Analysis on deformation and failure of rock mass in deeply buried Guanshan railway tunnel under high in situ stress

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Abstract

Guanshan railway tunnel, located in Gansu Province, China, is a deeply buried diorite rock mass tunnel under high in situ stress. On the contrary of the good rock mass classification predicted in investigation stage, the real situation after excavation shows a great change of rock mass parameters and structure, revealing not only the degradation of rock mass quality also the significant anisotropy of rock mass. The degradation and anisotropy of rock mass is described and analyzed based on the statistic analysis of the rock structural plane in field. Meanwhile, borehole camera is adopted to further observe the progressive deformation and failure of rock mass in an interval after excavation. As one of the hard and brittle plutonite, the uniaxial compressive strength (UCS) of diorite is a relatively high index on investigation stage, which is obviously larger than the in situ stress. However, the deformation and failure of diorite rock mass happened prevailingly. As the evidence obtained by borehole camera, the micro fracture or closed rock structural plane under high in situ stress will re-open and develop quickly after the excavation, which degraded the stability and quality of rock mass structure. To figure out the deformation and failure process, an angle rotation of uniaxial compressive test is designed. The uniaxial pressure is lower than the uniaxial compressive strength (UCS) of diorite. This test fully presents the feasibility of the progressive deformation and failure process of diorite rock mass during the strongly stress field change after excavation. It proves that the deformation and failure of rock mass is the result of the degraded rock mass structure, induced by stress field change after excavation. Besides the rock mass structure, stress field change is always a significant factor in deeply buried excavation project under high in situ stress.

Keywords: rock mass, deformation and failure, stress field, diorite

1. Introduction

Tianshui-Pingliang railway under construction is an important National Class I electrified railway, for both the passengers and freight service in the remote and undeveloped area in Gansu Province，EW China. Tianshui-Pingliang railway stretches about 99 km in main line, including about 78 km of bridges and tunnels. This project aims to built a connection between Tianshui station on existing Lanzhou-Lianyungang railway and Huating station on existing Baoji-Zhongwei railway towards Pingliang city (see Fig. 1). The challenges of this project focus on the deeply buried Guanshan tunneling project，with the maximum buried depth 831m, from Gongmen town to

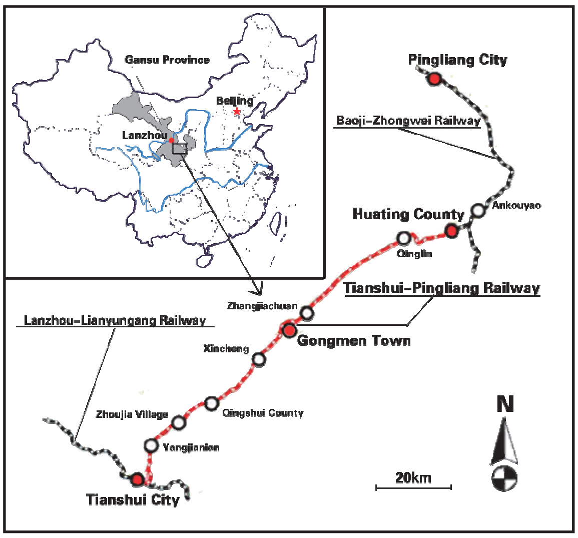

Fig. 1 Location and route chart of the project

Qinglin, acrossing the section of local Guanshan Mountain within the elevation range from 1000m to 3500m, belongs to Liupanshan Mountains area, one of the youngest mountains in China.

2. Geological settings

2.1 Topography

The direction of the Guanshan railway tunnel axial is mostly N46°E. As the result of changed terrain fluctuation and developed gullies, the natural slope is usually 20°~ 35°and some of them are steeper with visible bedrock scarps. The surface elevation is around 1695m~2608m with a maximum buried depth of tunnel 831m. The whole area is in a state forestry centre covered by afforestation over 10 years. Therefore this area is forested and the vegetation coverage rate is nearly 90%.

2.2 Lithology

The region is a large area of diorite intrusion in Variscan with huge batholite and associated small rock body. The rock mass consists mainly of amphibolite, biotite granodiorite, Biotite quartz diorite, biotite adamellite, hornblende diorite, quartzite, andesite, diorite and granite. The area near Guanshan tunnel is mainly diorite(δ4) , in the color of steel gray. The diorite particle size is fine grain to coarse grain structure and structure is massive. Some of rock mass are with developed structural planes.

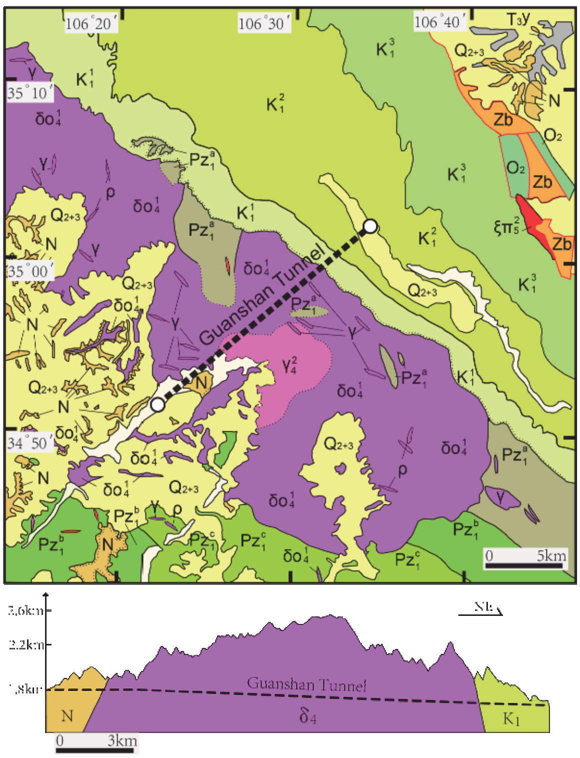

Fig.2 Geological map of Guanshan Tunnel

With the help of microstructure test and Scanning Electron Microscope (SEM) on samples at DIK76+558 in a buried depth of 795m (see Photo. 1 and Photo.2), it is observed that the diorite surrounding the tunnel is mainly consist of plagioclase, amphibole, chlorite and mica. Silicate minerals account for a great proportion and make the rock mass hard. In addition, after the hydrothermal process in magmatic period, amphibole and black mica altered into chlorite, while feldspar altered into mica, kaolinite and so on. In some places, the rich layer silicate minerals such as mica and chlorite gathered and created membrane or thin layer, which will weaken the strength of structural plane dramatically, especially enhanced by the water softening. Meanwhile, on the account of a large amount of plagioclase, mica, chlorite and other minerals, diorite became the direction-sense and developed sheet structure, revealing significant anisotropy. When the external force changes, it is easy to occur inter layer deformation and shear dislocation, spalling or bulking failure, especially the structure is weaken by mica and chlorite after encountering water.

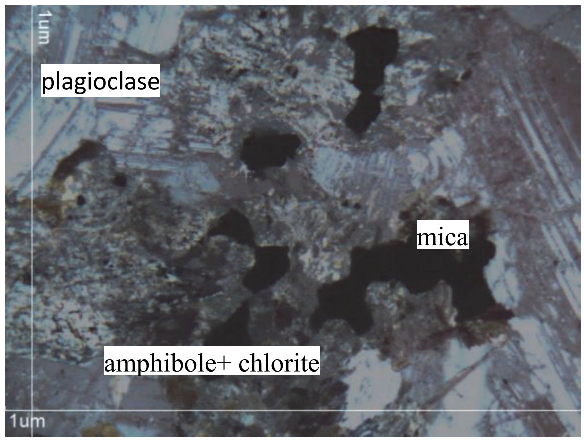

Photo.1 Composition analysis of diorite

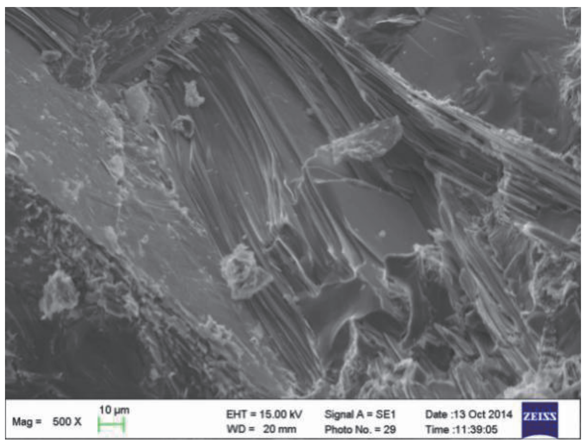

Photo.2 Microstructure analysis of diorite

2.3 Tectonic structure and in situ stress

In the aspect of tectonic structure, Guanshan mountains where the Guanshan railway tunnel located is in place within the Guanshan fold belt, which is a secondary structural unit of Liupanshan fold belt. Guanshan fold belt is a geosyncline uplift belt among Tongwei-Qingshui fault (F3) and Zhuanglang-Guguan fault (F4) (see Fig. 3). Except a large area of diorite intrusion in Variscan at the ridges of Guanshan mountains, there is no other evident of fracture structure nearby Guanshan railway tunnel. Based on the tectonic structures, we could infer that the orientation of maximum principal compressive stress is near NE direction. And it is basically parallel to the tunnel axial orientation, beneficial for releasing the rock mass pressure.

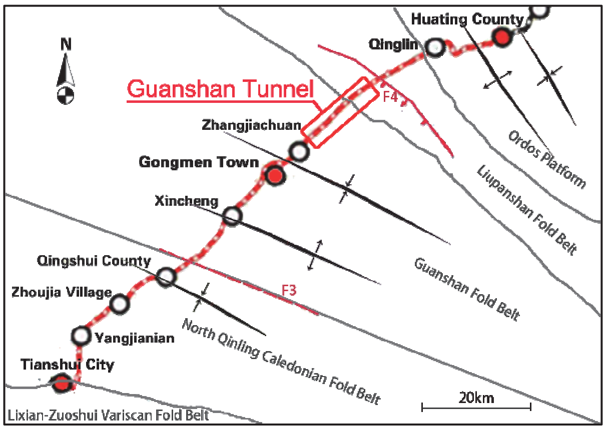

Fig 3 Tectonic structure map

By the hydro-jacking test at DIK76+070, it is analyzed that, in Guanshan railway tunnel, the maximum principal compressive stress is about 31.35 MPa, the in situ tensile strength is about 1.32 MPa ~ 15.0 MPa and the predominant direction of maximum principal compressive stress nearby the tunnel is N55ºE ~ N59ºE. As a whole, deeply buried Guanshan railway tunnel is under high~very high in situ stress.

3. Deformation and failure phenomenon of rock mass

According the displacement monitoring of upper bench near DIK76+558 in a buried depth of 795m, the displacement of support is still large at initial stage in the tunnel. After the ring closure of inverted arch, the displacement will end in stability, the maximum crown settlement (MP-C) is about 80mm and the maximum horizontal convergence is about 200mm. The horizontal convergence is far above the crown settlement (MP-C). Displacements of MP-A are greater than MP-B both in horizontal and vertical directions.

Corresponding with the severe horizontal convergence, it frequently appears that the failure rock mass and steel arches in side walls (see photo.3), indicating the strong deformation and failure after excavation. Also, the wave velocity inversion proves

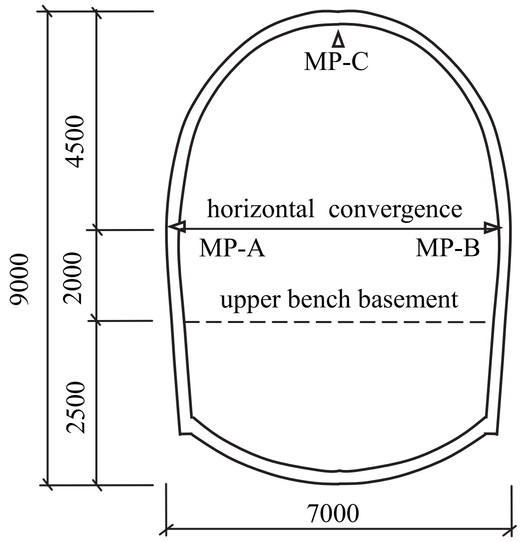

Fig.4 Measuring points for displacement (unit: mm)

the adverse effect of excavation . In many section of Guanshan tunnel, rock mass wave velocity ahead the working face, obtained by advanced prediction of geological radar, is usually nearly 5000 m/s. However, after excavation, the p-wave velocity of rock block dramatically drops down to 2300 m/s in many cases. As Equation 1 shown, elastic modulus is sensitive to p-wave velocity. The decrease in p-wave velocity of Guanshan tunnel rock mass describes the reductive elastic modulus and the weakening ability of rock mass to resist deformation and failure in the changed stress field during the tunnel excavation.

$$v = \sqrt{\frac{E(1-\mu)}{\rho(1+\mu)(1-2\mu)}} \quad (1)$$

Where E is elastic modulus, ρ is rock density and v is p-wave velocity and μ is Poisson' s ratio.

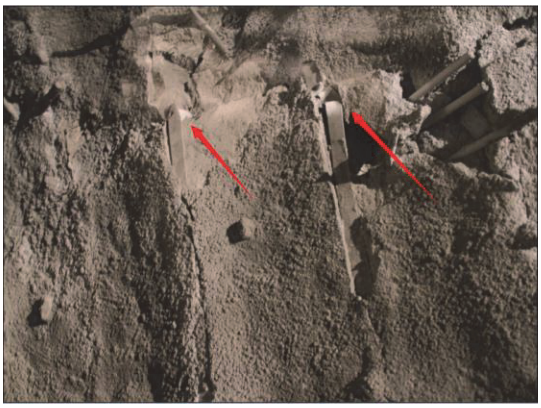
Photo.3 Failure rock mass and steel arches in side wall at DIK76+558

After excavation in deeply buried depth and high in situ stress, the rock mass is easy to occur deformation, especially towards the free faces. The decreased elastic modulus worsens this process. More cracks in different direction inside the rock mass will develop, weaken the integrity of rock mass and highlight the anisotropy of rock mass.

4. RQD anisotropy of rock mass

After field survey and structural plane statistics around the excavation surface of rock mass in the tunnel, it is found that Guanshan tunnel develops three sets of preferred structural plane (see Table 2). Assuming the intervals of intersection points obey negative exponential distribution along any direction, we could predict the RQD, Deere (1989), in any direction by Equation 2 (Wu Faquan, 1993; Wines D R et al., 2002, Hu Xiuhong, 2009).

$$\mathrm{RQD} = (1 + 0.1\lambda_s)e^{-0.2\lambda_s} \times 100\% \qquad (2)$$

Where λ_s is structural plane density in any direction, is given by:

$$\lambda_s = \sum_{i=1}^{n} \lambda_i \left|\cos\delta_{si}\right| \qquad (3)$$

Where n is the total amount of the structural planes and $\left|\cos\delta_{si}\right|$ is the cosine of included angle between any direction and normal of each structural plane.

As shown in Fig.5, when the tunnel axial is mostly N46°E, the RQD in all directions on tunnel cross section proves the apparent anisotropy, ranges from 9.1% to 50.5%. Besides the structure, the strength of rock mass is anisotropic as well. The rock mass quality nearby the excavation surface is very bad, especially in the side walls and crown of the tunnel. However, the RQD will gradually increase apart from the excavation damage zone.

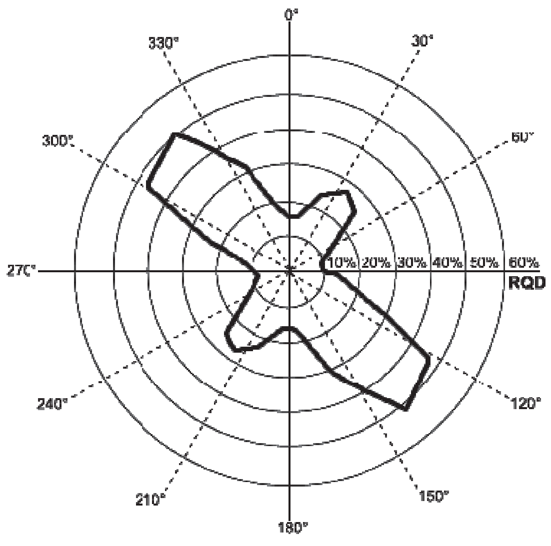

Fig.5 RQD in all directions on tunnel cross section

5. Borehole camera observation

For a more intuitive and comparative analysis on the degradation of rock mass after tunnel excavation, we adopted the borehole camera to observe the rock mass situation twice in 1# and 2# observation holes ahead the working face on DIK76+543 (buried depth 795m) respectively, given 6 hours apart (see photo. 4). The two observation holes are almost perpendicular to the working face, with a small inclination.

Table 2 Preferred structural plane parameters

ID	Dip direction (°)	Dip angle (°)	Normal density (m^{-1})
J_1	60	80	8.1
J_2	251	34	6.2
J_3	210	70	10

Photo.4 1# (left) and 2# (right) borehole camera observation holes at DIK76+543

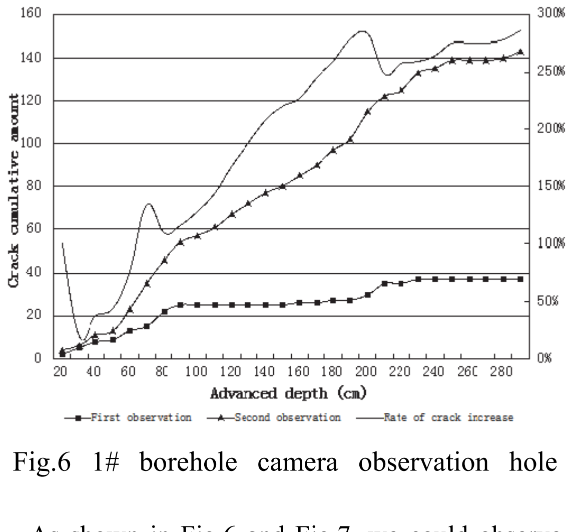

Fig.6 1# borehole camera observation hole

As shown in Fig.6 and Fig.7, we could observe that after 6 hours the rock mass structure ahead the working face is changing strongly with the evidence of soaring new crack amount. With the effect of stress field adjustment induced by excavation in a deeply

Table 3 Stress rotation test result of Sample No.2

No.	Surfaces	Fracturing description	No.	Surfaces	Fracturing description
1st round	1-1a	No significant changes	3rd round	1-1a	Through cracks within surface 1 and 2 respectively and cracks develop and intersect in surface 3.
	2-2a	No significant changes		2-2a	Cracks develop rapidly in surface 2 and 2a. A crossing crack between surface 1 and 2a.
	3-3a	No significant changes		3-3a	A crossing crack and wedge damage in surface 3 and 2a
2nd round	1-1a	Inclined cracks at the corners in surface 1,1a,2a and 3a.	4th round	1-1a	Through cracks in surface 1, 2a and 3 respectively.
	2-2a	New cracks in surface 1 and 3 and wedge damage at the border of surface 1 and 3.		2-2a	Failure. Cracks develop rapidly in all surfaces and significant tensile cracks along the direction of unaxial compressive pressure in the border of surface 1,2 and 3.
	3-3a	Existed cracks develop in surface 1 and 1a. New cracks and wedge damage in surface 2 and 3a.			

buried 795m tunnel in high in situ stress, the new crack along with the reopened close cracks develop further and degrade the structure and strength of rock mass. The rate of new crack increase could be 200% and 400% averagely in two observation holes but exceeds 600% in the advanced depth 110cm~130cm in 2#

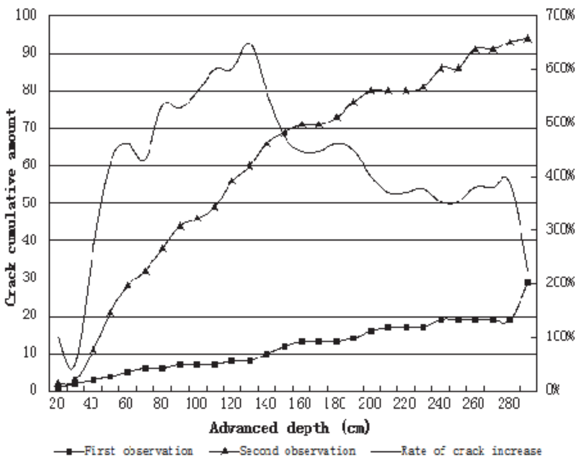

Fig.7 2# borehole camera observation hole result

observation hole. Importantly, analyzing the slope of the curves of the second time of observation in both 1# and 2# holes, we could find the crack amount increase more quickly both in the range of 1m advanced depth. It is also the severe damage zone of rock mass. It suggests that we should pay abundant attention to this key and very nearby zone, easy to impose engineering measures to mitigate the the bad effect of stress field adjustment induced by excavation, to control the degradation of strength and structure of deeper rock mass.

6. Stress rotation test

As one of the hard and brittle plutonite, the uniaxial compressive strength (UCS) of diorite, nearly 100 MPa, is a relatively high index on investigation stage, which is obviously larger than the in situ stress of Guanshan tunnel as well. However, as the evidences obtained by RQD anisotropy analyses and borehole camera observation of rock mass after excavation, the deformation and failure of the rock mass happened largely. To figure out the degradation on strength and structure of rock mass under the changing stress field induced by excavation disturbance, an angle rotation of uniaxial compressive test is designed.

In the stress rotation test, among several diorite rock blocks samples after excavation at DIK76+543 in the buried depth of 795m, we were successful to cut one of the rock blocks into 3 cubic rock samples (Sample No. 1, 2 and 3) in the size of 100mm x 100mm x 100mm and labeled the 6 surfaces by 3 pairs of parallel surfaces (1-1a, 2-2a and 3-3a).

Sample No.1 is taken to the uniaxial compressive test and the UCS is 82 MPa, which will be taken as the reference UCS for all the samples cut from the same diorite rock block. Then the Sample No.2 and No.3 were applied by an uniaxial compressive pressure on the 3 pairs of parallel surfaces in turns until failure. The cycling order is first 1-1a and then 2-2a and finally 3-3a. The uniaxial compressive pressure is 49 MPa, 60% of the reference UCS and about 1.5 times of the maximum principal compressive of in situ stress (about 31.35 MPa) in the Guanshan tunnel. The two samples finally failed in the second stage 2-2a and the first stage 1-1a of 4th round of stress rotation test respectively.

We take the detailed test result of Sample No.2 as example, listed in Table 3. During the 4 rounds of repeating compressive pressure rotation, no obivious cracks or wedges happen in first round and then the cracks and wedge damages begin to occur in second round. In third round, wedge damages occur severely and new cracks develop greatly. Finally, Sample No.2 failed with a significant tensile cracks along the direction of unaxial compressive pressure in the second stage of fourth round.

It implies that, under the influence of stress rotation, the existed and new developing cracks could more fully and easily reactivate and grow in lower compressive pressure than UCS, resulting in the anisotropy and whole degradation of rock sample. Some of the cracks show preferred orientation and induced the significant through cracks to failure. The stress rotation test fully presents the feasibility of progressive deformation and failure process of diorite rock mass nearby the surface during the stress field adjustment after tunnel excavation.

Photo.5 Wedge and crack damages of Sample No. 2 after the first stage 1-1a, 4th round of stress rotation test

7. Conclusions

The deformation and failure of rock mass in Guanshan tunnel is the result of the degraded strength and structure of rock mass, induced by stress field change after excavation disturbance. In this case, the anistropy and integrity of rock mass is much more important than the strength of rock mass. The disturbed stress field change is always a significant factor on rock mass quality in deeply buried excavation project under high in situ stress.

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