Cause Analysis of Soft and Hard Rock Tunnel Collapse and Information Management

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Abstract

Collapse is very common in tunnel construction, which can bring immense casualties and economic losses. The interface of soft and hard rock is the typical location for a tunnel collapse. During Shimenya Tunnel construction, collapse of the section ZK123+370~ZK123+365 of the left tunnel occurred. This paper discusses how distribution of layered rock, attitude of the stratum, hydrogeololgy, and advanced geological prediction can contribute to tunnel collapse. According to collapse mechanisms and *in situ* collapse conditions, managing and monitoring plans were applied to the tunnel collapse section. To ensure the efficiency and safety of collapse management, convergence displacement and arch crown settlement were measured in the process of tunnel information management. Cause analysis of soft and hard rock tunnel collapse and information management can provide a beneficial reference for avoiding tunnel collapse and developing collapse management programs.

Introduction

Tunnelling, with the ability to run through the mountains and across the sea, and the advantage of little effect of the external environment, has become the top priority of China's transportation construction. But the diversity of its construction environment determines that the risk of accidents in tunnel engineering applications is higher than that in other works, and the consequences are more serious. Among them, collapse has become one of the most common problems in tunnel construction. According to the details of 82 tunnel collapse accidents occurring in recent years in China, 10 were caused by deformation of surrounding rock, accounting for 12.2% of the total; 50 were impacted in varying degrees by groundwater, accounting for 61%; tunnel lining in 21 collapse accidents was not installed in a timely way, accounting for 25.6%. This shows that strata deformation, groundwater, and lining have largely affected the normal tunnel construction [1, 2], among which changes of lithological characteristics in the surrounding rock are, to some extent, unexpected and instable, and therefore, forecast and management must be carried out by technical measures [3, 4]. Currently, based on different environments of surrounding rock, different methods of tunnel construction have gradually formed. The interface of soft and hard rock [5] is relatively common during tunnel

construction. Thus, the interface area of hard and soft rock, with the characteristics of joint development and loose surrounding rock, has gradually drawn widespread concern and become the focus of prevention during construction. Studies on this type of surrounding rock have been continuously carried out. They focus on the collapse mechanism [5, 6], collapse reasons [7, 10], different construction methods [4, 9], and adopting a single section monitoring method [8,12] that monitors the stability of surrounding rock after management, and some progress has been made. But the studies on tunnel collapse at rock interface have seldom been in combination with the super geological forecast, without systemic summary of collapse reasons and comprehensive management, or adequate monitoring analysis after th management. Therefore, systemic research on such tunnel collapse is necessary.

Shimenya Tunnel has been excavated through the interface of hard and soft rock, with rich groundwater, unstable surrounding rock, and complex geological conditions. Therefore, we must summarize the tunnel collapse systemically and comprehensively [13]. First of all, collapse mechanisms should be analyzed based on the topographic and geological conditions of the location of the tunnel. Secondly, results of detection and monitoring collected before the collapse, such as TSP [14] (tunnel seismic prediction) and GPR monitoring data, should be analyzed to understand the changes of surrounding rock in the tunnel before the collapse and reasons of collapse. Finally, a practical management program should be formulated according to the actual situation to prevent the expansion of the collapse, and changes in the tunnel should be monitored by technological means such as monitoring measurement to verify the feasibility of the management program [15]. A comprehensive understanding of the tunnel collapse will be formed through full analysis before and after the collapse and informationizing management monitoring, and only in this way can the safety and efficiency of tunnel construction be ensured.

Tunnel Collapse

Shimenya Tunnel is located in Shuitianba town, Zigui County, with the tunnel trunk section toward 247°. The tunnel adopts the method of framing, with the station numbers of left line ZK118+963~ZK126+487, and the total length is 7,524.0 m, and right YK118+948~YK126+441, 7,493.0 m; an inclined shaft was set near ZK120+850. The maximum depth of the tunnel is approximately 1,300 m, which is an extradeep tunnel.

On December 19, 2011, during the slag process of ZK123+370~ZK123+365, the arch and arch loin on the right of the driving direction first slumped, and then the slump extended to the segment whose support had been completed, with most of the collapsed matter being massive muddy sandstone. There was linear running water at some points of the collapsed body, and free face of the collapse was the interface of feldspathic quartz sandstone and argillaceous siltstones. The slope toe of the slump body

extended to the mileage of ZK123+375. The height of the landslip was about 4.0 m (to the arch crown of the excavation contour). Fig. 1 is the actual situation of the collapse in the tunnel; Fig. 2 is the sketch map of the position of the collapse.

Collapse Analysis

Geological Analysis

The section ZK123+375~ZK123+355 has a depth of 1,200-1,300 m, with wide and slight knollspine, strong terrain cutting, and narrow intermountain valleys, the profile of which mostly has the shape of "V." the strata belongs to the Penglai group of the Jurassic system, with fuchsia layered sandy mud stone and offwhite feldspathic quartz sandstone interbedding with each other with unequal thickness, and sandy conglomerate between the layers. The design of the surrounding rock being level III, joint and fissure develop.

The groundwater in the tunnel area is mainly water from bedrock cracks, and the bedrock in the tunnel area is

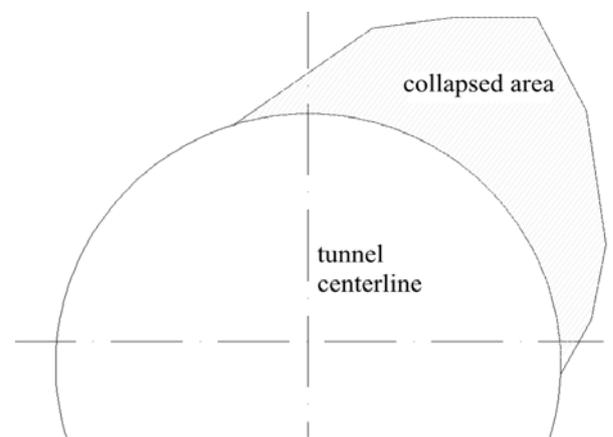Fig. 1. Actual situation of a collapse in the tunnel.

Fig. 2. Sketch map of the position of the collapse.

sandy mud stone and feldspathic quartz sandstone of the the Penglai and Suining groups in the Jurassic system. These groups are relatively water-resist layers within the district, but the weathered fissure in the shallow part develops and thus creates a tectonic fracture zone that provides physical space for groundwater, with vertical recharge from atmospheric precipitation. Due to factors such as fissure development and filling, gushing may appear in the tunnel. And the emollescence effect by groundwater on the surrounding rock can cause instability.

Advanced Geological Forecast

This advanced geological forecast utilizes the forecast system of TSP203 plus. When the seismic wave produced by the excitation of a small amount of explosives in the focal points of the design encounters interface (such as faults, a fracture zone, lithological changes, caves and groundwater, etc.) with different wave impedance of rock, part of the seismic signal reflects back and part of the signal transmits into the front medium. The reflected seismic signal is received by a highly sensitivite seismic sensor.

Tunnel Seismic Prediction (TSP)

TSP is an underground reflection seismic wave technology for advanced prediction of geological condition anterior to the tunnel face. The seismic waves are excited by several (less than 24 generally) small-scale artificial blastings at specific blasting points, received by electronic sensors. When the seismic waves encounter formation interface, joint interface (especially unfavorable geology interface such as fault fracture zone, karst cave, or underground river), reflected waves are generated and received by the receiver, amplified, output, and recorded by digital recorder [16, 17].

The calculation formula of the longitudinal wave speed V_p is:

$$V_P = \frac{L_1}{T_1} \tag{1}$$

...where L_1 is the distance between the seismic source and sensor and T_1 is the transmission time of the first wave arriving the sensor.

The transmission time of reflected wave T_2 can be calculated as follow:

$$T_2 = \frac{(L_2 + L_3)}{V_P} = \frac{(2L_2 + L_1)}{V_P} \tag{2}$$

...where L_2 is the distance between the blasting hole to the reflector and L_3 is the distance between sensor to the reflector.

The sensor can receive the reflected wave information, and present the characteristics and occurrence related to the interface by different dates. The characters and occurrence include the reflected wave speed, delay time, waveform,

strength, direction, etc. Poisson's ratio, the elastic modulus of the prediction tunnel, can be obtained according to the following formula so as to make the prediction of the unfavorable geology anterior to the tunnel face. Poisson's ratio is:

$$\nu = \frac{V_P^2 - 2V_S^2}{2\left(V_P^2 - V_S^2\right)} \tag{3}$$

Dynamic young modulus is:

$$E_d = \rho V_S^2 \left(\frac{3V_P^2 - 2V_S^2}{V_P^2 - V_S^2} \right) \tag{4}$$

Young modulus is:

$$E = \rho V_S^2 \left(\frac{3V_P^2 - 2V_S^2}{2V_P^2 - V_S^2} \right) \tag{5}$$

Shear modulus is:

$$\mu = V_S^2 \cdot \rho \tag{6}$$

Lame constant is:

$$\lambda = \rho(V_P^2 - 2V_S^2) \tag{7}$$

...where V_s is shear wave speed, V_p is longitudinal wave speed, and ρ is rock density.

TSP data is processed through the software of TSPwin in order to obtain a series of results such as the time profile, maps of depth migration, and reflector extraction of P-wave, SH wave, and SV wave, as well as parameters of the physical properties of rock. Figs. 3, 4, and 5 are separate maps of depth migration of P-wave and S-wave, maps of reflector extraction of S-wave, and a diagram of 2D results of the physical properties of the rock body, which are obtained through data processing.
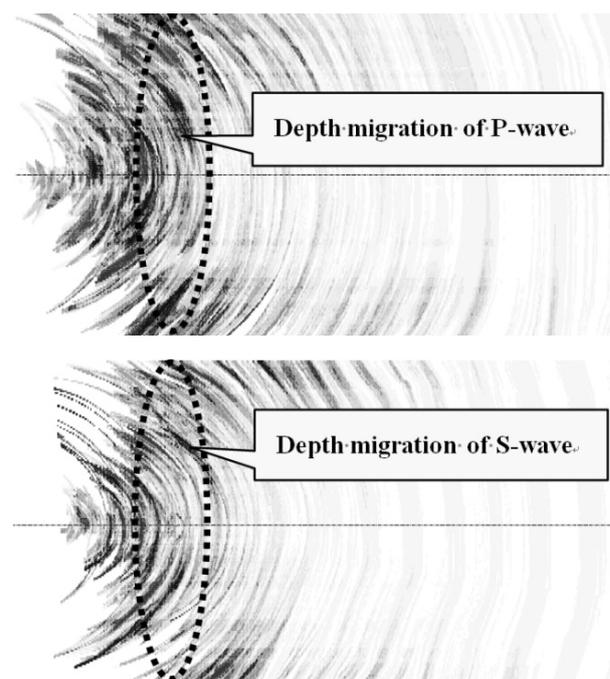

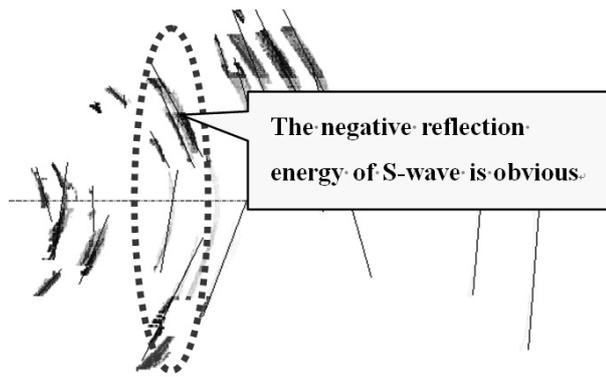Fig. 3. Maps of depth migration.
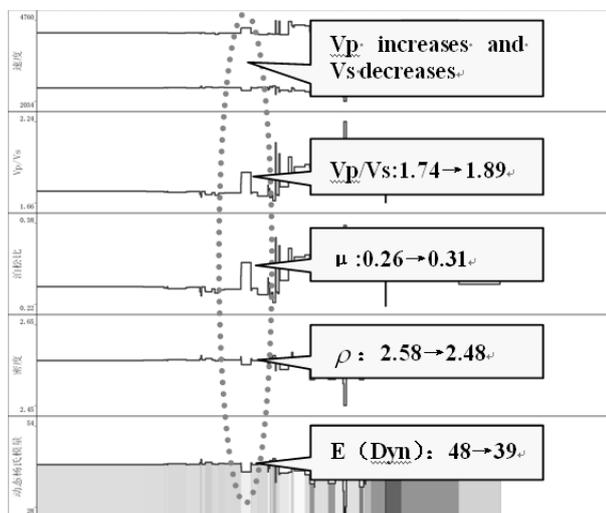
Fig. 4. Maps of S-wave reflector extraction.

Fig. 5. Physical properties of the rock body.

According to the maps of depth migration of P-wave and S-wave, maps of reflector extraction of S-wave and the diagram of the physical properties of rock body obtained through TSP data processing, the section ZK123+375~ ZK123+355 can be analyzed as follows:

(1) in the maps of depth migration, reflection of P-wave is weaker than that of S-wave, and the negative reflection energy of S-wave is stronger

(2) there is obvious negative reflecting surface in the maps of S-wave reflector extraction

(3) the velocity curve of S-wave obviously decreases in the diagram of the physical properties of the rock body

There is a significant increase in the speed ratio V_P/V_S of longitudinal wave to transverse wave and Poisson's ratio δ (V_P/V_S increases from 1.74 to 1.89, and δ from 0.26 to 0.31); rock body density ρ and dynamic Young modulus E.Dyn decreases clearly (ρ decreases from 2.58 g/cm³ to 2.48 g/cm³, and E.Dyn from 48GPa to 39GPa). According to the results of the above analysis, and combined with geological analysis, it can be inferred that ZK123+375~ ZK123+355 is the litho logical interface, where lithology changes from hard to soft and the joint and fissure of rock body develops, filled with fissure water.

Summary of Collapse Causes

Through the geological analysis of the collapsed area and a survey of the construction process, combined with the results of advanced geological forecast of this segment, the causes of the collapse are summarized as follows.

(1) The layered distribution of strata led to the instability of the surrounding rock in the tunnel. The rock of the collapsed area is the inter-bedding of sandy mudstone and feldspathic quartz sandstone with unequal thickness, with sandy conglomerate between the layers (Fig. 6). And the right spandrel of the tunnel in the collapsed area happens to be located in the interface of the two rocks, which is the overlap of moderately and weakly weathered rock. Affected by three joints, the surrounding rock is in an unstrained state after being blasted, and the joints between layers were destroyed at first, eventually leading to the general shear failure of the surrounding rock, with the collapse form being similar to that of a roof muster. This is the main cause of this collapse accident.

(2) Affected by groundwater. The collapsed area in the tunnel was rich in groundwater, with leakages on the top and on the walls of the cave. The long-term seepage caused the loss of self-stabilizing capacity of the surrounding rock, which affected the stability of the surrounding rock in the tunnel.

(3) Insufficient emphasis on advanced forecast. It was explicitly mentioned in the advanced geological forecast of ZK123+370~ZK123+365 that the geological conditions of this segment were special and that prevention was necessary. But the construction organization did not pay much attention, and did not change construction methods according to the deformation of the surrounding rock in time, eventually leading to the tunnel collapse.

(4) Inverted arch and secondary lining were not timely. It can be found through the research of construction conditions before the collapse that the construction of the inverted arch in the tunnel lagged, and that the arch-scaffolding near the tunnel face was unable to form a closed loop, which meant the stability of the surrounding rock could not be guaranteed. In addition, the lagging excavation of the secondary lining in the tunnel
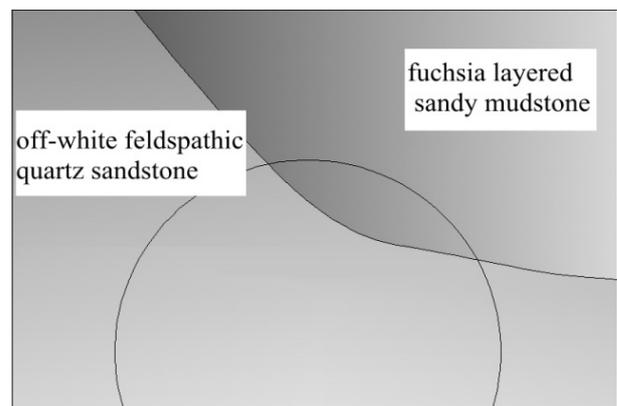

Fig. 6. Profile of the surrounding rock in the collapsed area.

was serious, failing to reach the required supporting strength. With the surrounding rock unable to converge, the collapse ultimately happened.

Management

The management process of the collapsed area is presented as follows.

(1) In order to avoid a possible chain reaction of collapse, the surrounding lining needs to be reinforced. The reinforcement measure for the mileage ZK123+375~ZK 123+370 adopts the support of I18 steel arch-scaffolding with an interval of 0.6 m. Between adjacent arch-scaffoldings, set vertical connection bars of Φ22, with toroidal interval of connection bars 0.6 m, and spray a layer of concrete of C20 strength and 24 cm thickness to wrap the arch-scaffolding. Put on 4 pieces of Φ42 small tube feet-locked rockbolts, with 3.5 m in length, at each arch-scaffolding, and solidly weld the outer fringe with the steel arch-scaffolding.

(2) After the deslagging of collapsed accumulation body, use the excavation trolley and hang Φ8 reinforcing fabric (20×20cm), which should tightly cling to the rockface. Spray a layer of concrete of C20 strength and 30 cm thickness several times to seal the collapsed area. After this, when the sprayed concrete of C20 strength reaches the designed strength, SFVb construction initial support should be put on ZK123+370~ZK123+365. At the same time, embed a Φ125 injection tube and a Φ108 exit tube, each with a length of 4.5m. Adopt Φ42 advanced small tube on ZK123+365~ZK123+355 as advance support.

(3) Embed the concrete to the backfill collapsed cavity. after the construction of initial support on ZK123+370~ ZK123+365, spray concrete of C20 strength and 20cm thickness and closely excavate the tunnel face to make the entire collapsed cavity sealed; after initial support on ZK123+370~ZK123+365 meets the design strength, pump concrete of C20 strength several times with the concrete pump to backfill the collapsed cave.
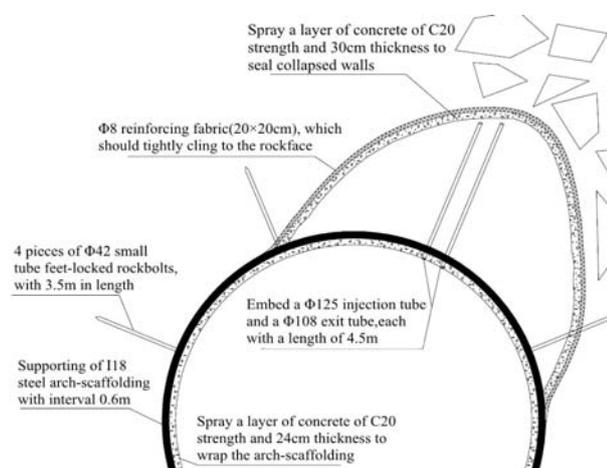

Fig. 7. Sketch of collapse management.

Table 1. Monitoring results of collapsed section.

Section number	Section 1æZK123+370	Section 2æZK123+365
Accumulated converging displacement (mm)	30.22	41.75
Maximum of relative converging displacement	0.29%	0.41%
Accumulated crown settlement of arch crown (mm)	67.28	40.17
Maximum of the converging rate (mm/d)	2.65	2.14
Final converging rate (mm/d)	0.13	0.08
Maximum of settling rate of arch crown (mm/d)	4.52	5.23
Final settling rate of arch crown (mm/d)	0.03	0.05

(4) Arrange monitoring sections on the reinforced collapsed area, ZK123+370~ZK123+365, to perform monitoring measurement, fully analyze and deal with the measured data, and test the reinforcement effect.

Analysis of Monitoring Measurement after Management

Monitoring Measurement

The main purpose of monitoring measurement is to modify the design and guide the construction with the monitoring results, and to verify the feasibility of the management program. Through the analysis and processing of the monitoring data as well as necessary calculations and judgment, the safety of tunnel construction and stability of the surrounding rock are ensured.

In order to judge the stability of surrounding rock after management of the collapsed area more fully and correctly, two monitoring measurement sections of ZK123+370 and ZK123+365 were taken to measure the arch crown settling volume and converging displacement of surrounding rock respectively. Among them, the monitoring time period of ZK123+370 was from 2011.12.25 to 2012.1.26, lasting 32 days; and ZK123+365 was from 2012.1.1 to 2012.2.9, lasting 39 days. Through regular monitoring, which is in accordance with the relevant design requirements and specifications, the final convergence of the two sections was obtained ultimately. Parameters such as the final settling volume are shown in Table 1.

Analysis of Monitoring Measurement Results

It is formulated in the monitoring measurement specifications that the relative displacement value of the tunnel (the burying depth of which is less than 50 m in level III
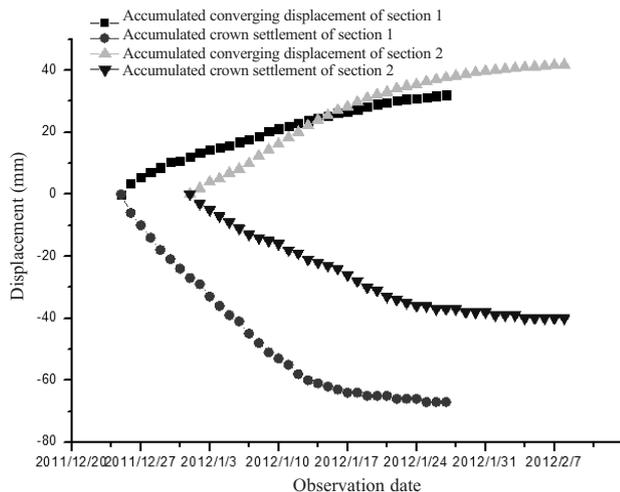
Fig. 8. Change curves of accumulated surrounding rock convergence and crown settlement of the monitoring sections.
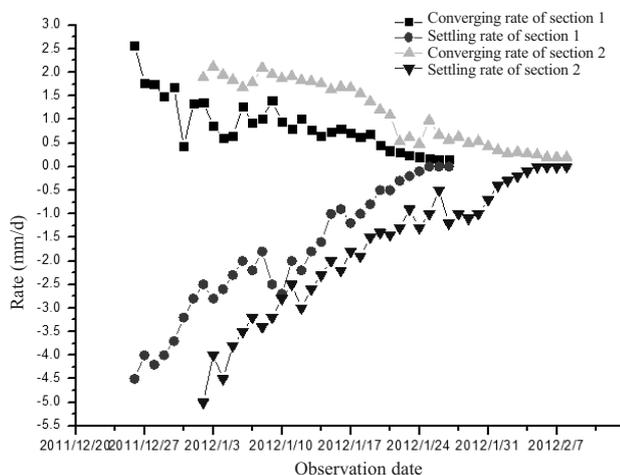

Fig. 9. Change curves of the convergence rate and crown settlement rate of the monitoring sections.

surrounding rock) be allowed to vary between 0.10% and 0.50%, that the converging rate of surrounding rock in late monitoring should be less than 0.2 mm/d, and that the settling rate of the arch crown should be less than 0.1 mm/d. According to the table above and index graphs, the data in the monitoring period finally stayed stable within the scope of the monitoring provision; the curves of converging time and arch crown settling time showed significant trend of converging; the surrounding rock after management all achieved the requirements of convergence within the specified time and stayed in a stable state.

Conclusions

(1) Through analyzing the monitoring results of TSP advanced geological forecast, combined with the specific circumstances of the project, it can be summarized that the joint destruction at the interface area of layered rock is the main internal cause of the tunnel collapse.

(2) The collapsed wall was reinforced with reinforcing fabric and shotcrete, and the collapsed cavity was filled by means of small tube injection. This method is practicable for collapse management in the interface area of layered rock.

(3) After the collapse management of the layered rock tunnel, monitoring measurement of the convergence of the multi-section surrounding rock and settlement of arch crown in the collapsed area was carried out. Through timely information feedback, stability of the surrounding rock in the tunnel was accurately predicted and construction safety was guaranteed.

Acknowledgements

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