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Investigations of Support Failure and Combined Support for Soft and Fractured Coal-Rock Tunnel in Tectonic Belt

Weijian Yu · Genshui Wu · Baifu An

Abstract The structural integrity of coal and rock tunnel affected by structural belts is generally poor, and the fracture development leads to the failure of support. Therefore, in this study, the tunnel deformation characteristics and deformation control technology were conducted using a combination of field investigation, laboratory experiment, theoretical analysis, field test, and other methods based on the Mukong coal mine. First, field investigations show that the main structural support failure often starts from some key parts associated with weak strengths, and the deformation is basically unevenly distributed. For this, the structural plane of tunnel rock mass was investigated and the internal structural deformation of the surrounding rocks was also monitored through the peepholes. In addition, point-load strength test was also conducted. Field observations and testing results show that rock fissures in the tunnel are well developed and a large loose circle will be formed after excavation. According to Barton Classification Q value and rock mass rating value, the mining tunnel stability falls into the category of Class V, i.e., the surrounding rock is poor or extremely poor. Finally, the support principles concerning surrounding rock stability control technology were proposed. The core idea is using support reinforcement repeatedly for several times until both internal and external load-bearing capacities of the surrounding rock are met. For the support reinforcement, a test with combined support scheme was conducted, including short bolts, long anchor cables, and U-steel brackets. Monitoring results of site application show that the proposed scheme contributes to the long-term stability of tunnel surrounding rock.

Keywords Soft and fractured rock mass · Surrounding rock control · Support failure · Deformation mechanism · Control principle

1 Introduction

After the tectonic movement, rock mass is basically characterized with poor structural integrity, fissure development and low mechanical strength. Under the influence of residual tectonic stress, which is mainly the horizontal stress, the fissures would further develop. Therefore, tunnel excavation in such rock mass is prone to causing large-scale deformation, or even failure. This would bring difficult to tunnel maintenance, cost-consuming and roof fall accidents

W. Yu (✉) · G. Wu · B. An
Hunan Provincial Key Laboratory of Safe Mining Techniques of Coal Mines, Hunan University of Science and Technology, Xiangtan 411201, Hunan, China
e-mail: ywjlah@163.com

W. Yu · G. Wu · B. An
School of Resource and Environment and Safety Engineering, Hunan University of Science and Technology, Xiangtan 411201, Hunan, China

(Yu et al. 2015a, b; Diering 1997; Dhang 2016; Nussbaum et al. 2017). In recent years, extensive researches have been reported on deformation mechanism and control method in soft and fractured rock tunnel and fruitful results have been achieved. Two groups of factors of the original geological factors and the human-induced factors can be categorized among various studies, which are responsible for the large deformations and instabilities (Du et al. 2017; Gale et al. 1992).

Concerning deformation mechanism based on quantitative GSI, rock rating system, and continuum theory, Zhang et al. (2016) established a post-peak strain-softening mechanics model for fractured rock mass by taking into account the confining pressure. The results show that the calculated results obtained from the perfect elastoplastic model and strain-softening model are quite different. Hence, the perfect elastoplastic model should not be applied to support design directly. In addition, Yu et al. (2009, 2015a, b) observed that the geological structure damages the surrounding rock and tried to reveal the internal cause of surrounding rock rheology. According to their results, the larger the lateral pressure coefficient, the more obvious the rheology is, and such variation of rock rheology tends to speed up deformation in the later stage. Jiang et al. (2009) reported an example of surrounding rock mechanics and deformation failure characteristics in Xiaokang coal mine. They pointed out that the major reasons for tunnel damage and support failure are caused by uneven deformation of the original support system, low supporting resistance and the loss of self-bearing capacity of surrounding rock. Li et al. (2011) studied the influence of joint fissure distribution upon surrounding rock stability in the tunnel, and concluded that the scale of rock mass structural plane has a significant influence on the stress state of tunnel surrounding rock. More specifically, the bearing capacity of fractured surrounding rocks is obviously lower than that of blocky rocks. Besides, Ren et al. (2008) and Moosavi and Grayeli (2006) quantitatively evaluated the asymmetric failure and deformation characteristics of fractured rock tunnel. The results show that the fractured roof strata depth, the degraded rock mass strength and the supporting mode are three factors that exert significant influence on rock mass damage. Hence, if connecting the fractured rock mass with deep stable rock mass using bolts (anchor cable), the structural stability could be effectively enhanced. Consequently, the asymmetric deformation, coal wall extrusion and sliding instability would be reduced, thus effectively reducing the occurrence of roof fall.

Concerning control of soft and fractured surrounding rock tunnel, Xu et al. (2014) proposed equations based on the Mohr–Coulomb strength criterion. The results show that with the rock mass index increasing, the increase rate of cohesive force is about 2–5 times that of the friction coefficient, and the growth rate of uniaxial compressive strength is about 2–3 times that of the uniaxial tensile strength. Zhu and Yu (2011) analyzed the deformation characteristics of surrounding rock on the tectonic belt tunnel of Tailai coal mine. Based on these analyses, they proposed a combined support system including grouting reinforcement, bolting, shotcreting and anchor cable. Yang (2010) put forward a theory of fluctuation equilibrium during the entire interaction between surrounding rock and tunnel support. Despite the fact that fractured rock mass cannot bear the tensile stress, they still have a strong compressive capacity when supported. Considering this fact as well as the mechanical characteristics of slabs under different thickness-to-width ratio conditions, a theory to stabilize slabs with thick bolts was further proposed. Another support system proposed by Wang et al. (2012) is based on bolt-mesh-shotcreting of high strength and high pretension for their active support that was complemented by U-steel yieldable bracket. Failure characteristics in soft fractured surrounding rock and tunnel were further analyzed using high-strength and high-pretension support technology. The technology was successfully applied to field construction and good effect was achieved. Subsequently this method or similar technologies is successfully adopted in some tunnels (Kang 2014; Shreedharan and Kulatilake 2016; Karanam and Dasyapu 2005). Soft and fractured surrounding rock and coal body are normally resulted from tectonic belt and they further deteriorate due to complex stress environments. Consequently, the fractured area constantly expands and the surrounding rock becomes increasingly unstable. According to Dr. Manchao He at China University of Mining and Technology (Beijing), soft rock failure under high in situ stress is considered as a progressive mechanical process that finally is represented to a stage of large plastic deformation. In this sense, the deformation field has nonlinear characteristics. Specifically, the

deformation damage and fracture normally start from the key parts and finally ends up in a form of entire support body instability (He et al. 2002; Sun et al. 2006). In addition, due to the large differences in terms of geological conditions, field applications are significantly varied. In this paper, an example of Mukong coal mine is used and its soft and fractured coal-rock tunnel in a site affected by tectonic belt is studied. Then a series of field investigations, rock mechanics tests, theory analyses and field tests are performed to analyze the tunnel deformation characteristics and finally the site-specific control principles and techniques to keep long term stability of the soft and fractured coal-rock tunnel in tectonic belt are suggested.

2 Regional Geological Investigations

2.1 Regional Geological Survey

Located in Guizhou Province of South China, Mukong coal mine is feathered by typical karst topography. Carbonate rock karst topography is the most widespread in this area, followed by clastic erosional landform. The exposed formations in this region include but not limited to Sinian, Cambrian, Ordovician, Permian, Triassic, Jurassic and Quaternary from old to new, among which Permian and Triassic strata are relatively developed. However, the Silurian, Devonian and Carboniferous formations are missing. The well field is located in the eastern part of the north-wing of Dadingpo anticline (Andi anticline). Folds and faults are barely developed in the region, with only four faulted structures distributed within the well field. The overall strata strike is northeastern and its inclination is northwestern with a dip angle of 8°–42°. Strata in the southeastern part of the well field are quite gentle with a dip angle of 8°–15°, while the northwestern part is steep with a dip angle of 17°–30°. The major faults within the field are F1, F2, F4 and F9. In addition, there is a small hidden fault structure in Changxing Limestone in the ZK1105 drill hole.

At the edge of Yangtze para-platform and its southeastern continent, this area is characterized by widespread carbonate rocks and karst landscape development. Hence, rock mass integrity is reduced in presence of many folds and fractured zones. Figure 1 illustrates the exposed rock mass on the surface and fractured surrounding rock in the tunnel. When rock is excavated, the fissures inside rocks would initiate and propagate under complex stress conditions, resulting in fractured rock mass.

2.2 Surrounding Rock and Coal Seam Occurrence Conditions

Minable seams in the region include coal seams #3, #4 and #5, among which the coal seam #3 is the major seam with average thickness of 1.35 m and leans northwestward with an inclination of 12°–18°. Its roof has a uniaxial compressive strength (UCS) of 33.36 MPa on average. The rock mass is featured by poor integrity and low strength. The detailed information of roof and floor of the coal seam #3 is shown in Table 1. Besides, the absolute gas gushing rate from the tunnel is 0.5 m^3/min and the gas content in #3 coal seam is 17.5 m^3/t. Therefore it can be identified as a seam-proneness of coal and gas outburst according to National Safe Production Industrial Standards of P. R. China (No. AQ1024-2006).

3 Failure Modes of Tunnel Supporting Structures in Fractured Rock and Coal Mass

In the Mukong coal mine, a series of support methods have been developed during tunnel excavation and maintenance, such as I-steel arch, U-steel arch, bolting-mesh-beam, and bolting-mesh-cable, etc. It should be noted that these methods have played significant role in the stability of tunnel surrounding rock and safe mine production. Nevertheless, field applications show that in extremely fractured rock mass where the tectonic stress is very complex, satisfactory results are not achieved yet with application of those methods. In addition, the project is cost-consuming at 20,000 RMB/m (1 USD = 6.76 RMB). Worse, the occurrence of roof fall and caving is reported from time to time, which poses great threat to tunnel safety and long-term stability. In this instance, field investigations were conducted in different types of tunnels in order to understand the deformation characteristics of tunnel surrounding rocks.

Fig. 1 Rock mass exposures in the region. **a** Surface rock mass, **b** underground rock mass

Table 1 Roof and floor description of coal seam #3

Roof/floor name	Rock name	Thickness/m	Rock feature description
Main roof	Clay mudstone	0.6	Clay mudstone
Immediate roof	Siltstone, clay mudstone	1.3	Light gray thin-layered siltstone, clay mudstone, thin-layered siderite limestone
False roof	Sand shale	1.5	Gray–black sand shale, containing siderite nodules in the lower part
Immediate floor	Clay mudstone	0.5	Gray–white clay mudstone
Main floor	Sand shale	3.0	Gray–black sand shale, banded limestone containing siderite, nodules in the lower part; white jointed surface

3.1 Arch Tunnel Deformation with I-Steel Shed and Metal Mesh Support

The original support structure of the main body is using arch shed welded by I-steel. However with pressures accumulated from the loose and fractured surrounding rocks, failures are normally observed in the arch, at welded joints on the wall and at the foot, as shown in Fig. 2. Specifically, in the arch area where the I-steel is bent and steel mesh is damaged, the fractured rock mass would collapse and cavities were formed (Fig. 2a). As for the welded joints on the sidewall, they are usually considered to be the weak links of such sheds structure. Therefore, the welded joints become disjointed easily when experiencing extruding pressure from fractured rock mass on both sidewalls. When this happens, the bearing capacity of the entire bracket would be greatly reduced (Fig. 2b). In addition, there are no fixed force-bearing points at the foot of the steel bracket. So when the floor-wall

Fig. 2 Support failure with I-steel shed and metal mesh. **a** Arch bending, **b** disjointing at the welded points on the sidewall, **c** foot moving inward, overall integrity reduced

angle becomes increasingly large and moves inwards, the support foot moves with the floor-wall angle as well. In this case, the entire support effect would be further reduced (Fig. 2c).

3.2 Arch Tunnel Deformation with U-Steel Yieldable and Metal Mesh Support

The U-steel shed support has both yieldable features and certain bearing capacity, which is prior to I-steel shed in structural manner. Therefore, it could be used for a long time with better effect in comparison with I-steel in Mukong coal mine. Nevertheless, since U-steel shed is a kind of passive support, large structural deformation and support failure would still occur in tunnels with a long service lifetime. The most notable drawback is the large inward deformation induced by uneven surrounding rock stress. Such eccentric deformation assumes the shape of a narrow peach and it usually occurs at joints between sections, as shown in Fig. 3a. Furthermore, due to high uneven stress, the sectional joints could become damaged or even fail, as shown in Fig. 3b. On top of this, since the brackets are not well connected by pull rods, distortion also occurs at the foot area, thus the load-carrying capacity of the entire support is reduced, as shown in Fig. 3c.

3.3 Rectangular or Trapezoidal-Shape Tunnel Deformation with Bolting-Mesh-Cable-Beam Support

To yield optimum support effects, we tried to apply bolting-mesh-cable-belt-beam support method in the trapezium-shape tunnel of this coal mine. The surrounding rock conditions were relatively good. In this circumstance, discussions were conducted with on-site technical staff and it was decided that this support scheme should be applied while excavating along the roof strata. The aim was to enhance the self-bearing capacity of rock mass while reducing the damage to roof strata structure. After implementation of the scheme, rock mass deformation was controlled to a certain extent in the early stage. However, the roof strata fissures continuously expanded with elapsed time, and the rock mass integrity became poor. Finally, the roof rock became so loose that they were separated from the host rock mass. This caused an excessive roof subsidence, as shown in Fig. 4a. At the same time, rock mass under compression on both sidewalls became loose on a large scale; consequently, the junctions of metal mesh were damaged, as shown in Fig. 4b. In the case that the loose circle continuously expanded at the floor, the loose rock mass would be immediately suspended on anchor cables. In case of cable failure or insufficient cable length, roof falls could occur. When this happens, single pillar support is needed, especially in conditions of uneven local stresses, non-compatible deformation, and steel belt distorting, bending and folding, etc., as shown in Fig. 4c.

3.4 Failure Mechanism of Tunnel Surrounding Rock and Supporting Structure

The stress environment, the properties of the surrounding rock, the type of tunnel structure and the supporting type of tunnel are the influencing factors of tunnel stability. Owing to the influence of geological factors, the joints and fissures in the surrounding rock of the tunnel are relatively developed, and there is bedding unconformity between the strata. Meanwhile, the infiltration of groundwater weakens these structural planes severely and forms the weak interlayer. In
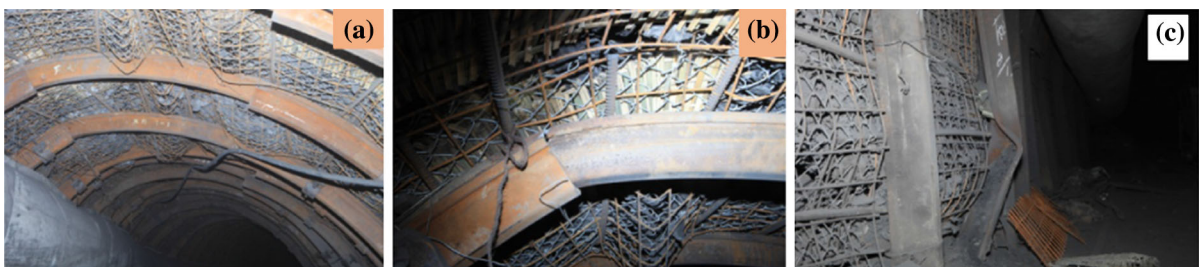

Fig. 3 U-steel yieldable support bracket and metal mesh support failure. **a** Narrow peach arch, **b** a cut at sectional joints, **c** distorted foot

Fig. 4 Combined support scheme with bolting-cable-beam-belt support failure. **a** Roof subsidence, **b** sidewall bulging, **c** severe roof support deformation

addition, the strength of weak interlayer is low, and under the action of horizontal stress, the rock mass is prone to dislocation and deformation between structural planes, coupled with the influence of compressive force inside the rock layer, which further leads to the deflection and delamination deformation of the rock layer. Finally, it leads to the differential deformation or asymmetric deformation of the tunnel, extrusion of side walls and large deformation floor heave, as shown in the Fig. 5.

Under the complicated deep stress field, the fractured rock mass is in a large deformation stage of nonlinear plasticity, and the deformation field of surrounding rock mass is in a nonlinear mechanical field. The destruction of the deep fractured coal-rock tunnel has its characteristics: the mechanical aspects of the support and surrounding rock are not coupled, from the destruction of critical parts to the instability of the whole support system. Therefore, the failure mechanism of the fractured coal-rock tunnel can be divided into the following three types:

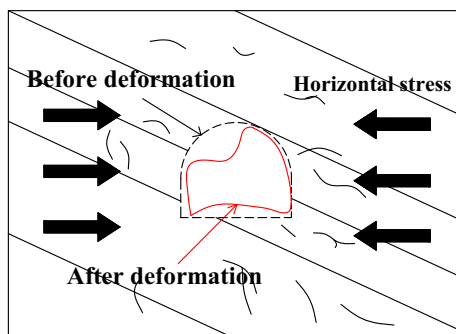(1) The strength decoupling, i.e., the support body and the strength of surrounding rock are not

coupled. Non-uniform load forms partial overload on the equal-strength support body and then generates partial damage, which will eventually lead to instability and failure of the support.

(2) Stiffness decoupling, including positive stiffness decoupling and negative stiffness decoupling. Positive stiffness decoupling means that the stiffness of the support is less than the surrounding rock. In contrast, the negative decoupling of stiffness means that the stiffness of the support is higher than the surrounding rock. Therefore, the internal storage energy of rock mass is not completely transformed into deformation energy, resulting in partial accumulation of energy, which eventually leads to partial overload failure of support.

(3) Structural decoupling, i.e., the decoupling of deformation between the support body and surrounding rock structures. Under the influence of various factors, the surrounding rock shows relatively complex deformation characteristics, and the deformation of the surrounding rock mainly includes elastic deformation, crushing deformation, creep deformation and water absorption and expansion of the surrounding rock. However, due to the uniform density of the support body, its deformation is relatively simple.

Therefore, according to the instability mechanism of the weak part of the fractured surrounding rock, the fracture surface in the surrounding rock of the tunnel, including the primary structure and the fracture surface generated after the destruction of the rock

Fig. 5 Deformable deformation of surrounding rock surface

mass, has a significant influence on the instability of the surrounding rock.

4 Structural Characteristics and Quality Evaluation of Fractured Rock and Coal Mass

The abovementioned failure modes show that these support methods have their respective defects. When these support methods were applied in fractured rocks affected by tectonic belt, the rocks would experience excessive deformation as a result of uneven rock stress applied. In view of the surrounding rock support system, the surrounding rock mass is difficult to control with respect to poor integrity and low mechanical strength. Hence the supporting structures are likely to be influenced by the loose ground stresses. In addition, a number of weak links would also result in support failure.

4.1 Initial Fissure and Its Expansion

1. *Initial fissure development* The structural plane of the rock mass excavated in the original fully mechanized haulage tunnel #1103 was investigated and some results are listed in Table 2. According to Table 2, the rock mass is significantly affected by tectonic fault and many loose and fractured rocks are observed. The fissure spacing ranges from 0.05 to 0.30 m, and there be generally three or more groups of fissures on one structural plane which is formed by many separated bodies. The rock mass integrity is greatly weakened and the overall strength is low, which is also prone to causing large-scale rock instability in combination with other faults and weak structural planes.

According to statistics in the tunnel section, there are 20–35 joints per cubic meter (called J_v) in the rock mass, and 28 joints/m^3 on average. The maximum spacing is 16 cm while the minimum is less than 0.5 cm, with an average spacing of 4 cm. The integrity coefficient of tunnel surrounding rock mass K_v is around 0.12. There are 2–5 groups of joints, 4 groups on average, which are filled with talc and chlorite with a thickness ranging from 1 to 10 cm.

Due to the field conditions restricted, the number of joints surveyed in the tunnel is limited. Nevertheless, the results obtained are applicable to reflect the actual situation. The joint pole and two-dimensional joint network simulation by Monte-Carlo method are illustrated in Fig. 6. In Fig. 6, joint fissures are quite scattered and there was a group of joints running across the northeast and southwest in the tunnel rock mass. On the whole, the surveyed tunnel has small joint spacing, large joint density, poor rock mass integrity, and poor engineering geological conditions.

2. *Contribution of stress release to fissure development* Upon rock mass excavation, the in situ rock stress is re-disturbed till a new balance. As a result, the surrounding rock moves towards the free face and the fissures constantly develop within the rock mass. On one hand, under the influence of surrounding rock stress, fissures within coal and rock mass develop at an increasing rate, resulting in tensile failure and large macroscopic deformation. On the other hand, there exists a high level of gas pore pressure within the coal seam. Hence, when the coal seam is excavated and the external stress is released, the high gas pressure inside the fissures promotes its further expansion. Consequently, these crisscrossing fissures around each gas pore and the initial fissures make the coal and rock body more loose.

Figures 7 and 8 are the internal inspection results obtained in the rock mass and coal seam of gate tunnel in Mukong coal mine, respectively. The tunnel is located at an elevation of 700 m above sea level. In those figures, fissures in tunnel sidewall and roof strata are relatively developed, and the rock mass is loose to a large extent. In comparison, the roof strata have better integrity with few fissures when it is 3.5 m away from the peephole. For the sidewall area, however, the rock mass is loose and intensively fractured when it is 0–3.9 m away from the peephole to fissures within the coal mass. Furthermore, at a distance of 4.6 m from the peephole, the rocks of the hole wall are still fractured and peeling off. In conclusion, the fissures in the tunnel are originally well developed and they constantly expand under various stresses. In particular, due to lower coal mass mechanical strength and the influence of gas pore pressure, coal mass fissures in the

Table 2 Survey of surrounding rock joints (part)

Base distance (m)	Occurrence (°)		Type	Continuous length (cm)	Roughness type	Open type	Filling materials and type	Degree of weathering
	Inclination	Tendency						
0.15	85	260	3A	17	VIII	Open	A few talc	Medium
0.18	70	267	3A	10	IX	Open		Medium
0.32	21	60	3A	15	IX	Closed		Medium
0.41	55	46	3A	21	IX	Closed	A few clay	Medium
0.47	65	209	3A	13	IX	Closed		Slight
0.80	43	330	3A	27	II	Open	A few clay	Slight
0.82	65	116	3A	75	I	Open	A few talc	Medium
1.10	78	59	3A	21	II	Open		Medium
1.13	75	63	3A	55	II	Open		Medium
1.25	62	220	3A	48	II	Open	A few talc	Medium
1.84	78	180	3A	29	III	Open		Medium
1.96	53	115	3A	26	IX	Open		Medium
2.04	66	245	3A	28	IX	Closed	A few talc	Slight
2.11	43	16	3A	43	IX	Closed	A few talc	Medium
2.83	65	116	3A	27	II	Open		Medium
2.90	78	220	3A	39	I	Open	A few talc	Medium
3.33	62	119	3A	43	I	Open	A few talc	Medium
3.41	44	271	3A	21	II	Open		Medium

Fig. 6 Rock mass structural plane survey results. **a** Equal density diagram of joint pole, **b** rock mass fissure network simulation

(a)　　　　　**(b)**
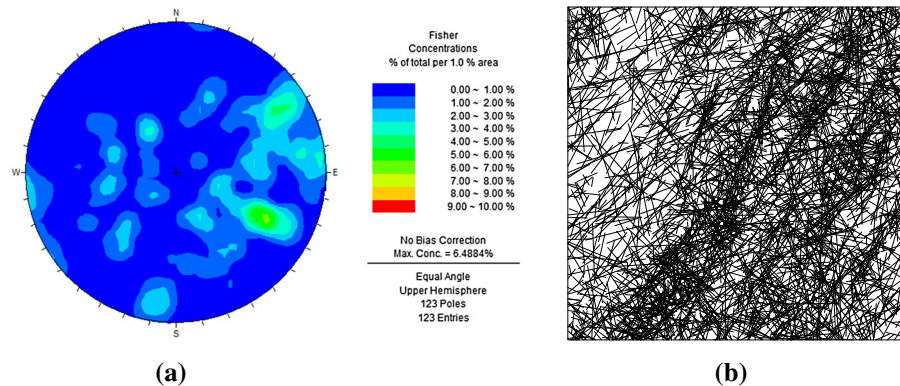
sidewall area are more developed than those in the roof rock mass. This brings about a larger scope of damage.

4.2 Low Rock Mechanical Strength and Rock Swelling

Point-load strength tests were conducted on the rock samples (as shown in Table 3). According to the test results, the rock strength is generally low. Specifically,

the compressive strength is 6.96 MPa for the floor, 7.16 MPa for the roof, and 4.78 MPa for the coal sample, respectively.

However, field observations show that the majority of the exposed surrounding rock mass in the tunnel is fractured (as shown in Figs. 2, 3 and 4). The observations confirm the reduced mechanical strength. Moreover, various stresses attribute to continuous development of initial fissures. As a result, the rock

Fig. 7 Peephole observation of fissures in tunnel roof rock mass. **a** 1.5 m away from the peephole, partially fractured, **b** 2.8 m away from the peephole, partially fractured, **c** 3.5 m away from the peephole, good rock integrity
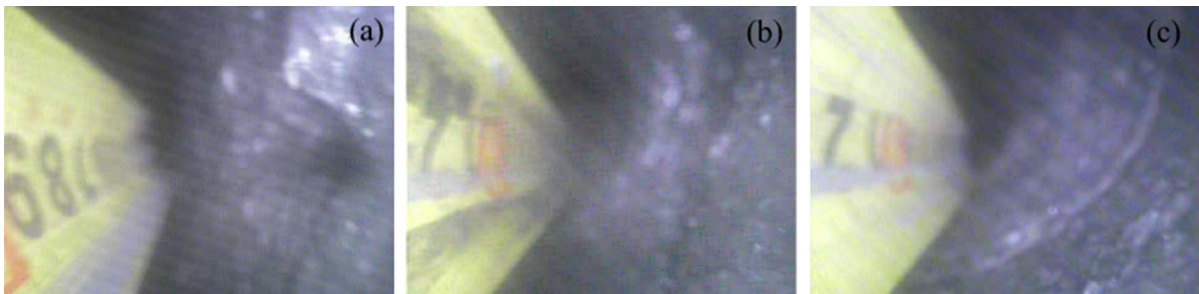

Fig. 8 Peephole observation of fissures in tunnel sidewall coal mass. **a** 1.5 m away from the peephole, partially fractured, **b** 3.9 m away from the peephole, fractured belt, **c** 4.6 m away from the peephole, rock fracture in the hole sidewall

Table 3 Corrected point loading test results of each rock sample

Sample location	Average point-load strength index prior to size correction, I_s/MPa	Average point-load strength index after size correction, $I_{s(50)}$/MPa	Uncorrected compressive strength, R_c/MPa	Uncorrected tensile strength, R_t/MPa	Corrected compressive strength, R_c/MPa	Corrected tensile strength, R_t/MPa
Floor	0.59	0.61	6.83	0.26	6.96	0.26
Roof	0.60	0.62	6.89	0.26	7.16	0.27
No.3 coal	0.38	0.42	4.34	0.16	4.78	0.18

mass integrity is damaged. When the rock strength is markedly reduced, the rock volume increases accordingly. Such swelling effect can reflect the macroscopic deformation of tunnel rock. Specifically, in Mukong coal mine, the swelling coefficient K_p is 1.1–1.2 for both the coal mass and roof rock (clay mudstone), and it is 1.3–1.4 for the floor rock (sand shale). Figures 2, 3 and 4 indicate that under various stresses, the entire tunnel cross-section deformation is not uniformly distributed and the floor rock mass deformation is relatively larger.

4.3 Stability Evaluation of Surrounding Rock

Field investigation was conducted in various tunnels in Mukong coal mine where different support methods had been applied. The preliminary evaluation of engineering geology and stability is shown in Table 4.

The results of stability evaluation show that the horizontal strata currently at service in the coal mine are loose and fractured. Under various stress and tectonic belt influences, the rock integrity is quite poor. For this, the rock integrity is so poor that it is difficult to control large-scale surrounding rock

Table 4 Stability evaluation of tunnels in Mukong coal mine

Site no.	Deformation and failure description	Engineering geological description	Barton Classification Q value	Geological mechanics RMR value	Loose circle estimation/ m	Stability category	Main ground pressure
1	I-steel arch support, with steel mesh inside; local collapse, arch crushed, welded area distorted, poor support structure integrity	Half coal and rock tunnel, loose and fractured rock, small amount of water on the floor; high gas pressure in coal seam, more fractured coal seam than both roof and floor strata, clear interface between rock strata and coal seam	0.05–0.1 (extremely poor)	32–35	3.6–6.0	V	Loose ground pressure
2	U-steel yieldable support, with mesh-shotcreting inside for maintenance. U-steel is like a narrow peach, disconnection at joints, feet distorted; concrete stripping, local protrusion of metal mesh	All rock tunnel, loose and fractured rock body, structural planes on both sidewalls filled with small amount of talc	0.1–0.5 (very poor)	35–41	4.0–5.0	V	Loose ground pressure
3	I-steel arch, with steel mesh inside; roof bent, serious Extrusion on both sidewalls, unsupported sidewall area of 0.9 × 0.6 m², a small amount of floor heave	All rock tunnel, loose and fractured rock in the upper and middle part of the sidewall; could be affected by F1 fault	0.1–0.5 (very poor)	35–41	3.5–5.0	V	Deformation ground pressure-loose ground pressure
4	Trapezoidal tunnel, bolting-mesh-cable-belt support, serious roof subsidence; serious Local protrusion on both sidewalls; much steel belt distortions and tensile failure, single pillar support needed in the later stage	Half coal and rock tunnel, both coal mass and rock mass are fractured; tunnel excavated along the roof, good structure integrity; could be affected by small hidden fault	0.5–1.0 (very poor)	41–44	3.0–4.0	V	Deformation ground pressure-loose ground pressure
5	U-steel yieldable support, with steel mesh inside; serious Extrusion on both sidewalls; 2.1 m of two support feet convergence, 0.3 m of floor heave	Half coal and rock tunnel, both coal mass and rock mass are fractured; muddy fillings on the structural plane, there exists a small amount of fissure water	0.1–0.5 (very poor)	35–41	4.0–5.0	V	Loose ground pressure
6	Trapezoidal tunnel, bolting-mesh-cable support in combination with I-steel Trapezoidal shed; roof subsidence; steel beam bent, poor load-bearing capacity for bolts, tensile failure for anchor cables	All rock tunnel, very fractured rock mass, tunnel excavated along the roof, affected by hidden fault, fault missing	0.01–0.05 (extremely poor)	26–32	5.5–6.0	V	Loose ground pressure

deformation in the tunnel, when using I-steel arch or U-steel yieldable support, or combined support of bolting-cable. According to Barton Classification Q value and rock mass rating (RMR) value, the stability of gateway surrounding rock is categorized as class V, suggesting the extremely poor or very poor of rock quality. There is loose ground pressure in the majority of the rocks, thus most of the fractured rock mass weight is born by the supporting structure. In this sense, the surrounding rock almost loses its self-bearing capacity.

5 Fractured Rock and Coal Mass Control Theory

5.1 Principles of Integrated Control Technology

(1) Timely control of excessive surrounding rock deformation and improvement of initial support strength. Once excavation starts, the surrounding rock deformation increases while its strength decreases. When the deformation reaches a certain value, the surrounding rock could be transformed from elastic medium to plastic and then into loose medium, resulting in the loss of its self-stability or even rock collapse. Therefore, before stress induced by ground deformation is transferred to the loose ground, supports with certain rigidity shall be taken timely to control of the surrounding rock deformation. Such support measures could also be used to control surrounding rock deformation from rapid expanding and avoid further damage.

(2) Strengthening the fractured rock with respect to self-bearing capacity of surrounding rock. Rock and coal mass under the influence of tectonic belt have developed various fissures and their overall structural integrity is rather poor. Therefore in order to improve the mechanical strength of the surrounding rocks, it is necessary to improve its integrity at early stage as possible before deformation ground pressure is converted into the loose ground pressure. Specifically, full-length anchoring support and grouting should be used if necessary. In addition, the floor should be sealed timely to limit potential rock deformation. Furthermore, the complete structure combined with floor sealing could provide large bearing capacity to combat tunnel deformation.

(3) Secondary or multiple supports to improve long-term stability of the surrounding rock. Under a long-time stress imposed by Monte-Carlo method, the rock deformation can last for a long time. Hence, the maintenance of tunnel stability in the late stage is of great significance. To achieve long-term tunnel stability, it is very necessary to carry out secondary or multiple supports to remain integrity.

5.2 Key Factors in Tunnel Design and Maintenance

1. *Allowable tunnel space* Under complex stress environments, the surrounding rock deformation in mining drift is considerably large and can last for a long time period, which would cause contraction of the entire tunnel section. The drastically reduced tunnel space makes it difficult to meet production requirements. In this case, to meet the passageway, transportation and ventilation requirements, the allowable tunnel space has to be guaranteed, i.e. the maximum tunnel deformation should be controlled. According to site-specific requirements, the maximum allowable tunnel displacement [u] could be determined by the difference between tunnel size (span or height) of design and operation, which is expressed by:

$$[u] = u_d - u_a \qquad (1)$$

where [u] is the maximum allowable tunnel displacement, m; u_d is the design tunnel size (span or height), m; u_a is the minimum space size (span or height) for operations, m.

2. *Critical deformation for tunnel collapse* The maximum allowable displacement should be the critical displacement value before rock collapse, so the deformation rate of tunnel surrounding rock should be taken into account.

   1. Approximately constant rate change. As shown in Fig. 9, although the deformation rate of the support tunnel is not a constant value fluctuating within a certain range, but it could be treated as one a constant value

approximately in the whole process. Therefore, the average deformation rate can be used to estimate tunnel deformation by

$$u = u_0 + \bar{v} \cdot t \tag{2}$$

where u is the total surrounding rock deformation after t days of tunnel support, m; u_0 is the deformation before secondary tunnel support, m; $\bar{v}$ is the average deformation rate after t days of secondary support, m/d; t is the monitoring time after the secondary support, d.

2. *Linear acceleration variation* In the case of very poor surrounding rock condition and high stress concentration, the support strength is not enough to meet the tunnel stability requirements. The tunnel deformation continues to develop until the occurrence of tunnel failure. In this case, the surrounding rock deformation shall be predicted following the trend of linear acceleration. The estimation equation is expressed as:

$$u = u_0 + v_0 + kt \tag{3}$$

where v_0 is the initial deformation rate when accelerated deformation begins after the secondary support, m/d; k is the slope of accelerated deformation curve, m/d.

3. *Stability of supported tunnel* For soft and fractured rock and coal mass tunnel under the influence of tectonic belt, its stability should be evaluated, and its deformation pattern with time should be determined. According to monitoring and calculation methods, the following equation can be used to describe the tunnel stability time:

$$t = \frac{[u] - u_0}{\bar{v} - v_0} \tag{4}$$

In Eq. (4), when the tunnel deformation rate is constant, the small value of $(\bar{v} - v_0)$ indicates longer stability time. However, when the deformation rate is accelerated, the large value of $(\bar{v} - v_0)$ illustrates a shorter stability time.

4. *Optimal timing for secondary support* After rock mass is excavated and the initial support is implemented, the surrounding rock will normally deform towards the free face under the influence of higher ground stress, as shown in Fig. 10.

During the first stage, rock mass deformation rate decreases rapidly with time and reaches its minimum value at point A. Meanwhile, at the macroscopic level, the surrounding rock moves towards the excavated space at a large velocity with new fissures developing within rocks. From point A to B, the deformation curve is similar to an inclined line with an almost constant strain rate. At this stage, the surrounding rocks continue to move towards the excavated space, only at a slower convergence rate than the previous stage, and the fissures begin to expand steadily. As for the third stage, the strain rate increases rapidly once it moves past point B. When it finally arrives at point C, rock mass failure occurs and fissures will expand at an accelerated rate. Eventually the surrounding rock loses its stability.

According to the field monitoring results, it is reasonable to conduct the secondary support under the following three conditions. Firstly, the surrounding rock convergence has reached 80–90% of its maximum allowable convergence. Secondly, the convergence rate is decreasing and has reached 0.15 mm/d. Finally, the displacement rate of roof arch is less than 0.1 mm/d.

5.3 Key Technology for Fractured Rock Tunnel Control

5.3.1 Improvement of Overall Strength of the Initial Support

The initial support structure strength is not only related to bolting length and spacing, but also the structural integrity. In this case, it refers to the overall strength of the bolting-shotcreting-mesh shell structure. Generally speaking, for fractured rock tunnel in complex stress environments, grouting bolts of 2.0–2.5 m long and steel mesh for concrete spraying are used to create the initial shell support. The bolt and mortar as well as the surrounding rock are fixed by the anchoring force. The uneven surrounding rock deformation along the bolting direction generates mutual restraint among one another, thus providing bolting support for the surrounding rock.

The anchoring mechanism of full-length mortar bolt is shown in Fig. 10. It can be seen that the displacement of tunnel surrounding rock decreases
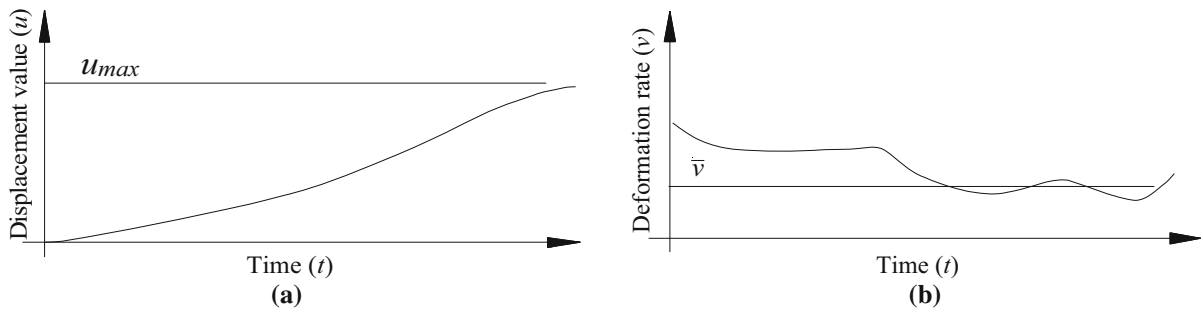
Fig. 9 Deformation curve of soft surrounding rock. **a** Deformation, **b** deformation rate

gradually with increasing distance. When full-length bonding bolts are adopted, the close contact between bolts and surrounding rock generates binding force (anchoring force). In this case, there exists a neutral point on the bolt where the shear stress is zero. Meanwhile, the shear stresses on both sides are in the opposite direction, resulting in a tension (see Fig. 11a). Besides, for full-length anchored bolts, their axial forces are somehow related to the pretension stress. For instance, the bolting axial force is the largest at the neutral point while the anchoring force is the smallest at both ends (See Fig. 11b). Therefore, the anchoring mechanism of the full-length mortar bolt is to transfer shear stress via the adhesives between bolts and the surrounding rock. Hence, to improve the overall anchoring effect of grouting bolts, high-strength rigid plates could be applied, bolting friction should be enhanced, and the bolting end structure should be improved as well. Finally, steel belt and metal mesh could be used to create a flexible shell for point-to-face force spreading.

Fig. 10 Unstable rheological process curve

5.3.2 Improvement of the Secondary Support Strength

Although the initial support strength has enhanced the tunnel stability to an extent, the tunnel deformation would still increase rapidly at the late service stage when the deformation rate becomes unstable. Since the surrounding rock has quite high stress value and the coal and rock mass are significantly fractured, the tunnel deformation can increase with elapsed time. Therefore, in order to improve the long-term tunnel stability, high-strength secondary support should be implemented. To mobilize the bearing capacity of deep and stable rock mass, a support system could be adopted by making full use of anchor cables and U-steel in practical engineering. Internally, anchor cables are used to produce anchoring force. Externally, channel steel, U-steel, flat steel and locks constitute a solid structure to provide strong support, as shown in Fig. 12. This structure greatly improves support strength, and tunnel deformation could be controlled in the early stage due to the anchor cables' active support. For the later stage of rock deformation, the U-steel support is able to play its role in a timely manner with high stiffness and yieldable features.
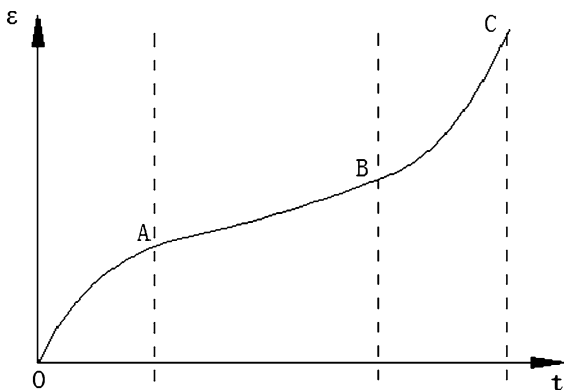
(1) Coupling effect between anchor cable and U-steel. In the early stage of tunnel excavation, deformation could be effectively controlled with active support from anchor cables of high-strength and high-pretension. While in the late stage, tunnel stability could be maintained by taking advantage of the high rigidity and yieldable features of the U-steel bracket.

(2) Internal and external bearing capacities. This combined support technique provides strong bearing capacities using internal anchor cables

and external powerful components, namely channel steel, U-steel, flat steel and locks.

5.3.3 Tunnel Section Designs Adaptive to In-Situ Rock Stress Field

The horizontal tectonic stress causes a considerable effect of extrusion damage on two tunnel wall sides. Field investigation indicates that the horizontal tectonic stress has negative effect on tunnel stability. Specifically, the cross-sectional shape could be adjusted to optimize the stress state of surrounding rocks and support structure. Theoretically, in the non-isobaric environment stress field, harmonic hole is the best shape for the tunnel, which contributes to keep the tunnel stability. The span-height ratio of tunnel surrounding rock should meet the following conditions:

$$K = \frac{a}{b} = \lambda \qquad (5)$$

where K is the span-height ratio of the harmonic hole; λ is the lateral pressure coefficient for the tunnel surrounding rock; a is the tunnel section span, m; b is the tunnel section height, m.

Equation (5) shows that the span-height ratio of the tunnel cavity should be equal to the lateral pressure coefficient.

5.3.4 Timing for the Secondary Support Based on Field Monitoring Data

Surrounding rock deformation monitoring is of great significance in the later stage of tunnel maintenance and operation. The monitoring results are important means to evaluate the effectiveness of the tunnel support scheme and parameters used. In addition, the monitoring results are helpful for optimizing support parameters and back analysis.

According to the on-site monitoring results and support timing principle, $\dot{u}_r(t)$, the deformation rate of tunnel surrounding rock should be: $0.1 \leq \dot{u}_r(t) \leq 0.25$ mm/d. Therefore, based on Eq. (6) (Yu and Gao 2012), reasonable support timing under different circumstances could be obtained, i.e. when the stress levels, excavated cross-sectional geometries and support methods are different. Based on the geological conditions and surrounding rock characteristics in Mukong coal mine, the optimal time for the secondary support should be 21–35 d after the first support applied.

$$
\dot{u}_r(t) = \frac{(1+\mu)(\sigma - \sigma_s)R_p^2}{r}
$$
$$
\times \left[\frac{1}{\eta_0'} e^{-\frac{E}{\eta_0'}t} + \frac{(1-\frac{\sigma_s}{\sigma})}{\eta_0'} e^{A(\frac{\sigma}{\sigma_s}-1)t} \left(A\left(\frac{\sigma}{\sigma_s} - 1\right)t + 1 \right) \right]
$$
$$(6)$$

where μ is the Poisson's ratio of rock mass; r is the center distance of the rock mass from any point to the excavation; σ is the current stress of rock mass; σ_s is the critical stress level of accelerating rheology; R_p is the plastic radius of the surrounding rock; η_0' is the initial viscosity coefficient when the critical stress level is σ_s; A is a positive constant; E is the modulus of elasticity of rock mass; t is time.
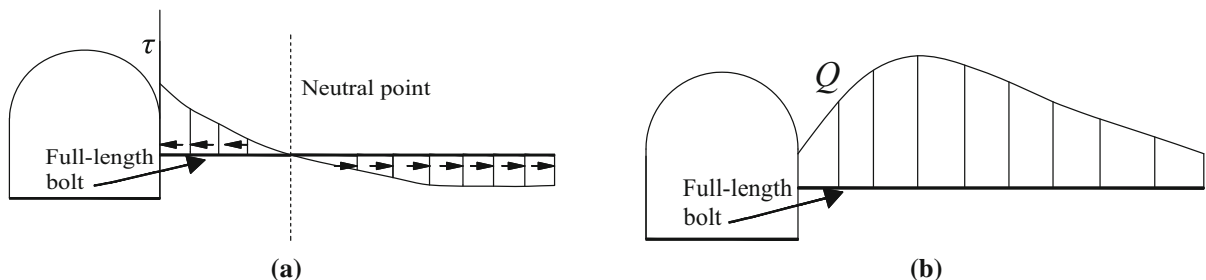

Fig. 11 Anchoring mechanism of full-length bolts. **a** Bolting shear stress curve exerted by the surrounding rock, **b** anchoring force of bolts (axial force curve)
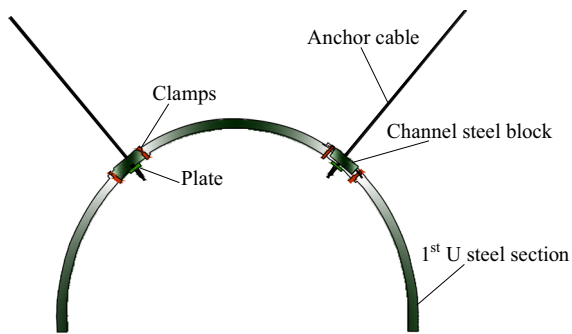
Fig. 12 Schematic diagram of complete support form

6 Field Test

6.1 Tunnel Engineering and Supporting Parameters

As for the haulage gate at #1303 fully mechanized coal mining work face in Mukong coal mine, according to the field investigations, the rocks surrounding the haulage gate are quite fractured and of low quality. As the result of stress influence, fissures within the rock mass expand rapidly and a large loose circle is formed. In this case, three support schemes were proposed and implemented in the haulage gate of #1301 mechanized coal mining work face. The on-site deformation monitoring results show that shed and truss support has a poor effect for tunnel stability. Hence, based on theoretical analysis and calculation results as well as the full demonstration by the coal mine technical staff, a combined support scheme was put forward. The core idea of the scheme is to achieve both internal and external load-bearing capacity by strengthening support for several times. Specifically, the combined support scheme uses the short bolts for full-length anchoring force, long anchor cables for high prestress and U-steel support for high bending resistance. The detailed tunnel section parameters are shown in Fig. 13.

(1) *Tunnel section design* Firstly, a margin spacing 200 mm is reserved for potential tunnel deformation. Based on the stress and deformation observations in Mukong coal mine, the lateral pressure coefficient λ =1.2–1.4 at an elevation of 600–700 m above sea level. According to Eq. (5), the condition for tunnel harmonic hole could be obtained, namely $a = (1.2–1.4)\ b$,

indicating that it is flat and oval. Based on haulage gate design parameters at #1301 work face, when semicircular arch section is adopted and when tunnel clear spacing is 4.4 m, wall height is 1.7 m, arch height is 2.2 m, clear height is 3.9 m, then $K = 1.13$. Such result does not match with the requirement that $\lambda = 1.2–1.4$, meaning that it is difficult to form a harmonic hole. To solve this problem, Eq. (5) was optimized with the same clear width and the wall height designed as 0.9–1.4 m. Furthermore, to ensure that the production requirements should be met after tunnel support and deformation, the wall height is determined as 1.4 m. In this case, the tunnel section conditions are more similar with those of a harmonic hole and the load-bearing capacity will be improved as well. At the same time, to further enhance the tunnel stability, the circular arch and high wall of those tunnels are designed to be flat and oval for longer service lifetime purpose. Therefore, semicircular arch section is adopted at haulage gate of #1303 work face with tunnel clear width of 4.4 m, wall height of 1.4 m, arch height of 2.2 m and clear height of 3.6 m.

(2) *Initial support reinforcement* Ten bolts of $\Phi20 \times 2200$ mm were used at a row interval of 800×1600 mm. Each bolt was fixed with mortar cartridges to generate full-length anchoring force. Metal mesh and steel belts were laid out across the entire section.

(3) *The secondary support parameters* Five anchor cables of $\Phi17.8 \times 7000$ mm were used at a row interval of 1200 mm $\times$ 1600 mm. Each anchor cable is installed with 3 rolls of K2335 resin cartridges. Two anchor cables of $\Phi17.8 \times 4000$ mm were used at a row interval of 1600 mm. The angle of anchor cables with the floor was around 30°–50°. Each of them is also installed with 3 rolls of K2335 resin cartridges. When rock failure occurs, U25 steel arch is used at a row interval of 800 mm. Figure 12 illustrates the connection between anchor cable and U-steel, which constitutes a integrated support system.

6.2 Deformation Monitoring after the Initial Tunnel Support and Timing Selection for the Secondary Support

In order to determine the optimal timing, the support implementation and observation are conducted for each section. The monitoring results after initial support are shown in Fig. 14. It shows that after the initial support of full-length anchorage, the inflection point of the rock deformation curve appeared after 19–32 d at Point No.1 and 13–29 d at Point No.2. Therefore, based on Fig. 7 and Eq. (6), the optimal time to start the secondary support is 21–28 d after the initial support.

6.3 Monitoring Results Analysis of Tunnel Deformation after Integral Support

The combined support scheme consists of short bolts, long anchor cables and U-steel bracket. As shown in Fig. 15a, after the secondary support, the tunnel is stabilized with limited surrounding rock deformation. After 20 days, the wall-to-wall convergence was 31.6 mm, roof-to-floor convergence was 55.1 mm and the deformation rate was about 2 mm/d. The

results show that support reinforcement for several times until both internal and external load-bearing capacities are mobilized.
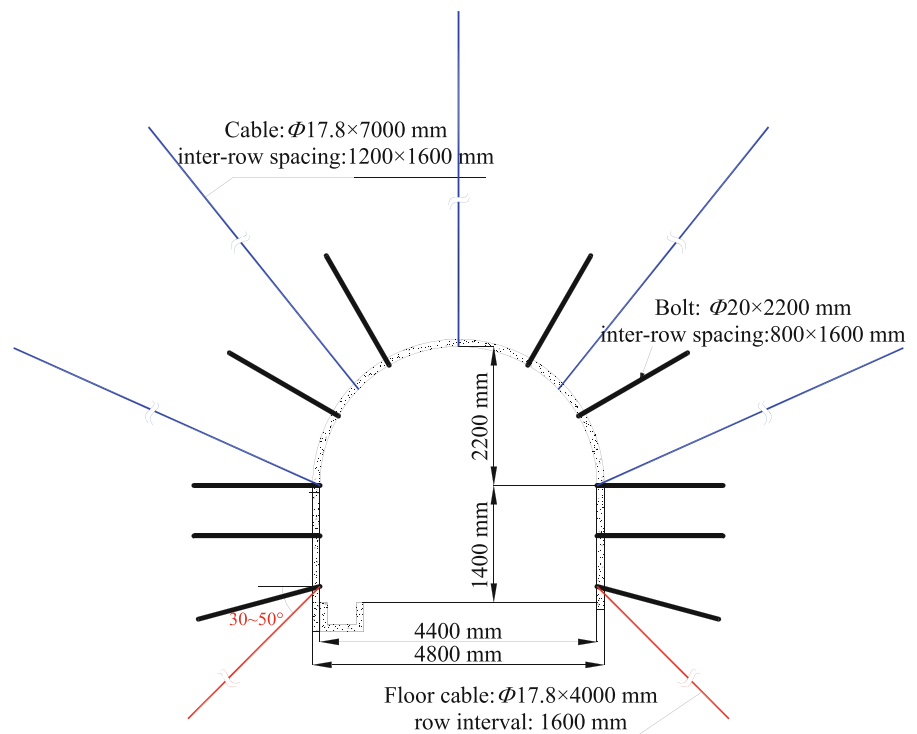
Figure 15b illustrates the force that bolts have to bear after the initial support. As monitored earlier, the surrounding rocks went observed with large deformation within the first 36 days when the bolts were continuously subjected to increasing stresses. In day 40, the secondary support had been carried out and the bolting force was gradually stabilized. This indicates smaller rock deformation in the later stage as a result of effective secondary support. In addition, the monitoring results show that a small number of bolts were reported with a low level of anchoring stresses. This indicates that the bolts' initial pretension stress was small, and the bolt plates' support resistance against the surrounding rocks increased at a slow rate.

7 Conclusions

The following conclusions could be drawn as follows:

(1) Different methods have been applied to support fractured surrounding rock in tunnels under the influence of tectonic belt, and their support

Fig. 13 Combined support parameters for the entire section

Cable: $\Phi17.8\times7000$ mm
inter-row spacing:1200×1600 mm

Bolt: $\Phi20\times2200$ mm
inter-row spacing:800×1600 mm

2200 mm

1400 mm

30~50°

4400 mm
4800 mm

Floor cable:$\Phi17.8\times4000$ mm
row interval: 1600 mm

failure modes were analyzed. The analyses show that the support structures feature by I-steel shed, U-steel yieldable support and bolt-mesh-cable combined support cannot effectively control surrounding rock deformation. The support failure often starts from the key parts and weak parts. Under the effect of the loose ground pressure, the support structures gradually lose their load capacity, thus the control effect of the entire support structure is reduced. Furthermore, rock collapse also occurs from time to time.

(2) The structural plane of the #1303 tunnel in Mukong coal mine was investigated. The results show that in the fault affected section, the spacing of fissure structural planes is 0.05–0.3 m. The integrity coefficient K_v of the surrounding rock mass is 0.12. Besides, joint fissures are quite scattered and there is a group of joints running northeast and southwest in the tunnel rock mass.

(3) After the excavation of surrounding rock is, fissures within coal and rock mass expand rapidly under the stress accumulated, in a form of tensile failure and shear plane. Due to rock swelling effect, macroscopic deformation of coal and rock mass was increased. In addition, high gas pressure inside the coal seam further promotes fissure expansion and makes the coal and rock mass looser.

(4) The deformation of the fractured coal-rock tunnel is non-uniform and the floor rock deformation is usually larger. The key factor herein is K_p, the swelling coefficient of each stratum.

(5) According to Barton Classification Q value and RMR value, the tunnel stability herein falls into the category of class V, meaning poor or extremely poor surrounding rocks, where the loose ground pressure exists. The supporting structure bears the most weight of fractured rock mass, and the surrounding rock almost loses its self-bearing capacity.

(6) The principle of integrated control technology is put forward. Firstly, the timely control of the excessive deformation of tunnel surrounding rock is emphasized and the initial support for fractured rock mass should be reinforced. Secondly, the self-bearing capacity of the surrounding rock shall be fully utilized. In addition, secondary or multiple supports should be carried out to improve the long-term tunnel stability. When it comes to tunnel design and maintenance stages, the following factors shall be taken into account, such as the allowable tunnel space, the critical deformation before tunnel collapse, the supported tunnel stability and the optimal timing for secondary support.

(7) For the fractured rock tunnel, a variety of support principles and key techniques have been proposed, including full-length anchoring support with bolts, cooperative support system with
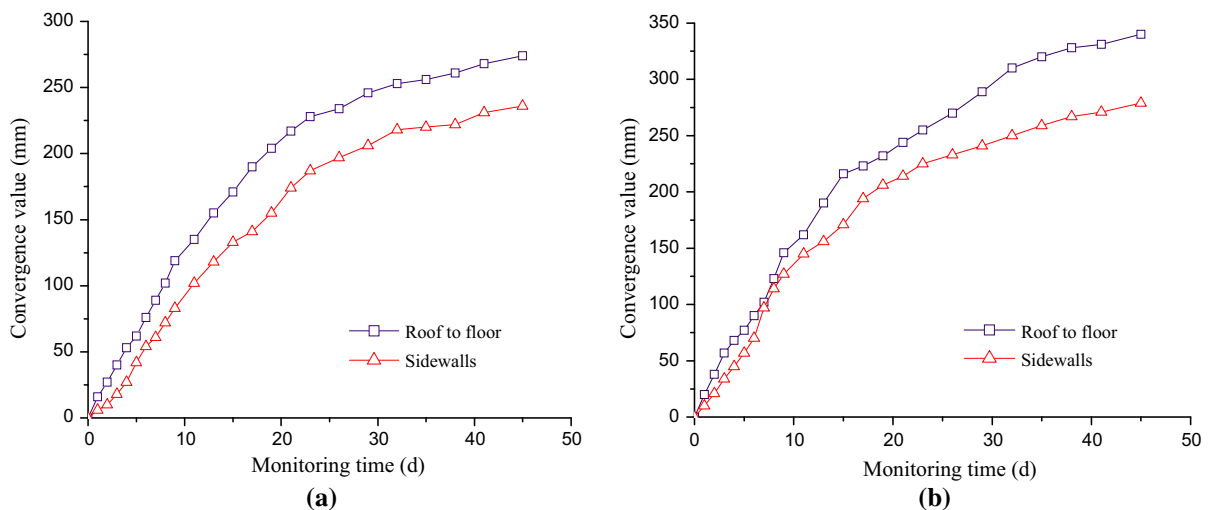

Fig. 14 Deformation monitoring after initial support. **a** Monitoring results at Point No.1, **b** monitoring results at Point No.2
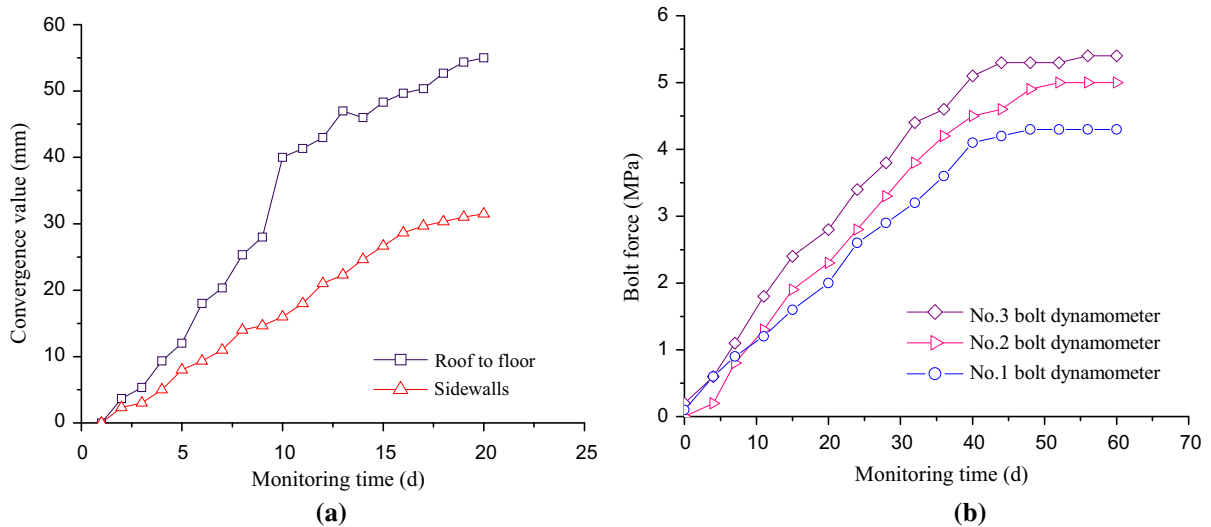
Fig. 15 Deformation Monitoring after Secondary Support. **a** Monitoring results of tunnel surrounding rock deformation, **b** monitoring results of bolt stress

U-steel and anchor cable, the tunnel cross-sectional shape optimization, span-to-height ratio of the harmonic hole, real-time support parameter optimization, and optimal timing selection for the secondary support, etc. As for the haulage gate of the #1303 fully mechanized coal mining work face of Mukong coal mine, a combined support scheme with short bolts, long anchor cables and U-steel brackets is proposed. The core idea is to mobilize both internal and external load capacities by implementing support reinforcement for several times. The monitoring results show that this support scheme is effective in control of tunnel surrounding rock deformation, which is favorable for tunnel long-time stability.

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