-11MA is laid out in the seam floor rock layer. The main return air roadway of the 11MA (RR-11MA) and the main belt transportation roadway of the 11MA (BR-11MA) run along the coal seam's roof. The horizontal offset between the three main roadways is 25 m. The cross-section dimensions of the three main roadways are 5.2 m and 4.2 m in width and 4.2 m, respectively. Full-height mining at one time is used on the working faces of the 11MA. The eastern wing working panels that have been mined are P21113, P21111, and P21107, and the western wing working panels that have been mined are P21114, P21112 and P21110 (shown in Figure 2b). The above-mentioned mining panel stop line is 40 m from the main roadways. P21105 in the east wing is the mining working panel. As shown in Figure 2c, the immediate roof of 11MA is siltstone, while the main roof is fine sandstone. The upper rock layers are siltstone, medium sandstone, and siltstone, in sequence. The immediate floor is fine sandstone, the main floor is siltstone, and the lower rock layers are medium sandstone, fine sandstone, and medium sandstone, in sequence.

Most areas of the TR-11MA showed the phenomenon of two ribs approaching, the roof sinking, the bottom pallet bulging, and accompanied by the failure of the supporting structures in some regions, such as the breakage of the anchor rod and the cable, the pallet being bent, and so on, from the excavation period of the TR-11MA for the above-mentioned mining process.

2.2. Field Study on Deformation and Failure Characteristics of the TR-11MA

2.2.1. Support Conditions of the TR-11MA

The TR-11MA is supported by the combined support of rock bolt and anchor cable, as shown in Figure 3. The detailed support parameters are the following: In terms of rock bolt support, the specification is a round steel bolt with a diameter of 20 mm and a length of 2200 mm. The rod and row spacing of the rock bolt is 900 mm and 900 mm, respectively. The pre-tightening force and the anchoring of the rock bolt are 150 Nm and 80 kN, respectively. The anchoring agent is 60 cm in length, and the size of the rock bolt pallet is 130 mm × 130 mm × 10 mm. In terms of anchor cable support, a steel-stranded anchor cable with a diameter of 17.8 mm and a length of 6300 mm was adopted. The anchor cable spacing is 2250 mm/1800 mm, the roof anchor cable row distance is 900 mm, and the "313" supporting form is adopted. The two ribs anchor cable row distance is 1800 mm, the anchor cable design anchoring force is 200 kN, and the anchoring agent is 180 cm in length.

The TR-11MA deformation and failure process can be defined as three periods: Period 1—the main roadways excavation. Period 2—the east panels retreat; the initial stress conditions are various, changing significantly in this period. Period 3—the west panels retreat; the stress has a more complicated distribution. In a word, the stress environment of the TR-11MA is quite different in the three periods. In order to obtain insight into the deformation and failure of TR-11MA in the above three periods, two aspects of monitoring strategies were applied in the roadway. In-site monitoring equipment, methods, monitoring steps, and monitoring results are listed as follow:

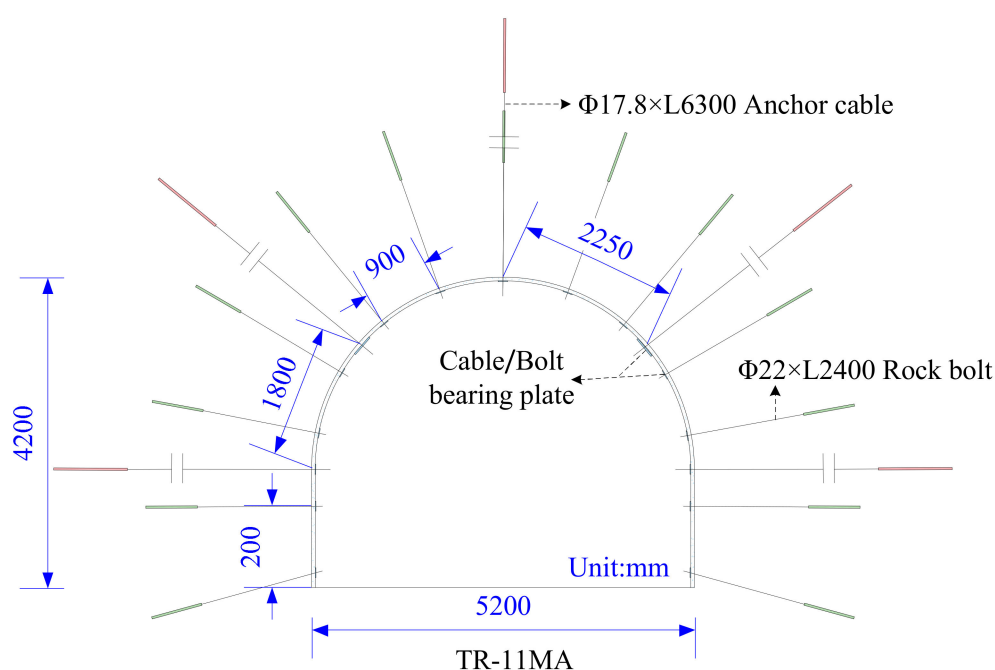


Figure 3. Main section diagram of rock bolt and anchor cable support parameters for TR-11MA.

2.2.2. Deformation of the TR-11MA at the Three Periods

Deformation measuring station layout: ten deformation monitoring stations were arranged during the process of TR-11MA excavation. The distance between the stations was 50 m, and each station had four permanent points fixed on the roof, floor, and side walls of the TR-11MA, respectively. (The eastern rib and western rib are terms used to distinguish the asymmetric deformation characteristics of the left and right ribs in three periods.).

Monitoring and measurement method: the cross method was used to measure the deformation of the roof, floor, and side walls (eastern and western) at the stations. A portable telescopic rod and measuring lines were used to measure the roof and the floor deformation. Both the western rib and the eastern rib deformation were measured using flexible tape and measuring lines. **Measurement frequency:** Period 1 was monitored once a day at the beginning of the excavation of the TR-11MA and once a week after the deformation was stable. Periods 2 and 3 are the most severely impacted by mining disturbances. Observations were performed once a day, and the dynamic horizontal distance between the eastern or western rib working face and the TR-11MA was recorded.

Figure 4 shows the deformation of the TR-11MA in the three periods. The eastern and western ribs of the roadway have relatively large deformations after the roadway is excavated (defined as period 1). The deformations of the eastern rib, western ribs, roof, and floor are 333 mm, 324 mm, 285 mm, and 164 mm, respectively. During the mining of the eastern panels (defined as period 2), the deformation of the roof and the two ribs of the roadway increased significantly. The deformation of the eastern rib, western rib, roof, and the floor increased by 50 mm, 74 mm, 75 mm, and 40 mm, respectively. The deformation of the TR-11MA is shown by the asymmetry of the western and eastern ribs. During the mining of the western panels (defined as period 3), the deformation of the TR-11MA was not obvious. The deformation of the eastern rib, western rib, roof, and the floor increased by 20 mm, 20 mm, 15 mm, and 15 mm, respectively. Therefore, after the excavation of the main roadway in the 11th mining area was completed, the stress disturbance caused by the excavation of the BR-11MA and RR-11MA is also one of the reasons for the increase in the amount of deformation during the roadway excavation. The mining of the eastern panels caused the stress environment of the TR-11MA to change, which is the main reason for the deformation of the TR-11MA. In addition, western panel mining has little effect on the TR-11MA stress environment distribution and surrounding rock deformation.

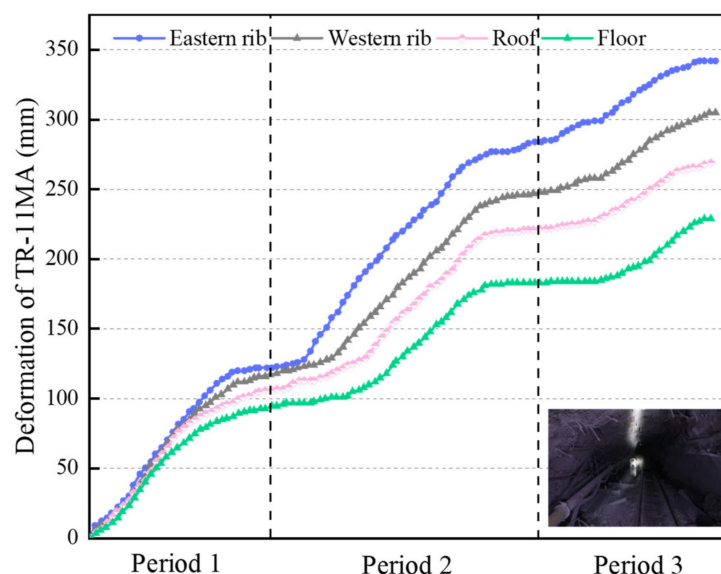


Figure 4. Deformation of the TR-11MA in the three periods.

2.2.3. Damage and Cracks of the Surrounding Rock

(i) Borehole camera exploration

In order to more reliably understand the deformation and fragmentation of the surrounding rock in the deep part of the roadway, in-situ tests were carried out on the roof and rib of the TR-11MA with a borehole spying instrument, mainly to detect the lithological characteristics of the surrounding rock and the characteristics of the development of cracks.

The TS-C1201 drilling multi-function imaging analyzer (as shown in Figure 5) mainly includes main components such as the main unit, probe, depth-sounding pulley, and so on. The depth detection pulley is used to record the depth of the probe in the borehole; there is no nickel-metal hydride battery pack in the passive probe, and the power supply is supplied by the host. The probe has a built-in LED white light-emitting diode (with a brightness adjustment circuit) and a camera to capture the whole wall image. The video signal, control signal, and digital compass signal in the probe are transmitted to the host through the cable. The host receives the probe signal, and the depth pulse signal of the depth-sounding pulley calculates the depth position of the probe and performs image recording and matching splicing on the video signal. Video and image matching and splicing can be carried out simultaneously. As the probe continues to move into the hole, the entire hole wall is automatically matched and spliced into a complete flat unfolded picture.



Figure 5. TS-C1201 borehole multi-function imaging analyzer.

Sketches of borehole surveys at different locations are shown in Figure 6. In the shallow region of the rib walls of boreholes, there are additional crossing cracks within the range of 0~2 m in the stable region of TR-11MA excavation deformation (period 1). At the same time, there are several separate horizontal and longitudinal fractures in the 2~4 m

range. The roof drilling findings reveal that the cracks interpenetrate at a depth of 0~2 m, with a few transverse cracks and minor fractures at a depth of 2~5 m. Cracks and fractures occurred in the shallow 0~2 m of the rib walls of boreholes at the time when mining in the eastern panels was seriously affected (period 2), and intersecting cracks appeared in the range of 2~4 m. In the eastern rib area, the crack growth degree includes both horizontal and vertical cracks. It is a lot more serious than in the western ribs regions. Broken and penetrating cracks may be seen in the range of 0~3 m on the roof, broken regions and water-conducting cracks in the range of 3~5 m, and minor transverse and vertical cracks in the range of 5~8 m. The two ribs of the drilling crack drawings indicated a small increase in crack propagation during the time when the TR-11MA was badly damaged by western panel mining (period 3), although the gain was not apparent.

(ii) Thickness range of surrounding loose rock zone

The ZBL-U510 ultrasonic detector is used to test the loose zone of the rock mass. By testing the acoustic parameters of the broken rock mass, the mechanical properties of the broken rock mass can be analyzed, and the plastic failure of the surrounding rock of the roadway can be judged. The loose zone test adopts a two-hole test sensor with one transmitter and one receiver, and the distance between the transmitter and the receiver is 0.4~0.5 m. After the test hole is drilled, the test should be carried out in time. Before the test, the test hole should be flushed with pressure water, and the coal and rock powder in the test hole should be washed out. The ZBL-U510 ultrasonic testing instrument is used in the acoustic method. Through the acoustic parameters tests of broken rock mass, the failure situation of surrounding rock is analyzed [27].

Three test sections are selected for the loose rock zone test, and a total of three test stations are arranged, among which are located on the two ribs and roof of TR-11MA. The first detection station is in the stable deformation area of TR-11MA. The second and third detection stations were in the most severely deformed regions of the TR-11MA during the period of the eastern and western panel mining, respectively. Each measuring point was drilled at the two waistline positions of the TR-11MA. The borehole direction was required to be perpendicular to the TR-11MA. The drill bit had a 42 mm diameter and a hole depth of 6 m.

Figure 7 shows the thickness range results of the surrounding loose rock zone. According to the previous field test results, the loose rock zone selected the eastern rib as the test object. Station 1 is the result of the loose rock zone of the eastern ribs during excavation (period 1). The thickness of the loose rock zone has a range of 1.7 m~2.6 m; Station 2 is the loose rock zone during the eastern panel mining (period 2). As a result of the circle, the range of the thickness of the loose rock zone is between 2.5 m and 4.2 m; Station 3 is the result of the thickness of the loose rock zone during the stopping period of the panel in the western area (period 3), and the range of the loose rock zone is between 3.0 m and 4.5 m. It can be seen that during the mining process of the working face in the eastern area, the thickness of the loose rock zone of the eastern area increased significantly, and the surrounding rock fragmentation was severely damaged, which may be caused by the unreasonable support parameters of rock bolts and anchor cable [28,29].

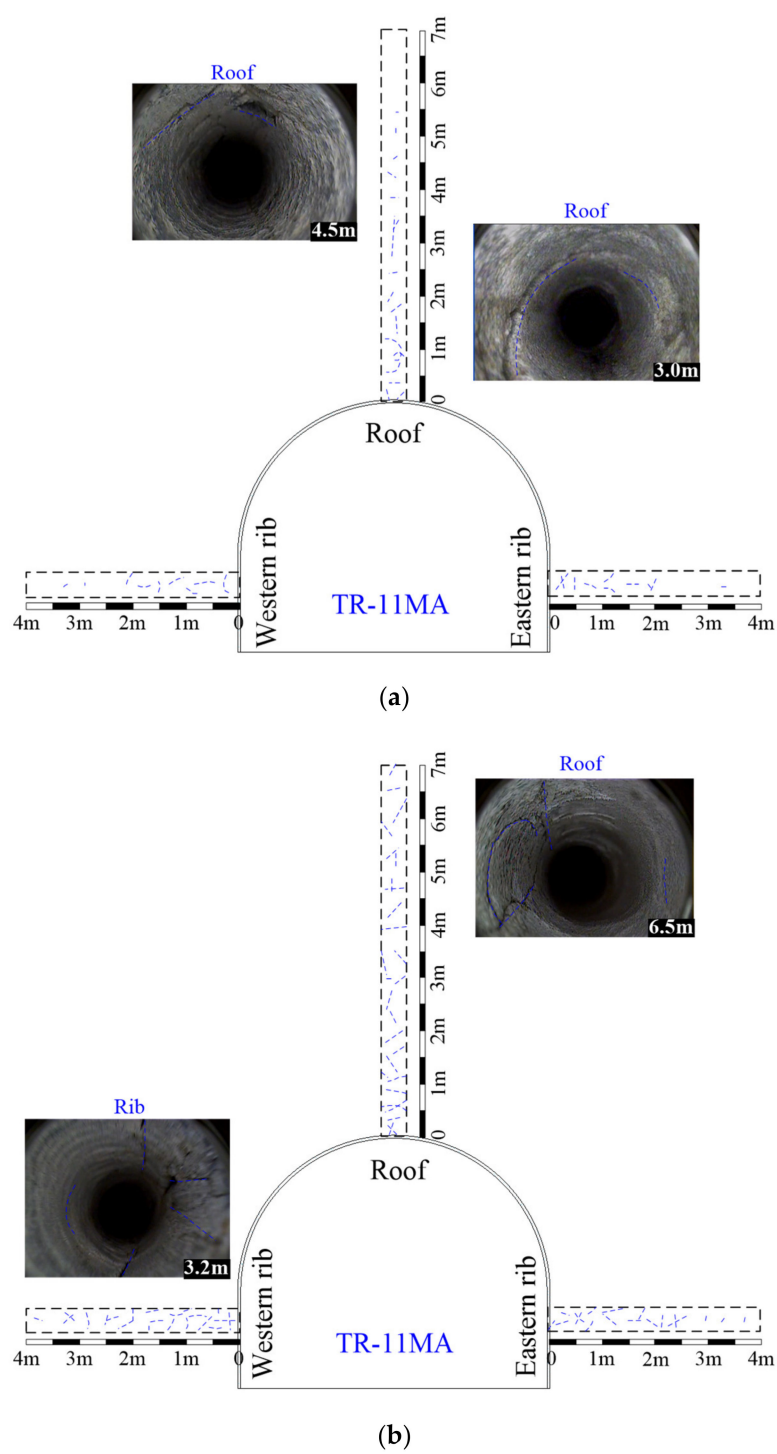


Figure 6. Cont.

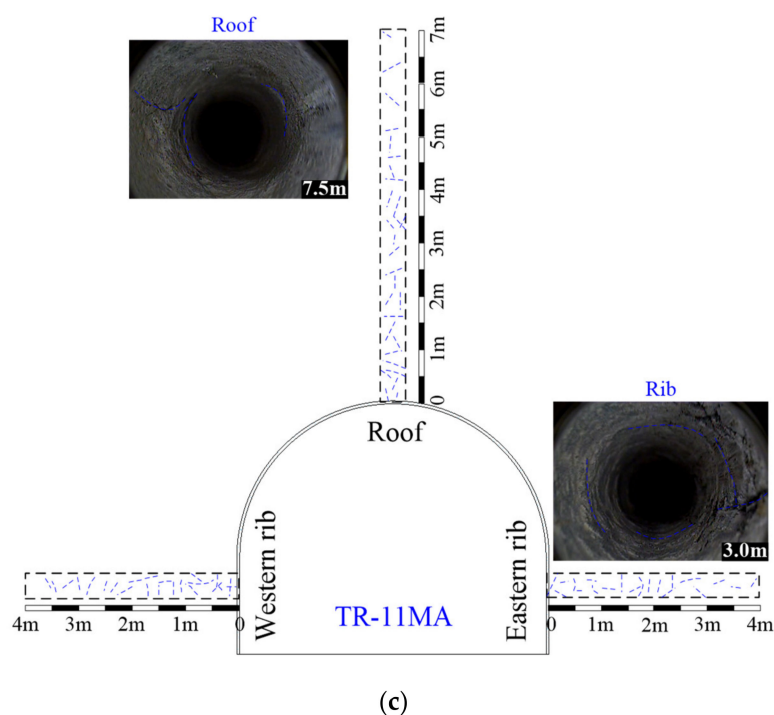


Figure 6. A clear view of the TR-11MA drilling results at three time periods. (a) Period 1; (b) Period 2; (c) Period 3.

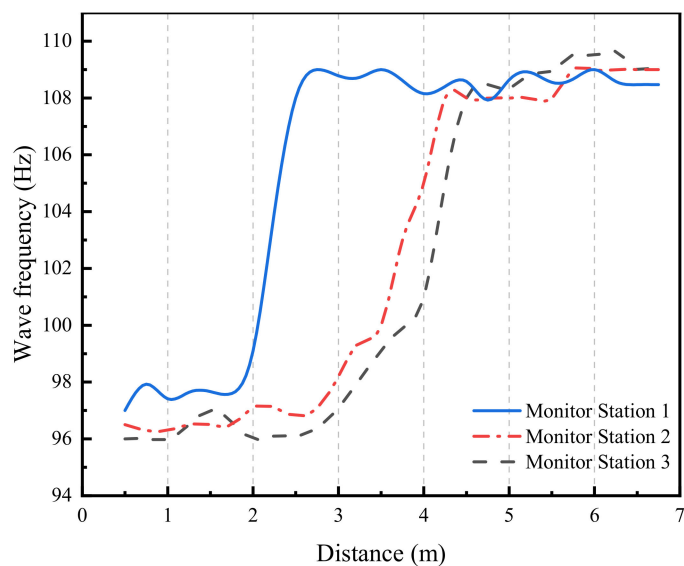


Figure 7. Thickness range of surrounding loose rock zone.

2.3. Matching Test of Existing Support Structure in TR-11MA

Test purposes: (i) Study the mutual matching mechanism of the rock bolt body, anchoring agent, nut, and pallet; (ii) The overall inspection of the rock bolt and its supporting components so as to determine the weak link of the bolt system.

Test method and procedure: (1) Use the M-II anchor rod installation device to anchor the anchor rod and the special steel pipe with an inner diameter of 32 mm or 28 mm with an anchoring agent; (2) Turn on the LW-1000 horizontal tensile testing machine; (3) Enter the sample information; (4) Install the sample, as shown in Figure 8; (5) Start the test; (6) End the test and save the data.

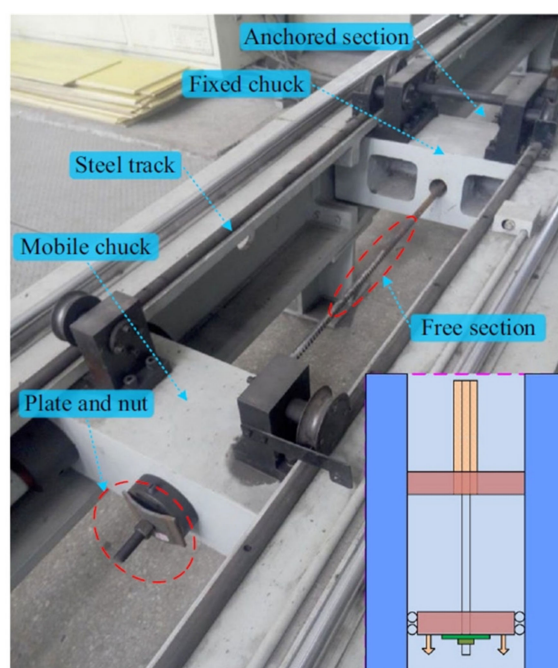


Figure 8. Pull-out test on the rock bolt performance test platform [30].

Figure 9 shows the breaking mechanical characteristic curves of different types of anchor cables. It can be seen in Figure 9 as the following:

- (1) The force process of the anchor cable is shown as a smooth curve rise. After rising to a certain height, the force remains unchanged, and the anchor cable continues to deform. When the deformation reaches a certain value, the steel strand gradually breaks, and the force drops sharply. The breaking load of the $\Phi 17.8$ mm anchor cable is about 370 kN; when the deformation of the $\Phi 21.8$ mm anchor cable reaches a certain value, the steel strand gradually breaks, and the unbroken steel strand continues to bear greater tensile force, and then is broken. After the wire is completely broken, the force of the anchor cable drops sharply. The breaking load of the $\Phi 21.6$ mm anchor cable is about 530 kN;
- (2) The force of the rock bolt first rises linearly, then slowly rises, gradually reaches its yield strength, and finally breaks, and the force drops sharply. The breaking load of the $\Phi 20 \times 2200$ mm rebar rock bolt is 125 kN, and the breaking load of the $\Phi 22 \times 2200$ mm rebar rock bolt is about 200 kN.

Figure 10 shows the relationship between the length of the anchoring agent and the change of anchoring force. When the length of the anchoring agent is 40 cm, the peak value of the anchoring force is only about 85 kN, which is much smaller than the breaking load of ordinary rebar bolts and does not provide a good anchoring effect, and the anchoring force decays quickly over time, reaching the peak value when the anchor is quickly released. When the length of the anchoring agent is 60 cm, the anchoring force reaches about 140 kN after the anchor enters the yield stage, and the anchoring force can maintain the anchoring force for a period of time before reaching its peak strength of 170 kN, indicating that the anchoring effect is relatively good, however after reaching the peak value, the anchoring force quickly drops to 0 kN. When the length of the anchoring agent is 80 cm, the peak value of the anchoring force is close to 200 kN, which is higher than the breaking load of different threaded steel bolts and is equivalent to the axial force of $\Phi 22$ mm threaded steel rock bolt, and the anchoring force decays very slowly over time, staying above 180 kN for a long time. Thus, the anchoring effect is better.

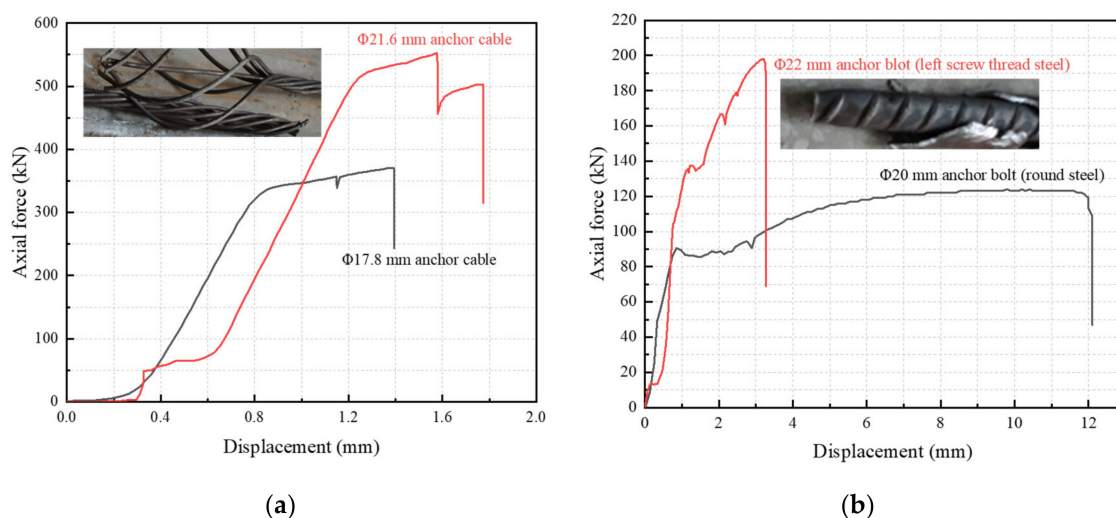


Figure 9. The breaking mechanical characteristic curves. (a) $\Phi 18.9$ mm anchor cable and $\Phi 21.6$ mm anchor cable. (b) $\Phi 20$ mm round steel rock bolt and $\Phi 22$ mm rebar rock bolt.

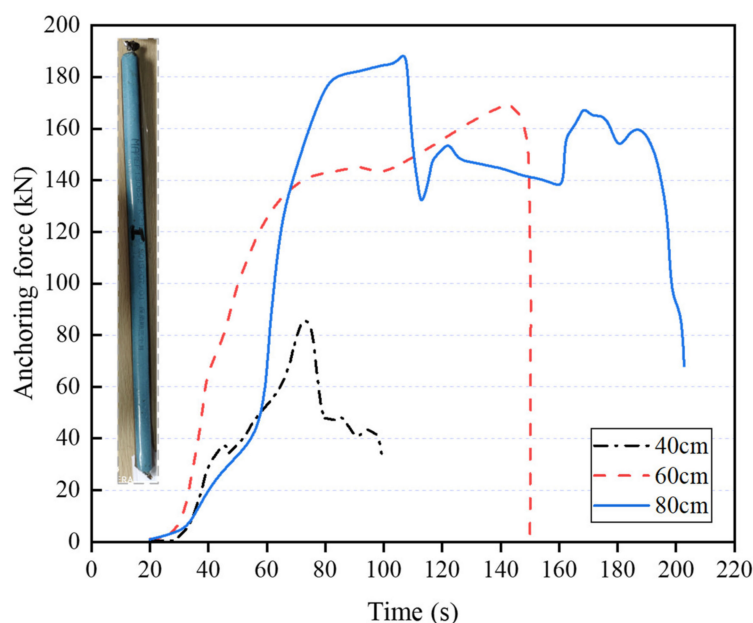


Figure 10. The relationship between anchoring agent length and anchoring force.

3. Numerical Simulation of Deformation and Failure Mechanism of TR-11MA

3.1. Numerical Model Set Up

In order to reveal the crack propagation process and the damage evolution of the roadway in the deep coal mine, a DEM model was created [31]. The width and height of the 2D model are 265 m and 82 m, respectively, as illustrated in Figure 11. To improve efficiency in the calculation, these triangular blocks had an average edge length of 0.2 m in the interesting region. The model took into consideration the 100 m wide eastern and western panel, respectively, in order to simulate the extraction of the two wing coal panels and provide accurate mining-induced stress on the surrounding rock of the roadway. The bottom and lateral boundaries were fixed in vertical and horizontal displacement, respectively. The model was run using an in-situ stress condition of $\sigma_V = 14.60$ MPa and $\sigma_H = 17.52$ MPa [32]. Overburden pressure was simulated by applying vertical stress of 14.60 MPa to the boundary.

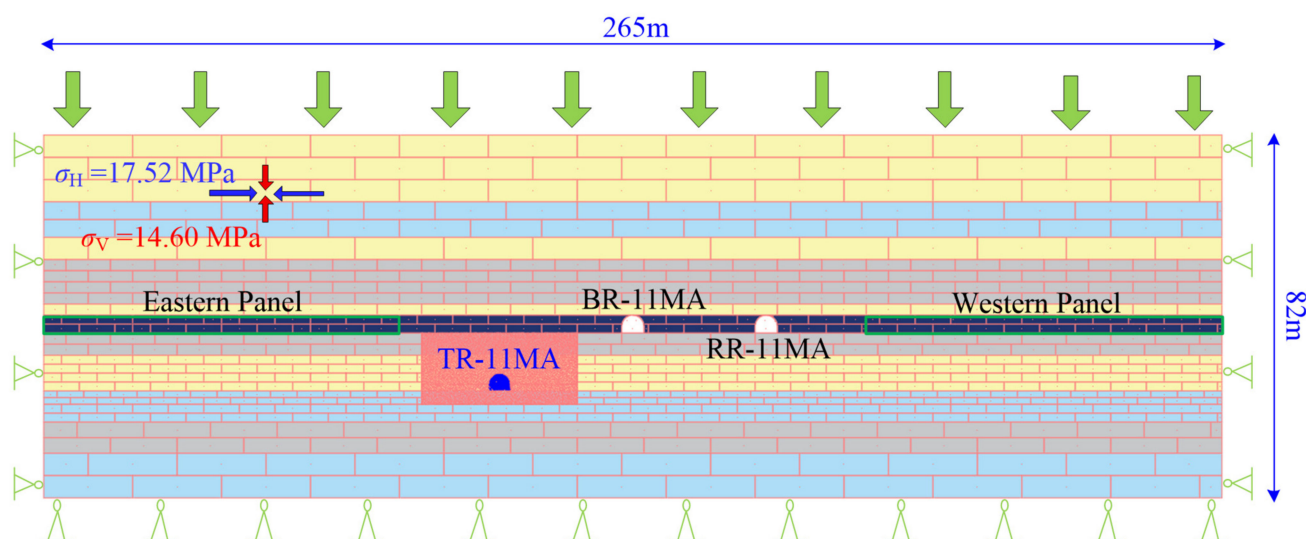


Figure 11. Numerical model with boundary and initial stress conditions.

3.2. Input Model Parameters and Simulation Planning Parameters Calibration Process

The mechanical behavior of the contacts is governed by micro-properties such as normal and shear stiffness, tensile strength, cohesion, and frictional coefficient. Calibration of these micro-properties to rock mass characteristics is frequently required [33,34]. To determine the rock mass micro-properties in the model, the calibration technique described below is used.

First, the laboratory unconfined compression tests on standard specimens and field measurements were used to establish the rock mass's uniaxial compressive strength (UCS), deformation modulus. The Trigon logic was used to create a calibration model. The calibration model used the same triangular block size as the field-scale model to eliminate the influence of block size on the outcome.

Table 1 shows the intact characteristics derived from compression tests on standard specimens. These factors, on the other hand, are unable to predict the inherent deformability of the rock mass. RQD is still extensively used to determine the rock mass deformation modulus, which is typically easier to obtain than RMR or Q . Zhang [35] derived a relationship, Equation (1), between RQD and the modulus ratio E_m/E_r , based on a large amount of field monitoring data. E_m and E_r represent the deformation modulus of the rock mass and intact rock, respectively. As a result, the rock mass parameters may be calibrated using this proposed relationship. The rock mass strength can be calculated by Equation (2)

$$E_m/E_r = 10^{0.0186RQD-1.91} \quad (1)$$

$$\sigma_m/\sigma_r = 10^{0.013RQD-1.34} \quad (2)$$

where E_m , σ_m and E_r , σ_r represent the deformation modulus (GPa) and the UCS (MPa) of the rock mass and the intact rock, respectively.

Table 1. Intact rock properties and rock mass properties in the numerical model.

Rock Strata	Intact Rock		RQD	Rock Mass	
	E_r (GPa)	σ_r (MPa)		E_m (GPa)	σ_m (MPa)
Siltstone	39.82	49.01	88	21.23	31.21
Coal	7.92	38.87	78	2.75	18.35
Medium sandstone	16.02	43.29	85	7.51	25.2
Fine sandstone	10.76	35.94	90	6.25	24.3

Second, the deformation modulus should be calibrated by making the deformation modulus of the blocks equal to the deformation modulus of the rock mass. K_n , the normal stiffness, is derived using Equation (3). After that, the Poisson's ratio should be calibrated by altering K_s/K_n .

$$K_n = n \left[\frac{K + 4G/3}{\Delta Z_{\min}} \right] \quad (3)$$

As shown in Figure 12a, the Trigon logic calibration models were developed for modeling the UCS with dimensions of 2.5 m wide, 5 m height. Because the models are made up of four different types of lithology, four different contact types must be calibrated. Equation (3) was used to compute the normal and shear stiffness of contacts, K_n and K_s , where K and G are the bulk and shear modulus (GPa/m) of the blocks, respectively, $\Delta Z_{\min}$ is the lowest width (m) of the zone surrounding the contact in the normal direction, and n is a multiplication factor.

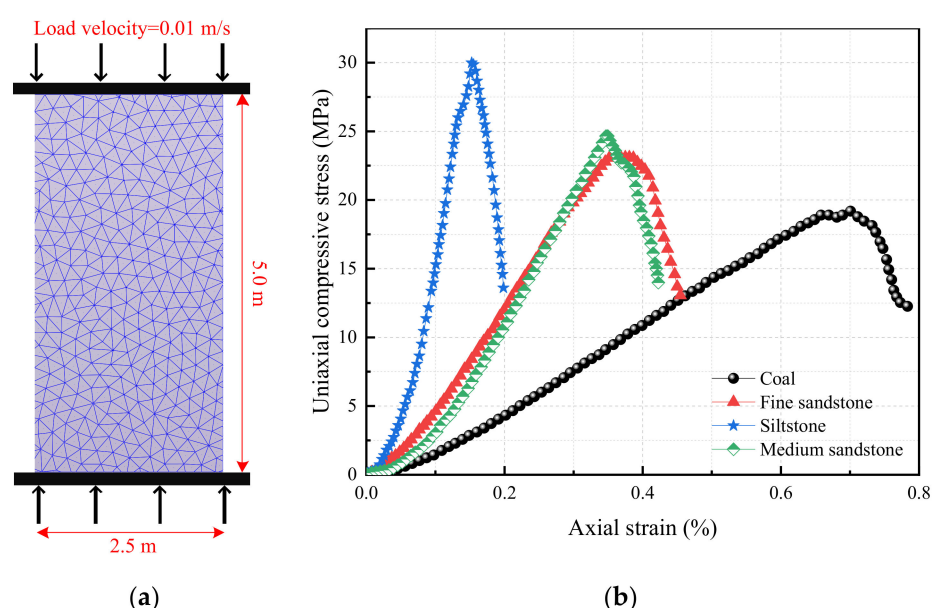


Figure 12. (a) UCS calibration model; (b) the numerical model result.

Finally, a series of unconfined compression tests using the calibration model was run. The strength properties provided for the contacts must be tweaked until the UCS and deformation modulus match the parameters of the rock mass. This involves two sub-steps: first, the contact cohesion, then the friction angle.

To match the rock mass properties shown in Table 1, the input parameters of the blocks and contacts were calibrated using an iterative, trial-and-error process. Table 2 shows the calibrated micro-properties in the Trigon model. These characteristics indicate the qualities of the rock mass at the research location. Table 3 shows the calibration results.

Table 2. Calibration of the model's mechanical behavior of coal measures.

Rock Strata	Matrix Properties			Contact Properties		
	Density (kg/m ³)	E (GPa)	k_n (GPa/m)	k_s (GPa/m)	Cohesion (MPa)	Friction Angle (°)
Siltstone	2500	19.69	945	190	12.0	40
Coal	1400	2.87	392	78	3.5	32
Medium sandstone	2500	7.11	521	104	7.2	36
Fine sandstone	2500	6.48	518	102	6.0	34

Table 3. Elastic modulus and compressive strength of rock mass are compared between theoretical and simulated values.

Rock Strata	E (GPa)		Error (%)	UCS (MPa)		Error (%)
	Target	Calibrated		Target	Calibrated	
Siltstone	21.23	19.69	7.82	31.21	30.00	−4.03
Coal	2.75	2.87	−4.18	18.35	18.90	2.91
Medium sandstone	7.51	7.11	5.63	25.2	24.65	−2.23
Fine sandstone	6.25	6.48	−3.55	24.3	23.11	−5.15

The rock bolts were represented as built-in “Cable” elements in the model, while the steel ladder beams were represented as built-in “Liner” elements. Table 4 lists the properties of the support components employed in this study [36,37].

Table 4. Modeling properties of the support components that were employed.

Contact Properties		Value
Rock bolt/Anchor cable	Elastic Modulus (GPa)	200/200
	Tensile yield strength (kN)	390/870
	Stiffness of the grout (N/m/m)	2×10^9
	Cohesive capacity of the grout (N/m)	4×10^5
Structure	Elastic Modulus (GPa)	200
	Tensile yield strength (MPa)	500
	Compressive yield strength (MPa)	500
	Interface normal stiffness (GPa/m)	10
	Interface shear stiffness (GPa/m)	10

3.3. Simulation Planning

In general, the simulation of the calculation of the stability of roadways in deep coal mines is divided into four steps:

- Step1: Set up the model and apply the initial stress and the boundary conditions;
 Step2: Input model parameters of rock mass and contact interface;
 Step3: Run the model to equilibrium and excavation of TR11-MA, BR11-MA, and RR11-MA. The head machine excavates the roadway gradually and constantly in the field, and the roadway border generates the static stress route. The material softening technique is used to replicate the mechanical excavation of the roadway in order to provide a more realistic excavation influence;
 Step4: The mining of eastern panels and western panels, respectively. A progressive excavation was used to represent the extraction of the panels. A 20 m advanced distance is required for each stage. To eliminate stress, each stage had 40,000 steps to run.

3.4. The Characteristics of Crack Propagation of TR-11MA during Three Periods

3.4.1. Stress of TR-11MA Surrounding Rock

During the TR11-MA excavation process (Period 1), the stress of the roadway surrounding rock was redistributed. The stress reduction zone, the stress rise zone, and the original rock stress zone appeared in the depths of the two ribs of the TR-11MA surrounding rock [38–41]. There is a stress concentration region on the two ribs of the roadway. The position of the peak stress and its distance from the roadside is shown in Figure 13. The vertical stress peak in the eastern side area is about 21.0 MPa, the stress concentration factor is 1.47, and the position of the vertical stress peak value is 4.3 m away from the eastern rib. The peak value of the vertical stress in the western side area is about 22.5 MPa, and the stress concentration factor is 1.57. The position of the vertical stress peak is 4.5 m away from the western rib and is affected by the BR-11MA and RR-11MA on the western side. The influence degree and range of the vertical stress rise area on the western region are larger than those in the eastern region.

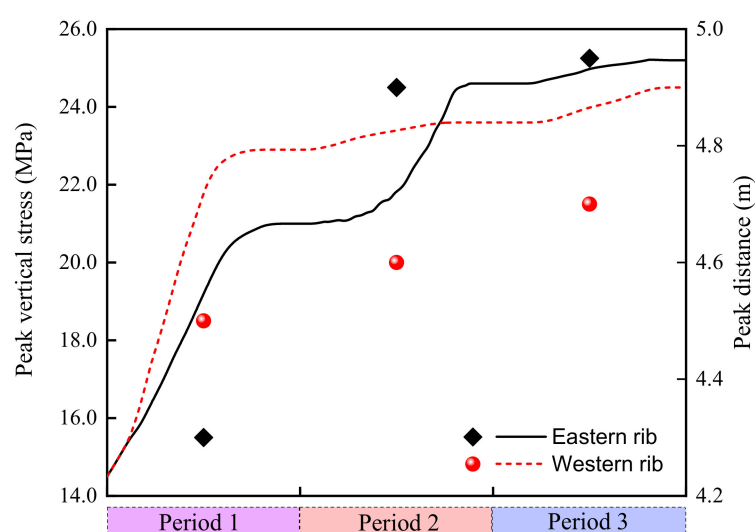


Figure 13. Variation law of the vertical stress peak value and its position on the two ribs of the TR-11MA in three periods.

While the mining process is on the eastern side of the panel (Period 2), due to the influence of the movement of the overlying strata, the roadway has undergone the superposition of mining support stress, and the area of stress rise is relatively large [41–43]. The vertical stress concentration on the eastern side of the roadway significantly increased. The peak vertical stress on the eastern side increased to 24.6 MPa, the stress concentration factor is 1.72, the stress concentration factor increased by 0.25, and the position of the vertical stress peak is 4.9 m away from the eastern rib. The peak vertical stress on the western side of the road increased to 23.3 MPa, the stress concentration factor is 1.62, the stress concentration coefficient increased by 0.05, and the position of the vertical stress peak is 4.6 m away from the western rib. The influence degree and scope of the vertical stress rise area on the eastern region are larger than those on the western region.

While the western panel is in the process of mining (Period 3), the relative horizontal distance between the western panel and TR11-MA is relatively large, and RR-11MA and BR-11MA play the role of pressure relief roadways to a certain extent [26,44]. Therefore, the western side panel has little effect on the vertical stress distribution of the surrounding rock of the TR11-MA. The vertical stress peak on the eastern side increased to 25.5 MPa, the stress concentration factor is 1.78, the stress concentration factor increased by 0.06, and the position of the vertical stress peak is 4.95 m away from the eastern rib. The peak vertical stress on the western side of the road increased to 24.2 MPa, the stress concentration factor is 1.69, the stress concentration coefficient increased by 0.07, and the position of the vertical stress peak is 4.7 m away from the western rib. The peak vertical stress of the surrounding rock of the eastern rib increased by 0.5 MPa, and the range of influence slightly increased. The peak vertical stress of the surrounding rock of TR-11MA increased slightly, but the change was not obvious, and the impact was small.

3.4.2. Cracks of TR-11MA Surrounding Rock

After the excavation of the TR-11MA, the surrounding rock first undergoes plastic deformation, resulting in a large number of tension cracks, which further penetrate and form and cause the destruction of the rock mass. They gradually extend from the surface of the surrounding rock to the deep part, and a large number of shear cracks appear in the shallow part of the surrounding rock. The shearing and tensioning cracks penetrate each other and finally form a broken zone [6,7,12].

As shown in Figure 14, during the TR11-MA excavation process (Period 1), because the two ribs of TR11-MA were affected by the RR-11MA and BR-11MA excavation to different degrees, the tensile cracks of the eastern and western sides were distributed

asymmetrically. The tensile cracks on the eastern rib are up to 1.5 m deeper, and the surrounding rock damage is more serious, and the tensile cracks on the western rib are mainly distributed in the surrounding rock within 2.0 m. The depth of the roof shear cracks is 2.5 m. The distribution range of shear cracks is roughly an inverted ellipse, which is because the side pressure coefficient of the surrounding rock of TR-11MA is 1.2, and the stress is redistributed after the TR-11MA is excavated. The horizontal stress in the TR-11MA is greater than the vertical stress [29,45].

During the mining process on the eastern side of the panel (Period 2), the development of tensile cracks on the eastern rib of TR-11MA is more obvious than that on the western rib. The deepest part of the shear cracks on the western rib reached 3.5 m, and the shear cracks in the eastern rib and the roof cracks of the roadway are more intense. The depth of the roof shear cracks is 6.5 m, and the depth of the eastern side shear cracks is 3.7 m.

During the mining process on the western side of the panel (Period 3), the cracks on the western rib of TR-11MA are more developed than those on the eastern rib. The depth of the shear cracks in the western rib is 3.7 m, the depth of the shear cracks on the eastern rib is 4.0 m, and the deepest part of the tensile roof cracks reached 3.0 m. The depth of the shear cracks in the eastern rib can reach 6.8 m and is partially connected with the tensile cracks on the eastern rib.

The evolution law of the number of cracks in the surrounding rock of TR-11MA is shown in Figure 15. During the TR11-MA excavation process (Period 1), the number of shear cracks in the roof region increased from 0 to 1664, and the number of tensile cracks increased from 0 to 349. During the mining of the eastern side panel (Period 2), the number of shear cracks in the roof region increased from 1664 to 1851, and the number of tensile cracks increased from 349 to 421. During the mining of the western side panel (Period 3), the number of shear cracks in the roof region increased from 1851 to 1941, and the number of tensile cracks increased from 421 to 458. Therefore, the mining of the eastern side panel has a greater impact on the increase in the number of shear and tensile cracks in the roof region.

During the TR11-MA excavation process (Period 1), the number of shear cracks on the eastern side region increased from 0 to 1468, and the number of tensile cracks increased from 0 to 350. During the mining of the eastern side panel (Period 2), the number of shear cracks in the eastern side region increased from 1468 to 1856. The number of tensile cracks increased from 350 to 427. During the mining of the western side panel (Period 3), the number of shear cracks on the eastern side region increased from 1856 to 1980, and the number of tensile cracks increased from 427 to 444. Therefore, the mining of the eastern side panel has a greater impact on the increase in the number of shear and tensile cracks in the eastern rib region.

During the TR11-MA excavation process (Period 1), the number of shear cracks in the western side region increased from 0 to 1493, and the number of tensile cracks increased from 0 to 348. During the mining of the eastern side panel (Period 2), the number of shear cracks in the western side region increased from 1493 to 1741, and the number of tensile cracks increased from 348 to 440. During the mining of the western side panel (Period 3), the number of shear cracks in the western side region increased from 1741 to 1821, and the number of tensile cracks increased from 440 to 464. Therefore, roadway excavation and mining of the eastern side panel had a greater impact on the increase in the number of shear and tensile cracks in the western side region.

Table 5 shows the cracks and degree of damage of TR-11MA's surroundings obtained from the numerical results.

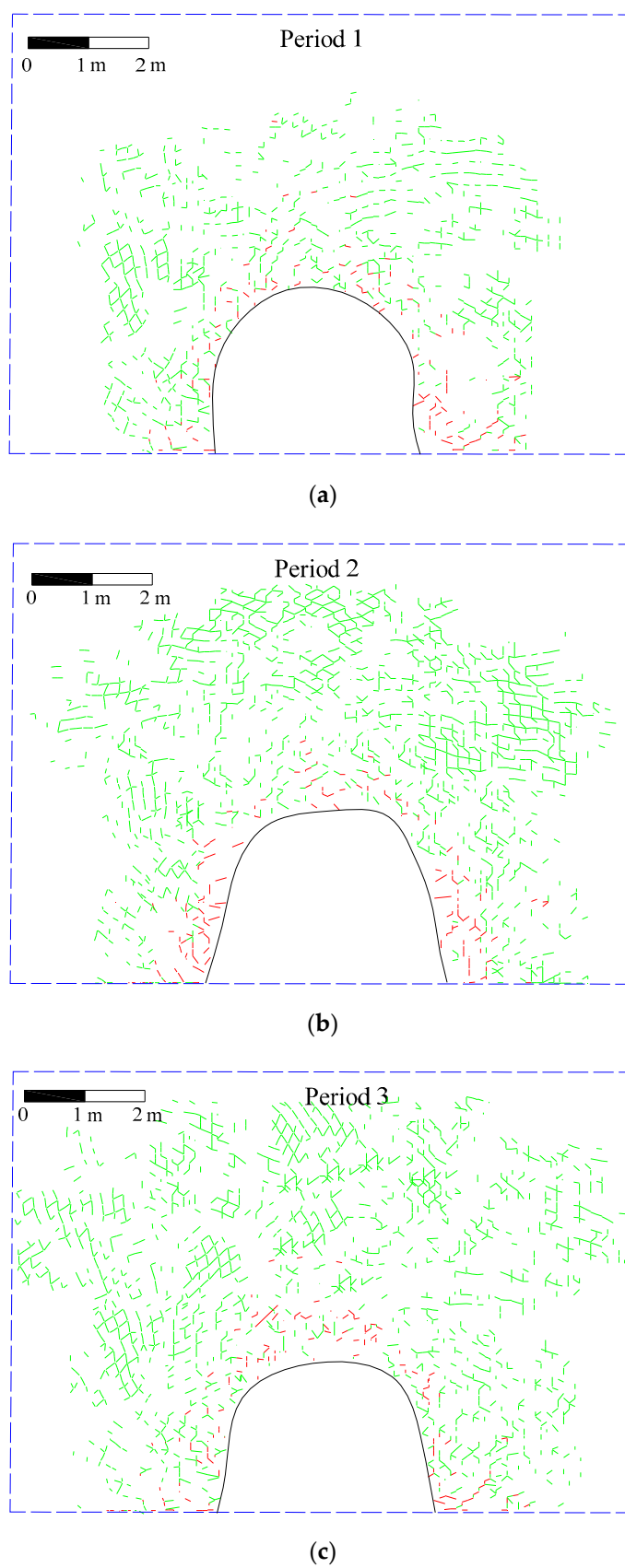
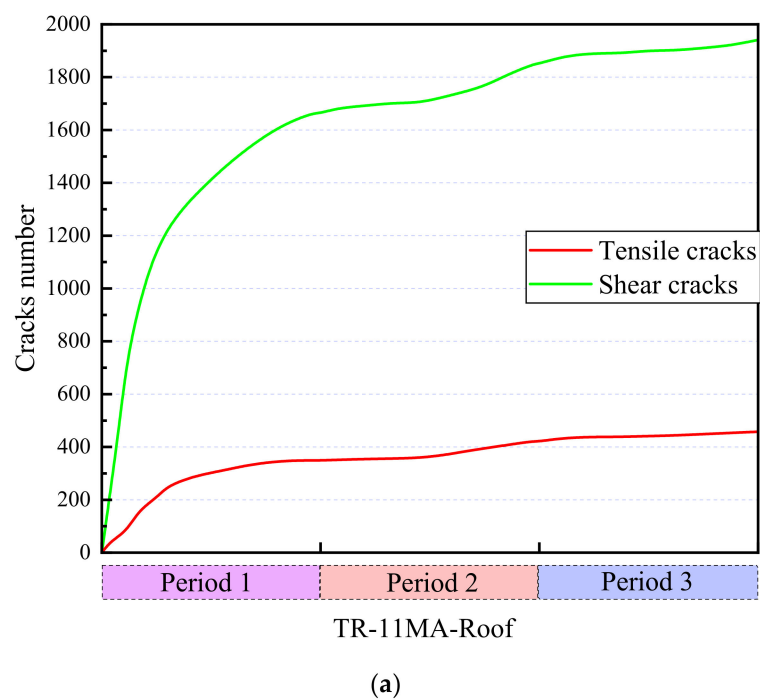
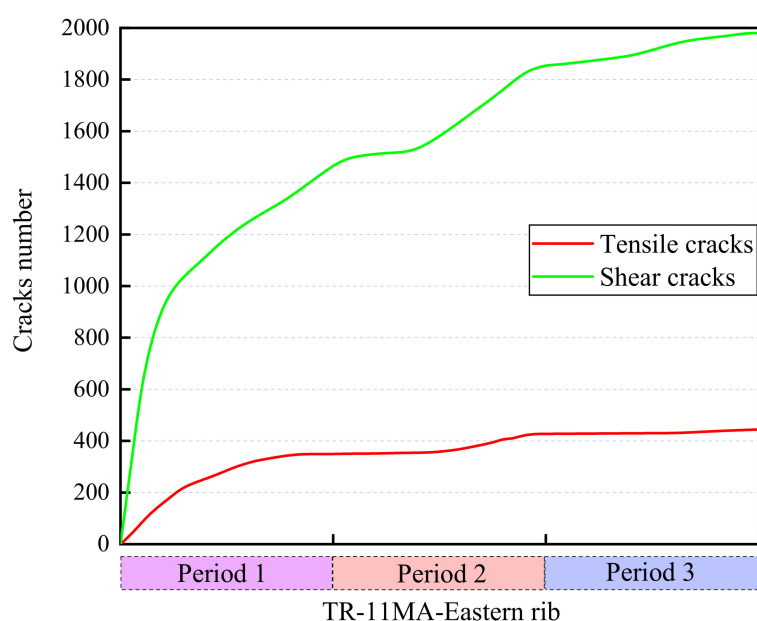


Figure 14. The characteristics of cracks change in the three periods. (a) Period 1; (b) Period 2; (c) Period 3; (In the figure, the red lines represent tensile cracks, and the green lines represent shear cracks).

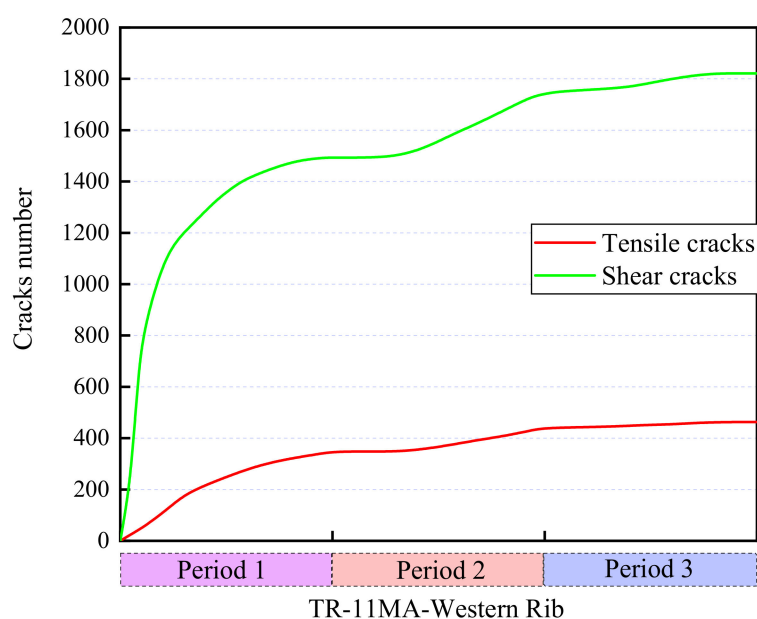
Table 5. The characteristics of crack propagation and degree of damage of TR-11MA.

Items		TR-11MA (Maximum Value)		
		Roof	Western Rib	Eastern Rib
The depth of the tensile cracks (red)	Period 1	2.5 m	2.0 m	1.5 m
	Period 2	2.8 m	2.3 m	2.5 m
	Period 3	3.0 m	2.7 m	2.8 m
The depth of the shear cracks (green)	Period 1	5.2 m	3.0 m	2.8 m
	Period 2	6.5 m	3.5 m	3.7 m
	Period 3	6.8 m	3.7 m	4.0 m
The number of the tensile cracks (red)	Period 1	349	348	350
	Period 2	421	440	427
	Period 3	458	464	444
The number of the shear cracks (green)	Period 1	1664	1493	1468
	Period 2	1851	1741	1856
	Period 3	1941	1821	1980
Degree of damage	Period 1	33%	29%	40%
	Period 2	70%	72%	64%
	Period 3	76%	75%	70%

**Figure 15.** Cont.



(b)



(c)

Figure 15. The evolution curve of the number of cracks in the roof region, eastern rib, and western rib region of TR-11MA in the three periods. (a) Roof region; (b) Eastern rib region; (c) Western rib region.

3.5. Degree of Damage around TR-11MA's Surrounding Rocks

The surrounding rock of the roadway is affected by the disturbance and is accompanied by a change in the number of cracks and a change in stress. Therefore, the damage to the rock will lead to the strength of the rock [46–48], which is an important index for evaluating the stability characteristics of the surrounding rock. Tang et al. [46] proposed a novel technique for predicting the depth of an excavation damage zone based on a modified nonlinear Mohr failure criterion and modifying a standard analytical solution using the perturbation method. In the discrete element model, to represent brittle materials, the rock mass is considered as a set of triangular blocks connected together by internal contact. These areas cannot fail, assuming that each triangular block is made of an elastic

material and split into triangular finite-difference regions. Shear or tensile stress should only produce damage along the contact surface, and the intensity of the contact surface determines how much damage occurs. The stress–displacement relationship is thought to be linear in the direction of vertical contact, and stiffness k_n is the controlling factor:

$$\Delta\sigma_n = -k_n\Delta u_n \quad (4)$$

where $\Delta\sigma_n$, MPa represents the effective normal stress increase, and Δu_n m represents the normal displacement increment.

In the shear direction, a constant shear stiffness governs the behavior. Contact micro characteristics, cohesive properties, and friction properties all play a role in determining shear stress, τ_s , MPa, thus, if:

$$|\tau_s| \leq c + \sigma_n \tan \varphi = \tau_s^{\max} \quad (5)$$

Then:

$$\Delta\tau_s = -k_s\Delta u_s^e \quad (6)$$

Or else, if:

$$|\tau_s| \geq \tau_s^{\max} \quad (7)$$

Then:

$$|\tau_s| = \text{sign}(\Delta u_s)\tau_{\max} \quad (8)$$

where c , φ , Δu_s^e , and Δu_s represent the cohesion, friction angle, elastic component of the incremental shear displacement, and total incremental shear displacement, respectively.

According to Gao's [37] specifications, the degree of damage is posited in the Equation (9):

$$D = \frac{l_{\text{shear}} + l_{\text{tensile}}}{l_{\text{total}}} \times 100\% \quad (9)$$

where l_{total} , l_{shear} , and l_{tensile} represent total contact length, total shear crack length, and total tensile crack length (m), respectively.

Figure 16 shows the change curve of the damage degree of the TR-11MA around the roof and the two ribs of the monitoring areas in three periods.

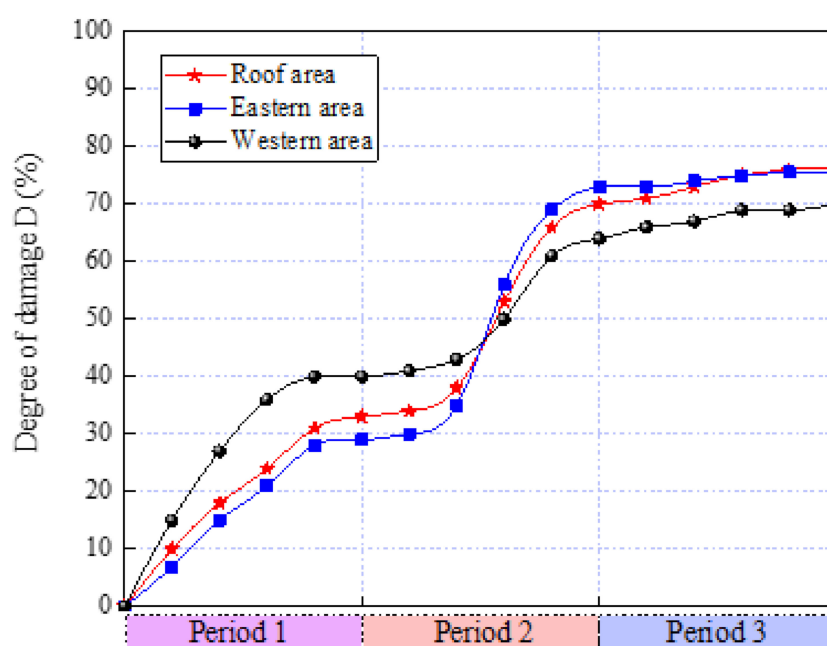


Figure 16. Degree of damage evolution of TR-11MA at three periods.

- (1) During the TR11-MA excavation process (Period 1), the degree of damage to TR-11MA in the roof region, eastern region, and western region ranged from 0% to 33%, 0% to 29%, and 0% to 40%, respectively;
- (2) During the mining of the eastern side panel (Period 2), the degree of damage to TR-11MA in the roof region, eastern region, and western region ranged from 33% to 70%, 29% to 72%, and 40% to 64%, respectively;
- (3) During the mining of the western side panel (Period 3), the degree of damage to TR-11MA in the roof region, eastern region, and western region ranged from 70% to 76%, 72% to 75%, and 64% to 70%, respectively;

4. Result and Discussion

4.1. Reinforcement and Repair Technology of TR-11MA

The grouting reinforcement of the roadway surrounding the rock mainly acts on the structural fracture plane. By injecting grout, the friction of structural planes is improved and cemented to each other. At the same time, the grouting pressure in the grouting process prevents the shear failure of the weak structural plane so that the physical and mechanical parameters of surrounding rock are improved, such as the internal friction and cohesion of rock mass will be greatly enhanced. Bolt support is used to strengthen the control of the surrounding rocks of the roadways [49]. The bearing structure formed by the surrounding rock of the roadway after grouting improves the stress environment of the rock bolt and greatly enhances the anchoring ability of the rock bolt so that the rock bolt anchor ring and the grouting reinforcement ring can bear the pressure more effectively.

(1) Reinforcement and repair section

The control process of the reinforcement repair section of TR-11MA is: Extend the roadway to the design section size → Initial sprayed concrete layer to ensure the integrity of the shallow surrounding rock → Grouting to strengthen the anchor ring → “Rock bolt + steel mesh + anchor cable” support in the full section of the roadway → Re-sprayed concrete to the design thickness of the roadway. (“→” represents the connection between the sub-steps.)

(2) New excavation extension section

The control process of the new excavation extension section of TR-11MA is: Initial sprayed concrete layer to ensure the integrity of the shallow surrounding rock → “Rock bolt + steel mesh + anchor cable” support in the full section of the roadway → Grouting to strengthen the anchor ring → Re-sprayed concrete to the design thickness of the roadway.

The specific support parameters of the TR-11MA repair section and the extension section are shown in Table 6, and the rock bolt and anchor cable supporting details are shown in Figure 17:

Table 6. TR-11MA support parameters and material selection.

	Parameters	Requirements	Remarks
Rock bolt	Size	Φ 22 mm L 2400 mm	Q335 Left screw steel bolt without longitudinal reinforcement
	Spacing and row spacing	800 mm and 800 mm	
	Anchoring agent	CK2335 + K2360	
	Pre-tightening torque	300 N·m	
Anchor cable/Normal	Size	Φ 21.8 mm L 4300 mm	1 × 19 Steel stranded mine anchor cable
	Spacing and row spacing	1500 mm and 1600 mm	
	Anchoring agent	CK2335 + K2360	
	Pretension	300 kN	
Anchor cable/Reinforcement	Size	Φ 21.8 mm L 8300 mm	1 × 19 Steel stranded mine anchor cable
	Spacing and row spacing	1500 mm and 1600 mm	
	Anchoring agent	CK2335 + 2K2360	
	Pretension	300 kN	
Reinforced ladder beam	Size	Φ 18 × 60 × 4000 mm	Round steel welding
Rock bolt pallet	Size	150 mm × 150 mm × 10 mm	Butterfly pallet
Anchor cable pallet	Size	300 mm × 300 mm × 16 mm	Butterfly pallet

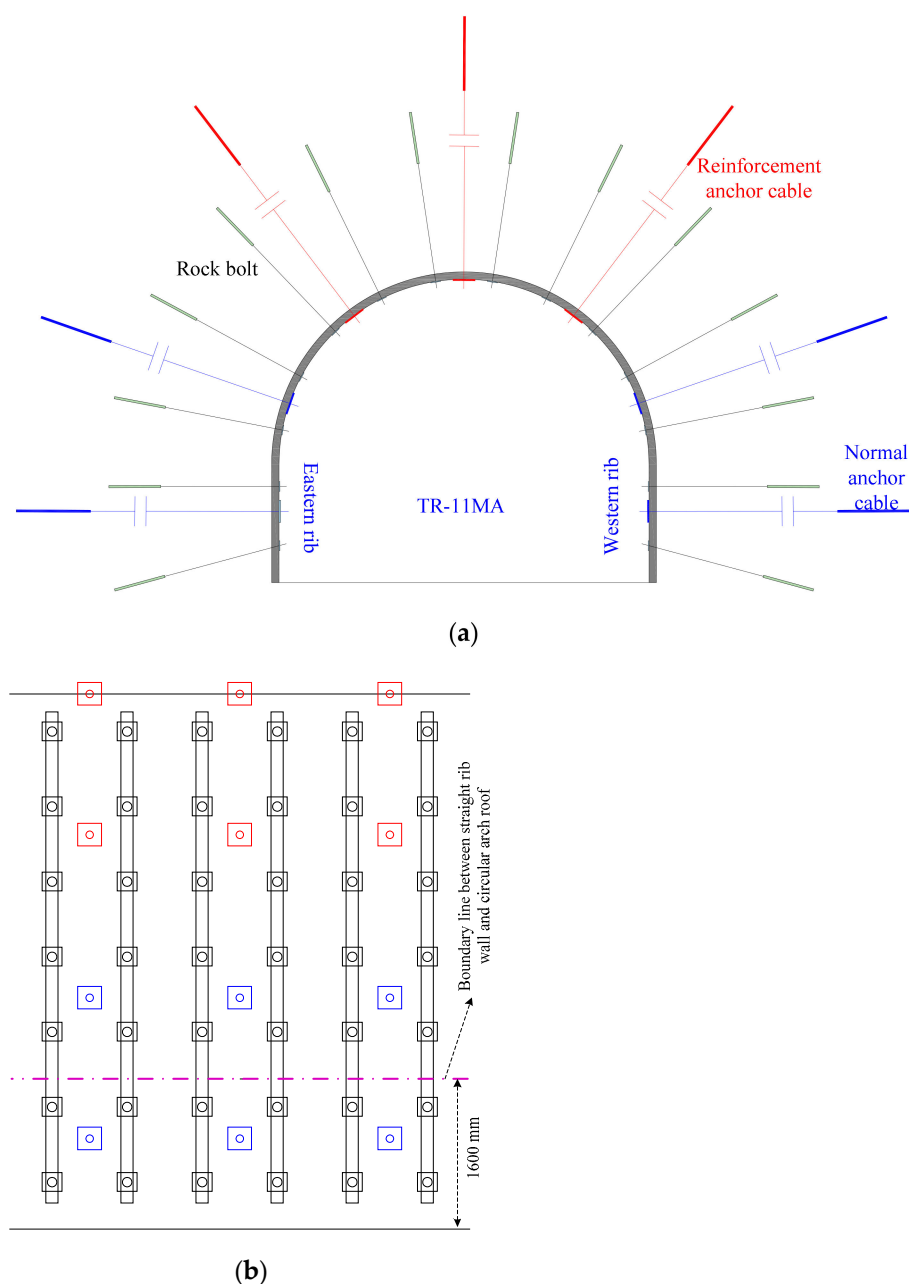


Figure 17. The rock bolt and anchor cable supporting parameters of TR-11MA. (a) Main support section view; (b) Lateral support view.

4.2. Control Effect of Reinforcement and Repair of TR-11MA

In order to study the control effect of the combined support technology on the deformation of the surrounding rock of the TR-11MA, three different monitoring stations were set up in the TR-11MA repair section and the new excavation extension section to observe the deformation and the bolt load of the TR-11MA.

The deformation of TR-11MA is shown in Figure 18a. When the above-mentioned roadway support parameters are adopted, the deformation of the roof-to-floor of the roadway and the surrounding rock of the rib-to-rib is obviously controlled. The convergence speed of the roof-to-floor is about 11.5 mm/d, and the deformation of the roadway tends to be stable after 30 days, and the maximum convergence value of the roof-to-floor is about 370 mm after 60 days. The convergence speed of the rib-to-rib is about 9.5 mm/d, and the convergence value of rib-to-rib is about 300 mm after 60 days.

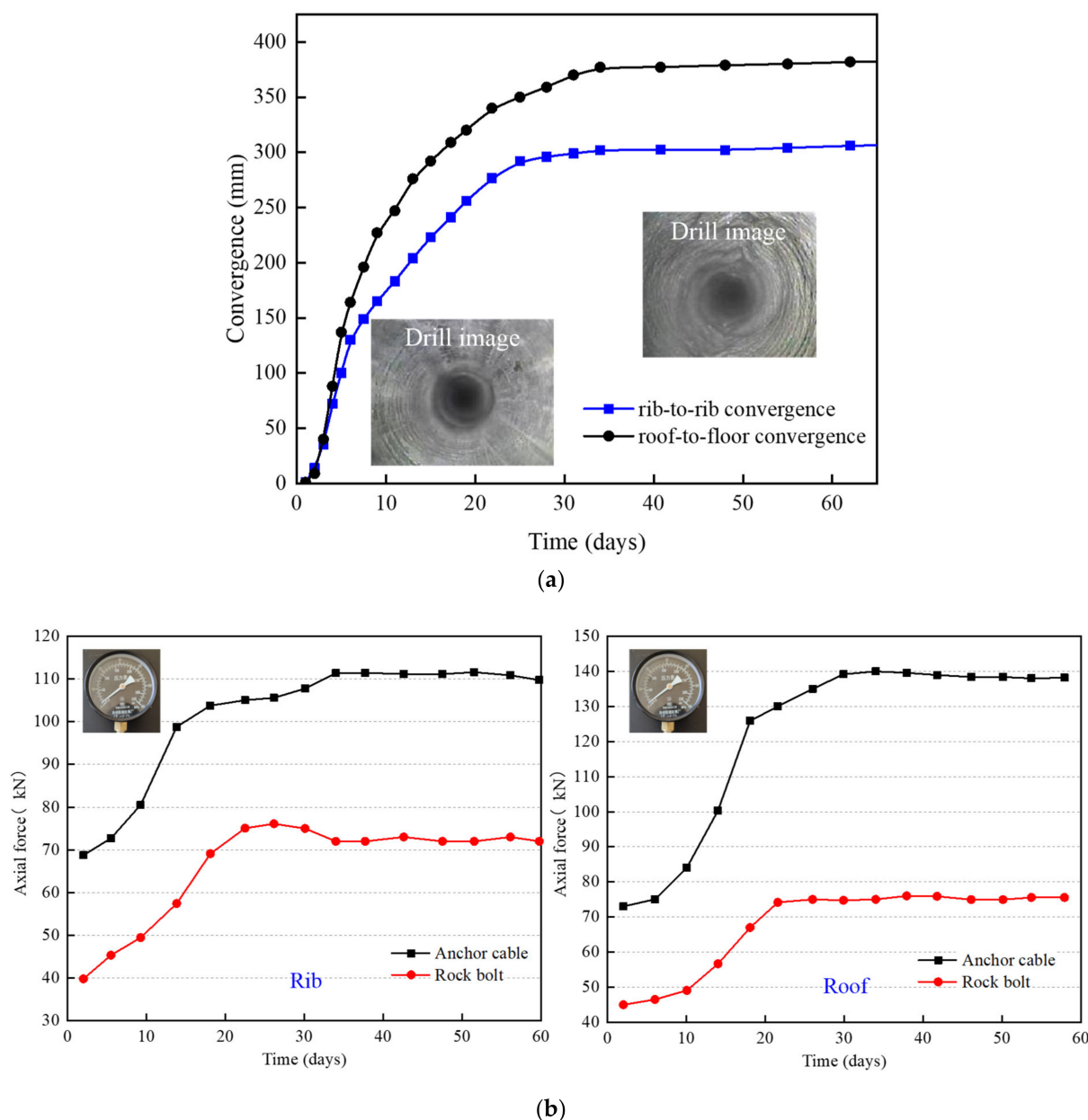


Figure 18. The monitoring results of TR-11MA in the station. (a) The deformation curves and drill image of TR-11MA; (b) The curves of rock bolts and anchor cables axial force.

The rock bolts and anchor cables axial force of TR-11MA are shown in Figure 18b. The observed initial value of the axial force of the anchor cable on the roof of TR-11MA is about 72 kN. The fluctuation of the axial force of the anchor cable on the roof increases as the number of monitoring days increases, and the axial force of the anchor cable on the roof of TR-11MA gradually stabilizes at around 140 kN. The initial observation value of the axial force of the rock bolt on the roof was about 45 kN. With the increase in monitoring days, the fluctuation of rock bolt force increases, and the axial force of the rock bolt in the roof gradually stabilizes at about 75 kN. The initial value of the axial force of the anchor cable in the ribs of TR-11MA is about 70 kN. With the increase in monitoring days, the fluctuation of the axial force of the anchor cable in the ribs increases, and the axial force of the anchor cable in the ribs of the TR-11MA gradually decreases and stabilizes at about 110 kN. The initial observed value of the axial force of the anchor cable in the ribs was about 40 kN. With the increase in monitoring days, the axial force of the anchor cable in the ribs gradually stabilizes at about 72 kN.

The axial force of the rock bolt and anchor cable on the roof and ribs of the TR-11MA then tends to be stable. When combined with TR-11MA's monitoring deformation results, it demonstrates that the supporting parameters of the rock bolts and anchor cables have a significant control effect on the surrounding rock, and TR-11MA's rock surroundings are effectively controlled.

4.3. Implications for Theory and Practice

Deep roadways are prone to a variety of challenges, such as asymmetrical high ground stress, making it hard to understand the reasons for structural instability and adopt effective approaches [11]. In this paper, the deformation and failure process of deep high-stress roadways is divided into three stages according to the different periods of experiencing mining disturbance. This research method can compare the deformation and instability characteristics of the deep high-stress roadways in three periods in detail, and finally invert the roadways instability mechanism and control technologies and provide a method and way for the control of the deep roadways. The in-site learning methods in this article are operable and repetitive, which is convenient for in-site application and promotion [26].

5. Conclusions

This study used a combination of field investigation and numerical simulation methodology to reveal the mechanism of abnormal deformation and failure characteristics of deep high-stress roadways induced by mining disturb. The following are the main conclusions:

The field study results demonstrate that the mechanism of deformation and failure of TR-11MA in the eastern panel mining disturbance. Moreover, the matching test of the support structure evidences the importance of the selection of supporting structure parameters.

The initiation, propagation, and coalescence of internal cracks were investigated throughout the formation process of the TR-11MA, which showed that the stress in the ribs wall of TR-11MA at period 2 is 23.3 MPa and 24.6 MPa, respectively.

The surrounding rock control parameters of the new extension section and the reinforcement repair section of TR-11MA are proposed, respectively, and the field application effect is significant, which provides a reference for the support design of the deep roadway and the sustainability of the development of a deep coal mine.

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