Original Research Article

Research on Construction Technology of Tunnel Face Slump in Soft Rock Tunnel of China-Laos Railway

WANG Kai-jun

China Railway 15th Bureau Group 3rd Engineering Co., Ltd., Chengdu 611731, China

Abstract: In Taos barrow of jade grinding rail tunnel engineering ii, of rich water karst region slipped in weak rock tunnel collapse mechanism and treatment technology for research, slipped through the establishment of the tunnel surrounding rock collapse mechanics analysis model, the theoretical calculation and finite element calculation software simulation method of combining the, constraints of the tunnel to slip down position at the beginning of surrounding rock and the structural stability analysis, According to the analysis results, the corresponding treatment measures are put forward. The results show that the settlement value of surrounding rock vault is 84.7mm, and the horizontal convergence value of arch waist is 167.4mm. The deformation value of surrounding rock is large, and it is easy to collapse. The plastic zone of surrounding rock mainly occurs at the junction of upper and lower steps on the face of the face, and the maximum plastic strain is 6.63×10^{-2}. The arch waist of the primary support structure is prone to stress concentration, which should be reinforced during construction to prevent instability failure. According to the actual situation of tunnel construction site, combined with the finite element calculation results, the treatment structure of concrete filling combined with small pipe reinforcement was used to treat the tunnel slide collapse section. The treatment effect of tunnel slide collapse section was better, which provided a reference for the treatment of the same situation in the subsequent construction section.

Keywords: Soft rock tunnel; Water-rich karst; Landslide treatment; Finite element simulation; Construction technology.

1. Introduction

China is one of the countries with wide distribution of karst landform, mainly concentrated in southwest Yunnan. With the rapid development of China's high-speed railroad construction, railroad transportation facilities are continuously extended to these areas, which are mostly traversed by long tunnels due to the influence of karst geologic terrain and other conditions. The weak surrounding rock, water-rich karst and other adverse geological conditions have increased the difficulty and safety risk of tunnel construction. Water-rich karst soft rock tunnels, the surrounding rock is weak and broken, low strength and poor stability, and under water-rich conditions, the surrounding rock is very easy to plastic damage by the excavation disturbance, due to the complex geological conditions, the weak surrounding rock tunnels are very prone to slipping and collapsing and other accidents during the construction process.

At present, the domestic experts and scholars on tunnel support and landslide treatment measures are more widely studied, the literature combined with engineering examples, analyzed the specific reasons for the collapse of high geo-stress soft rock tunnels, put forward specific measures to deal with, and evaluated the effect of the treatment through the relevant specifications and finite element simulation. Literature according to the construction process of tunnel collapse development process and collapse mechanism, put forward and implement a set of collapse comprehensive treatment measures, through the analysis of comprehensive measures after the implementation of the surface settlement, arch settlement and arch waist convergence and other rules

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of change, concluded that the comprehensive measures can effectively manage the soft rock tunnel collapse accident. Taking the Lan Yu Railway as the engineering background, the literature analyzes the causes of tunnel landslide accidents and proposes the treatment measures for landslide situations, and the comparative analysis of numerical simulation and on-site monitoring results proves the effectiveness of the treatment measures, and the effect of the treatment meets the engineering requirements. The literature analyzes the causes of tunnel landslides, and gives targeted landslide treatment measures, while using numerical simulation and monitoring and measuring means to analyze the effectiveness of landslide treatment, and finally concludes that the treatment measures can ensure the safety of tunnel construction, and can effectively avoid secondary landslides in the subsequent construction. Based on a tunnel cave-in project, the literature analyzes the main causes of tunnel collapse, proposes comprehensive treatment measures combining overrun pipe shed and grouting small conduit, and joint treatment of cave-in and surface, and analyzes the treatment effect in detail through the combination of on-site measurements and numerical calculations.

At present, although the research on tunnel collapse treatment measures is more extensive, there are relatively few studies on tunnel collapse treatment measures for carbonaceous mudstone strata in water-rich karst areas. This paper relies on the Baro No. 2 Tunnel Project of Yuma Railway Section of China-Laos Railway, establishes a tunnel model of charcoal mudstone stratum in water-rich karst area through finite element software, analyzes the mechanical characteristics of the surrounding rock and the primary support structure in the tunnel sliding collapse position as well as the tunnel sliding collapse mechanism, and proposes a special management plan for the actual sliding collapse situation, and makes reference to the similar situation for the subsequent construction sections.

2. Project Overview

China's "The Belt and Road" important project, China-Laos Railway, Yuma Railway section is located in China's southwest Yunnan Province, of which the Baro No. 2 Tunnel is a single-line railroad tunnel with a design speed of 160km/h, from the starting mileage of the section, D1K435+380, to the exit mileage, D1K439+280, the length of the tunnel is 3,900m, and the maximum depth of the tunnel is 260 m. The area where the tunnel is located is the Zhongshan landform, the ground elevation is 625m~925m, and the maximum height difference is 300m. The area where the tunnel is located is Zhongshan landform, with the ground elevation of 625m~925m, the maximum height difference of 300m, the natural cross slope of 50~450, locally steeper, the development of natural shallow ditches between the mountains, undulating topography, the overlying soil layer is thin, and the bedrock is partially exposed.

The tunnel crosses the Permian Upper Longtan Group mudstone sandstone, carbonaceous shale, lithological space staggered spread, low strength of the surrounding rock, joints and fissures developed, the surrounding rock as a whole is more broken, integrity and stability of the poor, the surrounding rock by the construction of the construction disturbances are very prone to fall blocks, slipping and collapse and other engineering problems. The tunnel is located in the area due to high precipitation, rich groundwater, low strength and broken peripheral rock, the tunnel line on the left side of the existence of downward bias, extrusion folding phenomenon is obvious, the peripheral rock by the structural surface cutting, local gravel gravel loose structure, the construction process is very easy to fall blocks and slipping collapse phenomenon

3. Reason and Mechanism Analysis of Sliding Collapse

3.1 Indicator System Construction

Barrow No. 2 tunnel construction mileage DIK437 + 662 in the construction section, the rock layer to carbonaceous shale is mainly local clamped mudstone, rock fragmentation, groundwater is more developed, the palm face at the groundwater can be seen in the form of seepage droplets out of the charcoal shale by the water-soaked rock is softer and more loose, very easy to fall blocks and sliding collapse. Mileage DIK437+662 palm face slag completion, in the slag raking process palm face right shoulder suddenly appeared to fall block phenomenon, after the palm face right arch began to slip and collapse, until the entire palm face is almost completely closed (see Figure 2 shows), slip and collapse of the composition of the body is mainly for the charcoal mudstone, and seepage of water into the line. As a result of timely evacuation, the collapse did not cause casualties and no equipment damage. The cross-section of the tunnel collapse and the site pictures are shown in Figure 1 and Figure 2.

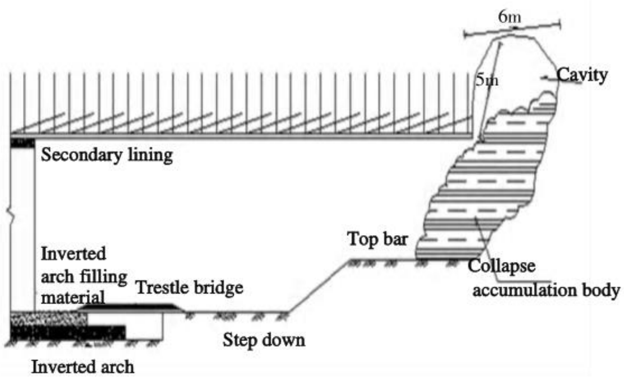


Figure 1 Barrow tunnel no. 2 slip and fall cross section.

Figure 2 Barrow II tunnel slip and fall site figure.

There are two main reasons for the tunnel collapse in Barrow No. 2 Tunnel. On the one hand, due to the tunnel passing through the stratum of charcoal mudstone, the surrounding rock is relatively soft, rich in groundwater, and the possibility of destabilizing and destroying the surrounding rock is higher in the local section that may form an irregular, fragmented, sandy and mud-containing water-rich body. On the other hand, due to the lag and inaccuracy of the over-advance geological forecast, the tunnel failed to make accurate and effective prediction of the geological condition in front of the palm face, which led to the failure to take effective measures in advance to prevent disasters from occurring during the construction.

3.2 Sliding Mechanism of Surrounding Rock

In the process of tunnel construction, the original ground stress field is disturbed and redistributed, and the surrounding rock that needs to be supported and reinforced to be stabilized is called shallow surrounding rock, and the surrounding rock that has good integrity and can maintain stability without support and reinforcement is called deep surrounding rock. Shallow surrounding rock from relaxation to the development of discrete, the surrounding rock will be destabilized, in the relaxation stage of the shallow surrounding rock and the initial support together to bear the deformation pressure, the support structure is mainly subjected to plastic pressure. Shallow surrounding rock with the deformation of the plastic zone of the tunnel surrounding rock continues to increase and eventually reach the discrete stage, at this time the shallow surrounding rock due to pressure loosening, under the action of its own gravity to slip and collapse or the formation of the supporting structure of the load. From Kastner's formula, the relationship between the maximum support reaction force p_i : and the displacement of the surrounding rock v_0 can be expressed as follows:

$$R_p = R_0\left[\frac{(P_0 + a\cos\varphi)(1-\sin\varphi)}{c\cos t\varphi}\right]^{\frac{1-\sin\varphi}{2\sin\varphi}} \quad \text{(Eq.1)}$$

$$P_1 = (P_0 - \cos t\varphi)(1-\sin\varphi)\left(\frac{R_0}{R_p}\right)^{\frac{2\sin\varphi}{1-\sin\varphi}} - csost\varphi \quad \text{(Eq.2)}$$

$$V_0 = \frac{\sin\varphi}{2GR_0} R_P^2 (P_0 + c\cot\varphi) \quad \text{(Eq.3)}$$

In the Eq.(1) (2) (3): R_0 is the radius of the tunnel surrounding rock; P_0 is the initial ground stress, a is the external collapse angle of the rock body; c and φ are the cohesion and internal friction angle of the surrounding rock, respectively; R_p is the radius of the plastic zone of the surrounding rock; P_i is the support reaction force; and G is the sliding collapse weight force.

Calculating the maximum support reaction force and perimeter rock displacement based on Eq.(1) to (3) plays an important role in the design of tunnel support structure and effective prevention of tunnel sliding collapse during tunnel excavation.

4. Numerical Simulation

4.1 Model Parameter

According to the geological survey report of the Barrow II Tunnel, and according to the "Railway Tunnel Design Code", the mechanical parameters of the tunnel surrounding rock and initial support structure are shown in Table 1.

Table 1 Mechanical parameters of surrounding rock and supporting structure.

Name	Surrounding rock weight/(kN/m²)	Modulus of elasticity/MPa	Poisson force μ	Cohesive force /kN/m²	Friction Angle/(∘)
Surrounding rock	20	50	0.45	80	20
Primary support	25.0	26	0.30	_	_
Anchor bolt	78.0	20	0.27	_	_
Tubule	33.9	90.2	0.30	_	_

4.2 Coupling Model

In order to effectively analyze the instability and failure mechanism of surrounding rock during tunnel excavation, the finite element software MIDAS-GTS NX was used to establish a tunnel excavation construction stage model. According to the principle of Saint-Venant XYZ, the tunnel diameter was about 3 times in each direction. The distance between the tunnel center and the boundary in the X and Z axes was 50m, and the excavation direction in the Y axis was 60m. The upper and lower left and right surfaces of the model are non-free, so the displacement in vertical direction and horizontal direction are constrained. The tunnel construction method is step method; φ42 small pipe advance support, longitudinal spacing of 3.0m, each ring 17, the length of a single small pipe 4.5m; The grille steel frame is used to strengthen the support, and the steel frame spacing is 1.5m. The tunnel construction stage model is shown in Figure 3.

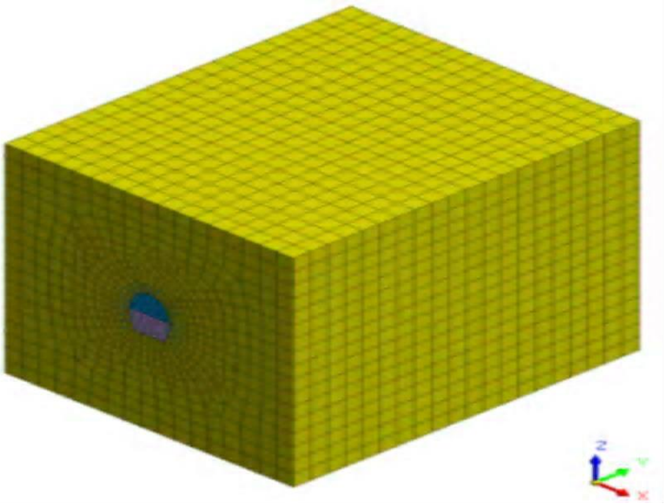
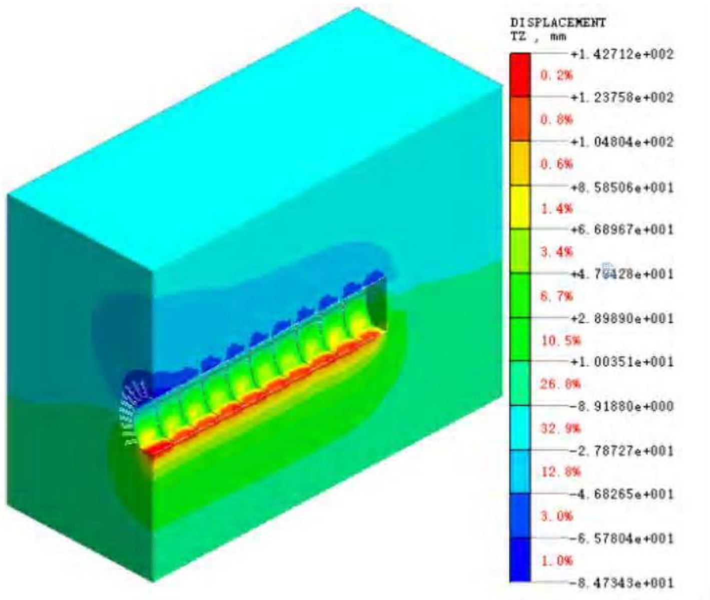


Figure 3 Tunnel construction stage model.

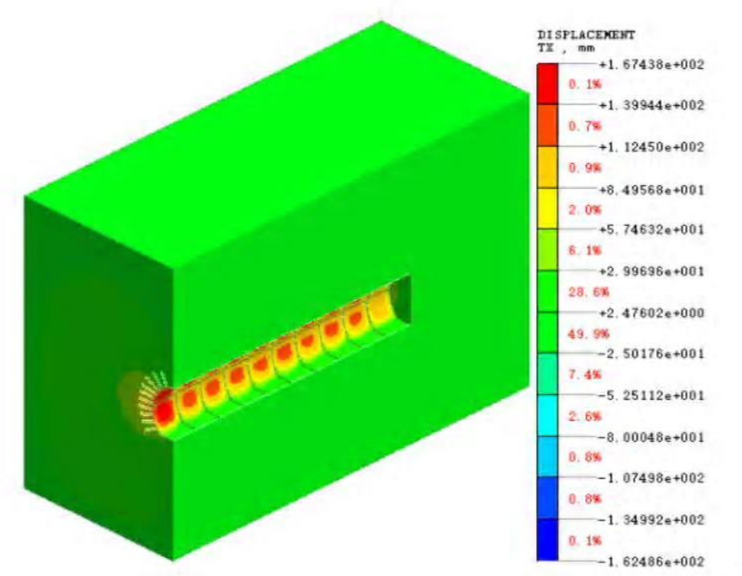
5. Analysis of Results

5.1 Deformation Analysis of Surrounding Rock

Through the finite element simulation to analyze the tunnel construction stage, it can be seen that after the tunnel construction started, the original structure of the surrounding rock was disturbed by the excavation, and the rock strength of the carbonaceous mudstone surrounding rock was low, and the mechanics of the flat and horizontal were damaged and began to sink significantly, which eventually caused greater pressure on the tunnel initial support structure. The deformation cloud diagram of surrounding rock is shown in Figure 4, from Figure 4(a), it can be seen that the maximum value of vertical subsidence of surrounding rock is 84.7mm, in addition to the location of the tunnel's elevated arch by the surrounding surrounded by the surrounding surrounding rock extrusion after the bulge obviously, the maximum value of the elevated arch bulge is 142.7mm, and from Figure. 4(b) it can be seen clearly, the tunnel arch girdle convergence at the location of the girdle convergence of the arch girdle convergence of the maximum value of 167.4mm, according to the results of the finite element simulation, it can be seen that The deformation of the tunnel surrounding rock is large, causing large pressure on the primary support structure, so it is necessary to add weeks to the primary support structure and strengthen the deformation monitoring of these locations.



(a) Horizontal convergence of surrounding rock



(b) Vertical displacement of surrounding rock

Figure 4 Deformation cloud map of surrounding rock.

5.2 Analysis of the Plastic Zone of The Surrounding Rock

The distribution of the plastic zone of the tunnel surrounding rock is shown in Figure 5. As can be seen from Figure 5, the plastic zone of surrounding rock is mainly concentrated at the upper and lower steps of the palm face, the arch waist and the super-elevation arch position. The maximum value of plastic strain at the position of arch waist and back arch of the tunnel is $6.63 \bullet 10^{-2}$, and the distribution of the plastic zone can be seen from the distribution map.

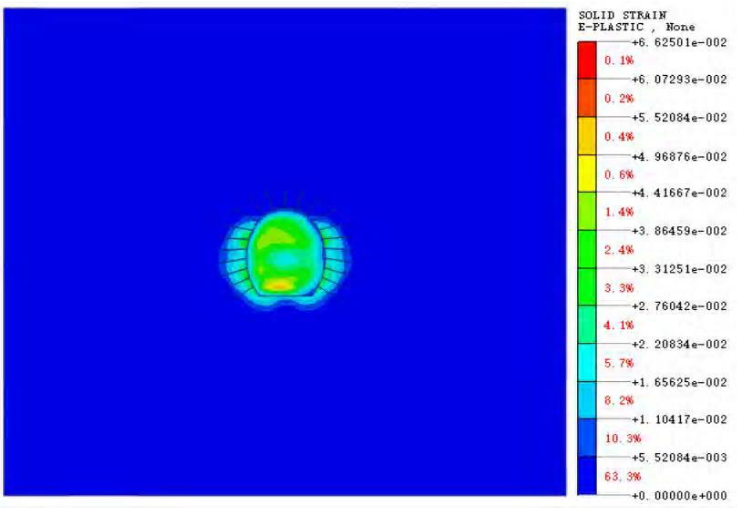
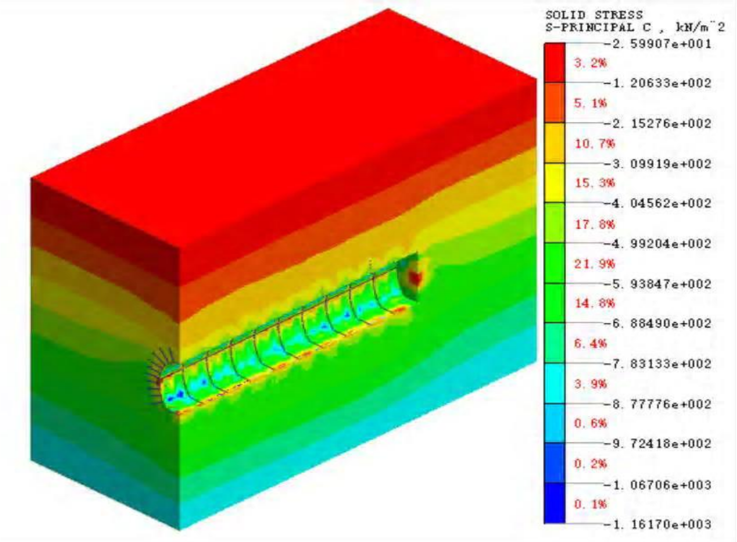


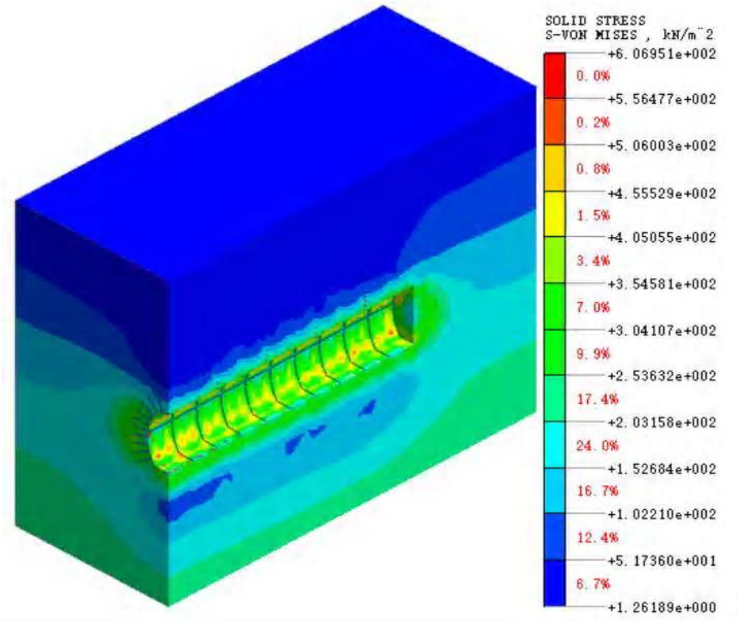
Figure 5 Analysis of the plastic zone of the surrounding rock.

5.3 Stress analysis of surrounding rock

The cloud diagram of surrounding rock stress distribution is shown in Figure 6. Through the finite element calculation of the tunnel construction stage, it can be seen that the surrounding rock of the tunnel was affected by the disturbance of the construction excavation, the stress was redistributed, the original mechanical structure of the surrounding rock was damaged, and the surrounding rock around the tunnel palm face showed stress relaxation phenomenon. As can be seen from Figure 6, the maximum principal stress and von Mises stress of the surrounding rock appeared at the junction of the upper and lower two steps, so the stress concentration occurred in the relative position of the initial support structure, which would cause fatigue cracks in the initial support structure, and due to the low strength of the carbonaceous mudstone rock, the bearing capacity is poor, and it is easy to lead to the occurrence of the sliding of the surrounding rock under the joint effect of the excavation disturbance of the palm face and the softening of the groundwater. Therefore, it is necessary to reinforce the locations where the initial support structure is prone to stress concentration to prevent damage.



(a) Cloud map of maximum principal stress of surrounding rock



(b) Von Mises distribution cloud map of surrounding rock stress

Figure 6 Stress distribution cloud map of surrounding rock.

6. Treatment measures and effects of sliding collapse

6.1 Treatment measures of sliding collapse

Combined with the construction site sliding collapse condition and numerical simulation results, for the tunnel palm face sliding collapse location to put forward targeted measures, the specific measures are as follows:

(1) First of all, the size of the cavity formed in the right shoulder of the palm face due to sliding collapse and the lithology of the accumulation body were judged, and at the same time, the cavity of the palm face at the location of the collapse was probed, so as to accurately grasp the conditions in the cavity.

(2) Rapidly on the surface of the pile of body spray anchor closed, standard C25 spray anchor thickness of 10cm, (see Figure 7) to ensure that the pile of body stabilization no longer collapsed and slipped, while ensuring the safety of the lower personnel and machinery and equipment.

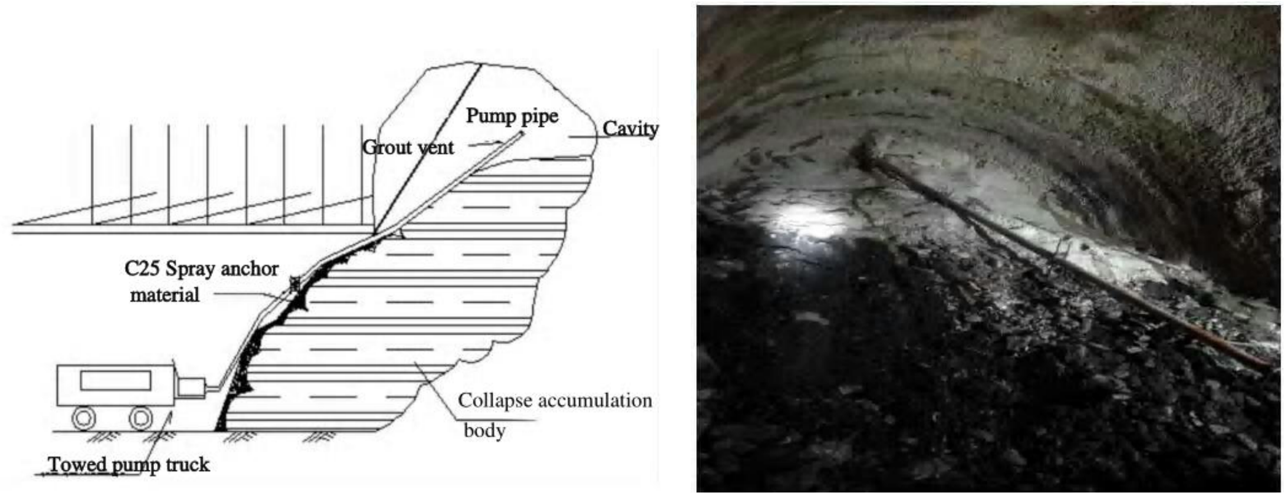


Figure 7 Spray anchor diagram.

(3) Comprehensive analysis of the collapse situation, to determine the overrun support measures. 042 wall thickness of 3.5mm overrun small conduit support effective distance is not more than 3m, and the collapse of the body caused by the longitudinal maximum distance of the cavity is 6m, it is not suitable to use the 042 small conduit as an overrun support measures. If 0108 or 089 pipe shed as overrun support, overrun support distance can be up to about 30m, but the pipe shed construction period is longer, the cost is larger. Comprehensive comparison decided to use the following measures: in the 50cm gap between the surface of the pile body and the vault reserved 0125 trailing concrete pump pipe and 042 small conduit, to ensure that the small conduit top to the highest part of the cavity. At the same time in the highest part of the small conduit pipe wall reserved overflow holes, the purpose is to check the concrete pump pipe pumping concrete extrusion to fill the entire cavity (see Figure 7, 8 shows). After pre-burying the pipe, the gap between the top of the arch and the surface of the pile is closed with C25 spray anchor material to ensure that the gap is firmly sealed. Then fix and stabilize the pump pipe on the pile, connect it to the trailing concrete pump, and press the C20 concrete into the sliding cavity through the concrete pump, and at the same time, observe the overflow orifice to ensure that the concrete fills up the whole cavity.

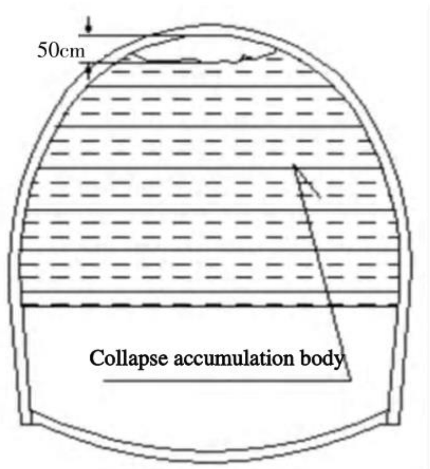


Figure 8 Schematic diagram of slough deposits.

(4) Slippery collapse cavity is filled with concrete, the cavity in the air surface disappeared, palm face in

front of the first branch along the contour line to form an irregular concrete arch circle, concrete arch circle thickness of about 1~2.5 m. Thus, the entire slippery collapse location of the surrounding rock reinforcement, enhance the stability of the surrounding rock, concrete arch circle is shown in Figure 9.

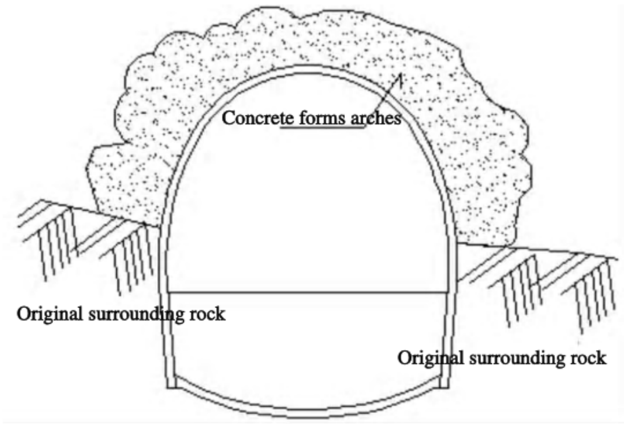


Figure 9 Concrete rolling circle diagram.

(5) After the concrete reaches a certain strength, it is constructed according to the step method. According to the finite element software analysis results to take the following measures: the use of the original design of the arch to increase the strength of the 118 steel arch, the spacing is adjusted to 1 m, the arch foot of the upper and lower steps using Ø42 double-locking angle anchor pipe, the length of 4 m. After the completion of each excavation, before the erection of the frame of the exposed perimeter rock and the palm surface of the spray anchor closure to ensure that the perimeter rock is safe and stable. The overrun small conduit support is applied first, and then excavation is carried out to set up the frame. One joist for each advance, spray anchor support in the lower part of the concrete arch ring, forming a lining together (see Figure 10). The upper step is about 5m in length and then follow up the lower step, the left and right sides of the lower step are staggered by 6~8m, and the excavation is carried out unilaterally. At the same time to ensure that the elevation arch in time to follow up, the second lining lining in time to close into a ring. Until the whole construction paragraph all through the weak surrounding rock section.

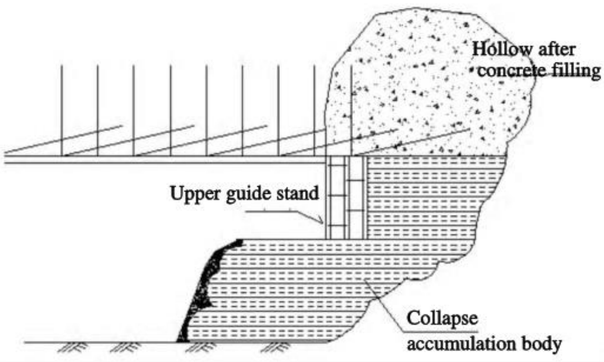


Figure 10 Excavation diagram of mixed deposits.

(6) Due to the slipping and collapsing position construction section is rich in groundwater and broken peripheral rock, after using concrete arch ring support and the original peripheral rock density difference is large, easy to form waterproof layer, behind the formation of water pressure. After the completion of the initial support spraying and anchoring in the concrete arch ring parts, along the ring and longitudinal intervals at a certain distance from the Ø50 drainage decompression holes, to ensure the stability of the arch ring. At the same time, the frequency of monitoring and measuring should be strengthened, paying close attention to the convergence of the surrounding rock and changes in settlement to provide accurate information for the disposal of the site.

6.2 Treatment Effect of Sliding Section

After the initial support construction of 611T sliding collapse section was completed, the monitoring and measuring section of the sliding collapse position DK437+662 and the initial support of the subsequent construction section was increased to 10m each, to ensure that the monitoring and measuring points were safe and firm, and the monitoring frequency was at least 2 times/d, and the deformation of the treatment arch top and the arch waist position was monitored using the full station instrument. After applying the above construction measures to treat and reinforce the surrounding rock at the sliding collapse location, the cumulative deformation measured data of the sliding collapse location and the surrounding rock of the subsequent construction section are shown in Table 2. As can be seen from Table 2, after the treatment with the above construction measures, the values of the settlement of the vault and convergence of the arch waist of the peripheral rock at the slipping collapse section are small, and they are all in the safe range, which indicates that the reinforcement effect of the above treatment measures is obvious.

Table 2 Cumulative deformation data of surrounding rock.

Stake mark	vault crown settlement/(mm)	Arch convergence/mm
D1K437+662	11.2	17.3
D1K437+672	9.2	11.7
D1K437+682	12.4	14.6

7. Conclusion

Based on Yunnan Yumo Railway Baro No.2 Tunnel Project, through finite element simulation of tunnel construction stage, slippage section displacement and other deformation, the tunnel palm face above the peripheral rock slippage mechanism is analyzed, and palm face slippage treatment measures are proposed, and the following conclusions are obtained.

(1) Through geological investigation, we can grasp the lithology of the sliding pile and the development of groundwater at the location of the tunnel sliding collapse, analyze the causes of the sliding collapse at the palm face, and determine the lithology of the surrounding rock and the weaker parts of the surrounding rock stability in the sliding collapse section, so as to provide a reference basis for the treatment of the sliding collapse at the palm face of the subsequent construction section.

(2) According to the geological exploration data of the sliding collapse section and the analysis of the sliding collapse mechanism, it can be seen that there is groundwater seeping out from the location of the tunnel palm face, the strength of the surrounding rock is lowered by water immersion, and the primary support structure is subjected to greater pressure on the arch top and the arch girdle after destabilizing and destabilizing the surrounding rock, coupled with the disturbance of the construction excavation which ultimately led to the sliding collapse of the tunnel.

(3) Through the finite element software, the simulation of the tunnel construction stage, there are simulation results can be seen, the tunnel arch and arch waist position at the peripheral rock deformation value is larger, and the peripheral rock plastic zone is mainly concentrated in the distribution of the arch waist position, indicating that the peripheral rock stability is poor, in the excavation process occurs sliding collapse risk is greater.

(4) According to the specific situation of slipping and collapsing at the site, adopt safe and efficient slipping and collapsing disposal measures, that is, to ensure the quality of the project and guarantee the target of construction period.

(5) Based on the actual situation and combined with the finite element simulation results, it is proposed to form an arch ring through concrete grouting to enhance the strength of the original peripheral rock, and then use a small catheter to reinforce the supporting structure of the treatment measures to accurately control the excavation footage, reduce the construction of the peripheral rock perturbation, and ensure the safety of the

tunnel sliding section of the excavation construction.

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