



Research on Tunnel Detection and Collapse Treatment in Karst Fissure Development Areas

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Abstract. In China, soluble rock formations are widely distributed, and the unique physical and mechanical properties of filled karst geology are formed under special environmental conditions and geological forces. With the large-scale construction of tunnel projects in karst areas, the issue of surrounding rock collapse disasters and their prevention and control has become increasingly prominent. This paper takes the karst geology of the Liangmaoshan Tunnel on the Rongfu Expressway in Guangxi as the research object. By comprehensively using geological mapping, advanced drilling, microseismic geophysical exploration, and other detection methods, combined with numerical simulation, theoretical analysis, and field measurement techniques, the study investigates the microseismic detection and prediction image analysis of karst and fractured bodies of different scales and spatial positions. For large-scale karst areas, a comprehensive treatment plan of "clay backfilling for waterproofing at the tunnel roof + fine sand filling + foam concrete pumping + advanced large pipe roof + radial grouting" is proposed. The effectiveness of this treatment plan has been verified through subsequent tunnel monitoring, ensuring the safety of tunnel construction.

Keywords: Tunnel engineering, Karst fissures, Microseismic monitoring, Roof collapse, Prevention and control measures

1 Introduction

Since the 14th Five-Year Plan, transportation infrastructure in the southwestern region has developed rapidly^[1]. Tunnels with complex geological conditions and high construction difficulty frequently appear. Karst (also known as limestone) is widely distributed in the southwestern region^[2], and its unique geomorphological environment and geological conditions provide the necessary conditions for disasters such as collapses and roof falls. When tunnels pass through karst fissure development areas, roof fall accidents can occur due to insufficient advanced geological prediction, inadequate geological survey depth during the design phase, and unreasonable response measures during the construction phase^[3]. To ensure tunnel construction safety, it is of great significance to study the causes and control of tunnel collapses and roof falls in karst fissure development areas.

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K. Yuen et al. (eds.), *Proceedings of the 2025 International Conference on Engineering Management and Safety Engineering (EMSE 2025)*, Advances in Engineering Research 267,

https://doi.org/10.2991/978-94-6463-780-9_50

Domestic and international tunnel scholars have analyzed the mechanisms of tunnel roof falls in various regions and obtained relevant research results. Wang Changjin^[4], based on the Donghuangmiao Tunnel roof fall collapse disaster, pointed out that the tunnel roof fall was influenced by rock layer structure and rainy weather, and proposed measures such as steel pipe grouting. Zou Jinjie et al.^[5], based on the basic principles of the "shallow buried tunneling method," proposed that the key to tunnel roof fall control lies in the selection of auxiliary construction methods. Hu Qiang et al.^[6], through on-site investigation and data analysis of a weathered granite tunnel collapse, found that the varying degrees of granite weathering caused strong mutations in the tunnel surrounding rock, which was the root cause of the tunnel collapse. Jia Housheng et al.^[7], to explore the frequent causes of roof fall accidents, used numerical simulation methods to analyze the main stress of surrounding rock in the Baode mining area, revealing the intrinsic relationship between changes in the surrounding rock stress field and roof fall disasters. From the above research results, it can be seen that the causes of tunnel collapses and roof falls are complex and varied, influenced by geological conditions, excavation methods, and support structures during tunnel construction. When large-scale karst fissure development areas are encountered, the treatment methods still need to be studied in depth based on relevant engineering cases.

Therefore, this paper takes the severe roof fall collapse accident in the Liangmaoshan Tunnel on the Rongfu Expressway in Guangxi as an example. Through geological mapping and geophysical exploration methods, the key causes of the roof fall disaster are analyzed, and corresponding treatment measures are summarized. The effectiveness of the proposed comprehensive treatment plan is verified through on-site convergence monitoring, providing a reference for similar projects.

2 Project Overview

2.1 Engineering Geological Conditions

The main line of the Rong'an to Yongfu Expressway generally runs northwest-southeast, passing through Rong'an County in Liuzhou City and Yongfu County and Lingui District in Guilin City. The total length of the project route is 111.517 km, constructed according to the standards of a two-way four-lane expressway, with a design speed of 100 km/h, an overall roadbed width of 26 m, and a separated roadbed width of 2×13 m.

The proposed Liangmaoshan Tunnel is located in Yongfu County, passing through nearby mountains, as shown in Figure 1. The tunnel area belongs to the karst peak cluster landform. The terrain is undulating with deep cuts. The entire mountain is composed of Upper Devonian limestone, with thin Quaternary cover layers in valleys and ridges. The main aquifers are karst fissures and caves in the limestone. Due to the differential dissolution, the water-bearing space and its distribution tend to be uneven, often causing severe surface water shortage in karst areas while groundwater is abundant at depth.

The left tunnel of Liangmaoshan Tunnel is 1431 m long with a maximum burial depth of 242 m, and the right tunnel is 1345 m long with a maximum burial depth of 1224 m. The V-grade surrounding rock is 394 m, accounting for 14.3% of the total

surrounding rock length; the IV-grade surrounding rock is 825 m, accounting for 30%; and the III-grade surrounding rock is 1535 m, accounting for 55.7%. The maximum continuous lengths of V-grade, IV-grade, and III-grade surrounding rock are 99 m, 365 m, and 500 m, respectively. At K0+098 and K1+030, geophysical exploration data suggest the presence of karst fissure development zones, where the surrounding rock integrity is poor and the rock mass is fractured, making the tunnel prone to medium to large collapses during excavation.

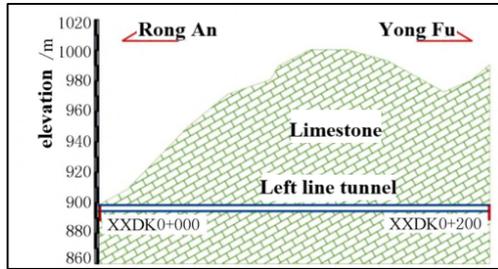


Fig. 1. Schematic diagram of tunnel section

2.2 Tunnel Support Design

The support parameters for the III-grade surrounding rock tunnel are as follows: 100 cm × 120 cm (longitudinal × circumferential) staggered arrangement of 3.0 m long $\phi 22$ mm resin bolts, 25 cm × 25 cm $\phi 6.5$ mm steel mesh, 15 cm thick C25 early strength shotcrete, EVA waterproofing board (with non-woven fabric), and 45 cm thick cast-in-place C30 concrete, as shown in Figure 2.

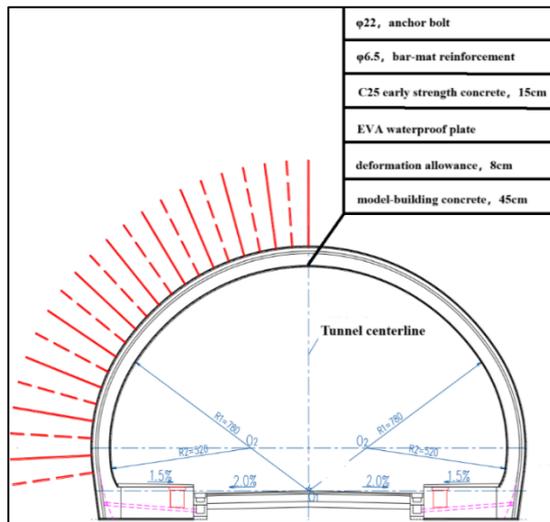


Fig. 2. Class III Surrounding Rock Tunnel Support Diagram

3 Tunnel Collapse Disaster Overview

The tunnel section K0+060~K1+031 was originally designed for III-grade support and constructed using the double-step method, as shown in Figure 3. On October 24, 2023, during the excavation of the upper bench to K0+073, after the initial support was applied, a local rock mass instability collapse occurred at the top right 2 m of the tunnel during the next process (advanced small pipe/ blasting hole drilling/advanced exploration hole). The collapse expanded, naturally accumulating at the face, forming a cone-shaped body of about 250 m³, which is a large-scale collapse (>100 m³). This caused the advanced small pipes to completely fail and partially damaged the steel arch at the crown and right arch waist, but no casualties occurred. The collapse at the site is illustrated in Figure 4.

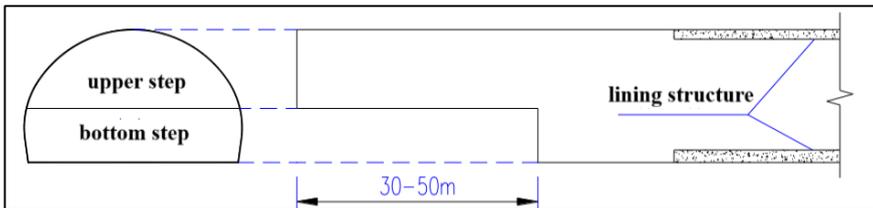


Fig. 3. Construction sequence diagram of Class III surrounding rock tunnel

4 Collapse Treatment Plan and Effect Evaluation

4.1 Collapse Treatment Plan

(1) Use horizontal drilling to determine the scale of the filled karst and the state of the filling material.

(2) Within a 160° range of the tunnel arch, alternately install 6 m and 4 m long grouting pipes with a circumferential spacing of 20 cm. The long and short grouting pipes have external insertion angles of 15°~20° and 20°~25°, respectively. Use cement-sodium silicate double-liquid grout to reinforce the rock and soil. The grout consists of 42.5# Portland cement slurry with a water-cement ratio of 1:1, mixed with 5% sodium silicate, and a grouting pressure of 0.5~1.0 MPa.

(3) After the grouting reinforcement reaches the design strength, use a double-layer steel arch frame with a longitudinal spacing of 0.5 m, combined with 6 m and 4 m long grouting pipes and systematic bolt support for the filled karst section behind the K0+073 face. The outer arch uses I22a steel with a double-layer steel mesh, extending 5 m longitudinally. The inner support structure uses the V-grade surrounding rock shallow burial reinforcement parameters. After construction, perform grouting reinforcement again using 42.5# Portland cement slurry with a water-cement ratio of 1:1, mixed with 5% sodium silicate, and a grouting pressure of 0.5~1.0 MPa.

(4) Use the three-bench method with a reserved core soil and short advance.

(5) After passing through the filled karst area, promptly use geophysical exploration or digital borehole imaging to determine the scale and filling of the cavity outside the tunnel support structure. If the thickness of the loose rock and soil on the support structure is ≥ 6 m, perform cement-sodium silicate double-liquid grouting to reinforce the rock and soil within 6 m of the support structure, improving its stability and bearing capacity to buffer the impact of subsequent collapsing rock and soil on the tunnel support structure. Otherwise, depending on the thickness of the loose rock and soil on the support structure, perform double-liquid grouting, pump-filled foam concrete, and fine sand or fly ash filling from the inside out. The collapse response at the site is illustrated in Figure 5.



Fig. 4. Real shot of tunnel surrounding rock collapse

4.2 Collapse Treatment Effect Evaluation

According to the progress of the Liangmaoshan Tunnel collapse treatment and on-site conditions, monitoring instruments were installed in a timely manner. Representative monitoring sections were selected to measure the tunnel perimeter convergence deformation, contact pressure between the initial support and secondary lining, concrete stress, and steel bar axial force. Based on on-site construction conditions, perimeter convergence deformation monitoring began on November 5, and pressure cells, concrete strain gauges, and steel bar axial force gauges were installed on December 20. The data were collected and plotted according to the "Highway Tunnel Construction Technical Specifications," as shown in Figure 6.

The tunnel perimeter convergence deformation is a macroscopic indicator of the initial support structure's effectiveness and surrounding rock stability. The deformation development went through four stages: slow growth, rapid growth, slow growth and stabilization, and temporary stabilization. The maximum crown settlement was 37.81 mm. The stabilization of the tunnel perimeter convergence deformation indicates that the double-layer steel arch frame provided good support, and the collapse treatment measures achieved initial success. After nearly six months of monitoring, it was found that the collapse treatment measures effectively eliminated the threat of filled karst to tunnel stability, and the treatment technology and control experience were successful.

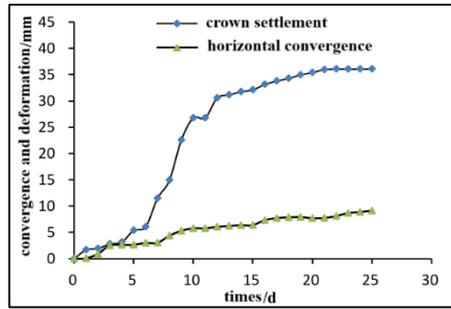
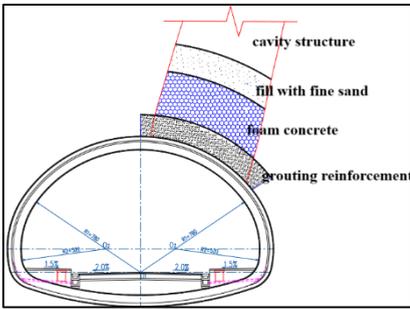


Fig. 5. Control of tunnel surrounding rock collapse Fig. 6. Convergence-time curve of tunnel

5 Application of Microseismic Detection Technology in Karst Development Area Tunnels

5.1 Microseismic Monitoring Plan

Preliminary survey data indicate that the Liangmaoshan Tunnel has karst fissure development sections. To further understand the karst cave development ahead of the tunnel face, microseismic monitoring technology was used for real-time prediction.

During microseismic monitoring in karst area tunnels, the sensor array needs to closely follow the tunnel face, and the sensors need to be moved forward frequently. Considering the number of effective microseismic signals and sensor movement efficiency under the same tunnel, excavation method, and face distance conditions, five sections with five sensors were selected, as shown in Figure 7.

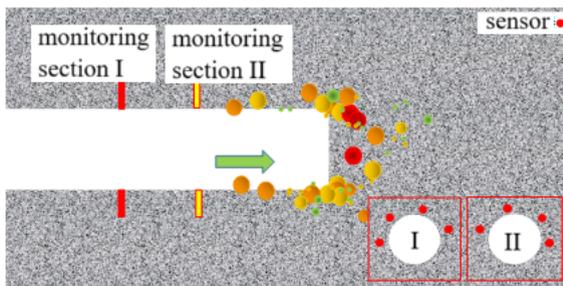


Fig. 7. Tunnel microseismic monitoring scheme

5.2 Microseismic Monitoring Results and Analysis

The tunnel face was excavated in upper and lower benches, with the upper bench floor set as the 0 m height position. The stress gradient slice map (Figure 8), water-bearing probability slice map (Figure 9), and P-wave velocity slice map (Figure 10) at 0 m height were output.

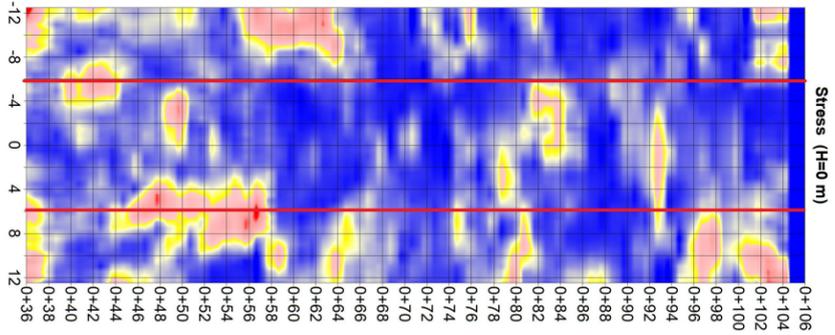


Fig. 8. Stress Gradient Slice

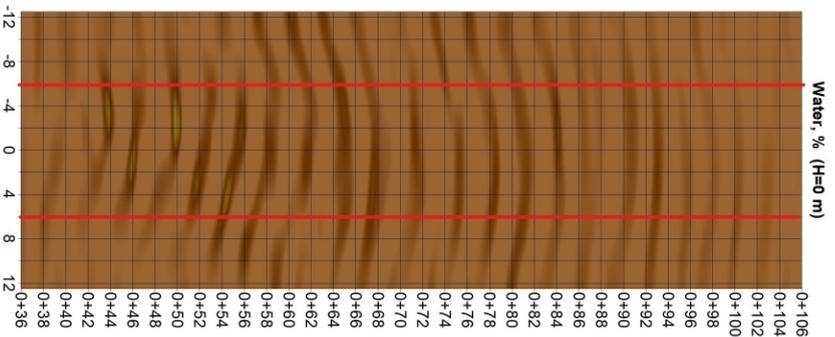


Fig. 9. probability slice diagram of water cut

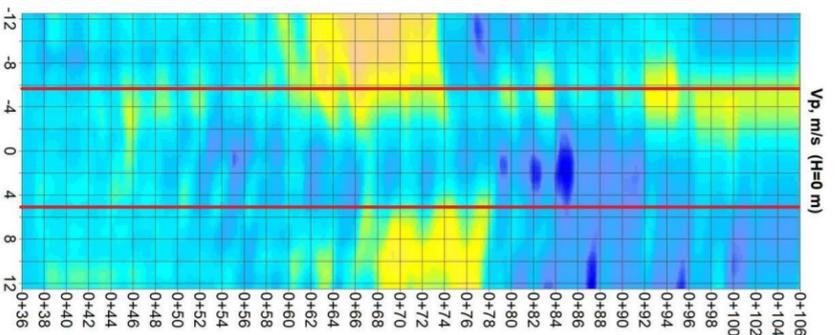


Fig. 10. Slice diagram of longitudinal wave velocity of seismic wave

In the data analysis, the stress gradient, P-wave and S-wave velocities, and water-bearing probability slice maps were mainly referenced. Based on the current face conditions, if the P-wave velocity ahead of the face is close to the current face and the stress gradient shows no significant change, it is judged that the surrounding rock conditions ahead are similar to the current face. If the wave velocity increases significantly, it is

judged that the surrounding rock conditions have improved, with reduced joint and fissure development. If the wave velocity decreases significantly, it is inferred that the surrounding rock conditions have worsened, with dense joints or faults and karst. Areas with larger stress gradient values correspond to areas with significant wave velocity changes, often with higher water-bearing probability.

Ahead of the face, the stress gradient shows no significant anomalies, and the wave velocity shows no overall increasing trend, suggesting that the surrounding rock conditions are similar to the current face, with weak and fractured surrounding rock and poor self-stability. From the water-bearing probability results, the section from k0+80 to k0+150 has low water-bearing probability, suggesting that the surrounding rock is mainly dry to moist. No karst cave development was found within 70 m ahead, and microseismic advanced geological prediction will continue.

6 Conclusion

(1) For large-scale collapses in karst area tunnels, the comprehensive treatment plan of "clay backfilling for waterproofing at the tunnel roof + fine sand filling + foam concrete pumping + advanced large pipe roof + radial grouting" is effective.

(2) The application of microseismic monitoring technology enables real-time prediction of karst cave development ahead of the tunnel face, continuous monitoring of surrounding rock fracture processes, and early warning of potential collapse risks. This method avoids large-scale drilling, minimizing disturbances to the surrounding rock. Through multi-parameter fusion analysis, it achieves precise identification of the scale and filling conditions of karst caves and fractures.

(3) Future research should integrate artificial intelligence (AI) and machine learning (ML) technologies to further enhance disaster prevention capabilities in karst tunnel engineering. Deep learning algorithms, such as convolutional neural networks (CNN), can be employed to automate the classification of microseismic signals, distinguishing tectonic fractures, karst cave expansion, and construction-induced disturbances, thereby improving prediction accuracy. AI models based on real-time monitoring data could predict the evolutionary trends of surrounding rock stability and be integrated into construction management systems to enable risk-level-based early warnings and emergency response coordination.

Acknowledgments

Supported by R&D Program of China Construction Second Engineering Bureau Co. Ltd (2022ZX050001).

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