Anisotropic Failure in Foliated Rock Tunnels: Comparison between Deformational Behavior and Confined Pressure

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Abstract. Foliated rock mass, such as slate, schist, and phyllite, is widely distributed in Northwest China. During tunnel excavation, some geotechnical problems are encountered in the surrounding rock, such as excessive over break and intense deformation by asymmetric squeezing, because of low strength, poor self-stationary, and intense anisotropy of structural strength. The large deformation results in intense damage of the primary support and splitting of the secondary support, which can significantly influence the construction and security of tunnels. Anisotropic failure in foliated rock tunnels under high stress conditions has become an engineering challenge. The deformational anisotropy of both rock and underground structures has drawn increasing attention in the field. The relative orientation of foliation plays an essential role on the deformational behavior of layered surrounding rocks by laboratory experiments and numerical simulations. Meanwhile, the symmetrical rigid support structure cannot effectively resist the asymmetric deformation exhibited by the excavation in foliated rock mass. Consequently, the excessive confined pressure generated between surrounding rock and support is essential for anisotropic failure in tunnels. In-situ monitoring tests of deformation and contact pressure were conducted in different positions of sections within a deep-buried tunnel in the Lanzhou–Chongqing railway. The comparison results indicated that the contact pressure of the support system show ed irregular space distribution, which agreed with the concentrated deformation position of the surrounding rock. However, the monitoring data showed that the occurrence of failure obviously lagged behind the stabilization of convergence deformation. The internal confined pressure could last for a long time period than the deformation monitored outside. The failure of the support takes place in the weak axis plane because of its small lateral anti-bending rigidity. The authors believe that the proposed findings will be a step further in supporting the design of tunnels passing through foliated rock mass under high in-situ stress conditions.

Key words. Railway tunnel; Large deformation; In-situ monitoring; Foliated rock mass; High in-situ stress

1. Introduction
With the rapid development of tunneling to mountainous terrain in northwestern China, tunnels gradually exhibit long length, large section, and deep depth. Difficult situations often have to be encountered such as high in-situ stress, poor formation conditions, and abundant groundwater. Excavation problems, such as tunneling-induced large deformation, have been a major obstacle in railway construction in mountainous areas [1-3]. In particular, anisotropic deformation in foliated rock mass under high stress condition has become an engineering challenge and has attracted extensive attention worldwide [4-9].

Researchers have concluded that the relative orientation of foliation plays an essential role in the behavior of underground structures. To date, the effect of dip angle on the deformation of layered surrounding rocks has been verified by experiments and numerical simulations on the premise of fixed
dip directions [10-14]. The relative dip direction can also affect the main deformation regions around tunnels [15]. The normal direction of rock layer points to the inside of a tunnel because the minimum tensile strength of a bedding plane appears on this region, and rock bending is generally easier than rock slipping.

When anisotropic deformation was encountered in the tunnels, significant problems with the support system developed, and they were observed in the deformed steel sets. Asymmetric deformation characteristics were exhibited by surrounding rock excavation, and the symmetrical rigid support structure could not effectively control the deformation. Thus, the tunnel generated significant initial support deformation and damage. The concrete performance was indicated by cracking and crushing of the sprayed concrete, and longitudinal splitting occurred between the steel arches, as shown in Figure 1. The steel arch was affected by the convergence deformation of the surrounding rock, and local deformation was severely distorted; thus, the bearing effect on the surrounding rock was lost. A large area of initial support damage caused frequent changes in the surrounding rock level of each tunnel.

![Figure 1](image.png)

**Figure 1.** Typical failure phenomena of preliminary bracing
(left: Muzhailing tunnel in China; right: Saint Martin La Porte access tunnel in France)

### 2. Project overview

#### 2.1. Presentation of Lan-Yu Railway

Lan-Yu railway (from Lanzhou to Chongqing), which is located in the northeastern Qinghai–Tibetan Plateau of China, is a typical case. A large number of tunnels along this railway line had to be slowed down or stopped during excavation because the lining systems were broken by intense squeezing. During tunnel excavation, some geotechnical problems were encountered such as hoop extrusion and portrait rift on the surface of primary support, rupture of sprayed concrete, and distortion of steel arc truss. A large proportion of designed supporting system of tunnels has been changed because the class of rock quality declined compared with the original classification (see Table 1). Consequently, a remarkable amount of investment must be input for the change.

| Tunnels    | Excavated (m) | Length of lining changed according to modification of rock class (m) |
|------------|---------------|-------------------------------------------------------------------|
|            |               | III to III+ | III to IV | III to V | IV to IV+ | IV to V | V to V+ | Sum | Proportion (%) |
| Majiashan  | 3910          | 40          | 3049      | 25       | 558       | 60     | 0       | 3732 | 95.4           |
| Tongzhai   | 3618          | 85          | 1743      | 15       | 330       | 0      | 20      | 2193 | 60.6           |
| Maoyushan  | 2863          | 0           | 2193      | 50       | 447       | 10     | 8       | 2708 | 94.6           |
| Tianchiping| 5091          | 10          | 3162      | 71       | 322       | 321    | 0       | 3876 | 76.1           |
| Muzhailing | 7687          | 125         | 855       | /        | 3275      | 5      | 480     | 4739 | 61.6           |
| Muzhailing | 8999          | 192         | 573       | /        | 2987      | 8      | 540     | 4350 | 48.3           |

*From China Railway First Survey and Design Institute Group Co. Ltd.*
2.2. Geological settings of case study

The Muzhailing tunnel on the middle part of Lan-Yu railway, which has already been completed, is located in Gansu Province, China. The starting and ending points of the tunnel are located at DK 173 + 350, 0 and DK 192 + 395, 0, respectively. The total length of the tunnel is over 19 km, and the maximum depth of the tunnel is up to 600 m. The Muzhailing tunnel is a single-track double-hole tunnel; a single section is about 84 m² with a diameter of 7 m and width of 12 m. The construction of the tunnel began in March of 2009 and was completed in July of 2016.

![Figure 2. Location and geological settings of the research area](image)

It is in the northeast edge of Tibet Plateau and the intersection part of North and South Tectonic Belt and West Qinling Orogen [16]. The tunnel was excavated in the south wing of anticlinorium called the Dacotaot anticline, which has its axial direction of EW and five subsidiary folds. The main fault in which the tunnel excavated is named the Meiwu–Xinsi fault zone (F2), which is developed in the anticlinorium mentioned above. The in-situ stress measurement was conducted in three boreholes (Table 2). Thus, tectonic stress plays the main control role in current stress field in the research region. The maximal horizontal stress $\sigma_H = 27.16$ MPa, which is regarded as high in-situ stress with the lateral coefficient $k$ of 0.8–1.1. However, the lateral coefficient $k$ approached 3 after excavation, which exhibited anisotropy distribution.

| Boreholes | Code   | Depth/m | $\sigma_H$/MPa | $\sigma_h$/MPa | $\sigma_v$/MPa | Direction of rupture/° |
|-----------|--------|---------|----------------|---------------|---------------|-----------------------|
| MSZ-01    | 1      | 244.64–245.44 | 24.95          | 14.95         | 7.97          | N34°E                |
|           | 2      | 294.80–295.60 | 27.16          | 16.16         | 8.53          | /                     |
|           | 3      | 315.55–316.35 | 6.77           | 4.53          | 7.08          | /                     |
|           | 1      | 272.35–273.15 | 6.34           | 4.86          | 7.32          | /                     |
| MU Z-08   | 2      | 281.60–282.40 | 9.40           | 7.52          | 8.01          | N29°E                |
|           | 3      | 307.93–308.73 | 10.47          | 6.70          | 8.74          | /                     |
|           | 4      | 336.11–336.91 | 10.24          | 6.86          | 8.91          | N39°E                |
|           | 5      | 342.59–343.39 | 16.70          | 8.70          | 4.42          | N32°E                |
| DZG-01    | 1      | 24.35–25.33   | 15.74          | 8.24          | 4.50          | /                     |
|           | 3      | 29.64–30.44   | 13.79          | 7.79          | 4.60          | N23°E                |

The major stratum Muzhailing tunnel traverses occurs within the lower Permian carbonaceous slate (Figures 3a and 3b) and tectonic mélanges, which comprise more than 80% of the tunnel length. The thickness of a single layer of the rock varies around 2–10 cm and connected with carbonaceous films. This major layer has very weak rock quality, and no swelling was detected [17]. However, the weighting material between terranes softened in water. Tectonic mélanges are the chaotic,
heterogeneous geological mixtures of blocks formed by tectonic fragmentation and mixing of rocks in fault zones [18].

![Figure 3. Foliated feature of carbonaceous slate in different scales](a, meter scale; b, centimeter scale; c, micrometer scale)

Previous studies have shown that layered rock masses present significant differences in failure characteristics due to the anisotropic characteristics of structures and the influence of load directions. Correspondingly, we determined the mechanical properties of carbonaceous slate through laboratory tests. Seven sets of different loading directions (this paper defines the loading angle between the principal stress and the specimen bedding for loading direction) were used under uniaxial compressive strength (UCS) tests. As shown in Figure 4, the UCS of all specimens was within 30 MPa, which indicated that the carbonaceous slate belongs to soft rock. The curves exhibited A “V” type distribution with the change in loading direction, in which the minimum value was obtained at 30°. Meanwhile, the UCS of specimen under saturated conditions was significantly low, and the strength of surrounding rock was significantly weakened by water.

![Figure 4. UCS curves of carbonaceous slate specimens](dry state)

We define the anisotropy coefficient of rock as

$$R_{\sigma_c} = \frac{\sigma_{max}}{\sigma_{min}}$$

where $R_{\sigma_c}$ is the anisotropic coefficients defined based on UCS; and $\sigma_{max}$ and $\sigma_{min}$ are the maximum and minimum uniaxial compressive strength values measured, respectively. The anisotropy coefficients of carbonaceous slate were found to be 2.05, and the mechanical properties of the specimens demonstrated significant anisotropy characteristics. The layered structure had a significant influence on the strength of rock mass, and the groundwater promoted the weakening of surrounding rock mass. Therefore, the strength of surrounding rock was extremely low and showed anisotropy with the loading angle. When the load of the tunnel section exceeded the ultimate compressive strength
during the excavation process, large deformation was bound to occur. The anisotropy of rock mass strength could induce the anisotropy distribution of large deformation in space.

3. In-situ measurements

3.1. Deformation monitoring

The lining system of the tunnel was designed under safety considerations compared with the current technical standard. Table 3 lists the design parameters for the double-line tunnel in the rocks belonging to IV and V classes under high geo-stress condition. However, the deformation of the tunnel still could not be controlled. Under high ground stress conditions, the initial deformation of the surrounding rock after tunnel excavation was fast and strong and lasted for a long time. There was no tendency to converge after 30 days of excavation.

| Class of rock | Reserved deformation space (cm) | C30 Jet concrete (cm) | ø22 Bolts | ø8 Steel mesh (cm) | H150 Steel frame, spacing (m) | Second lining, C35 reinforced concrete (cm) |
|--------------|---------------------------------|-----------------------|-----------|-------------------|-----------------------------|----------------------------------|
| IV           | 30                              | 25                    | 6.0       | 1.0×0.8           | 20×20                       | 1.0                              | 55–65                            |
| V            | 35                              | 30                    | 6.0–8.0   | 1.0×0.8           | 20×20                       | 0.5                              | 60–70                            |

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As shown in Figure 5, the displacement convergence of DK187+895 increased quickly in the first 5 days after excavation, and the accumulative displacement accounted for more than 50% of the entirety. The investigation in the tunnel and displacement monitoring data revealed that horizontal convergence was much more intense than vault crown settlement in the tunnel section, thereby indicating the contradiction between horizontal and vertical displacement in the same section. The ratio between the horizontal and vertical cumulative displacement reached 4–10, which showed obvious anisotropic deformation. Furthermore, given the replacement of the surrounding rock intrusion, or the secondary disturbance caused by the step excavation, the stable section was repeatedly deformed, and the deformation law was the same as the previous deformation.

![Figure 5. Deformation monitoring in Muzhailing tunnel (DK187+895, 0)](image-url)

3.2. Confined pressure measurements

To dynamically evaluate the security of tunneling excavation via the construction method and support measures, some representative sections were selected to monitor the confined pressure before primary support and the secondary lining. The surrounding rock of these sections belonged to grade V.
As an example of confined pressure measurements, we set a monitoring section in DK187 + 895, 0. The layout of observation points of confined pressure of steel frames is shown in Figure 6. Seven monitoring points were set up in both primary support and secondary lining. Points 0-1 to 0-7 were the observation points of the primary support in the crown, spandrel, wall waist, and wall feet. Similarly, Points 1-1 to 1-7 were the observation points of the secondary lining in the same section.

![Figure 6. Confined pressure monitoring between surrounding rock and primary support](image)

**Figure 6.** Confined pressure monitoring between surrounding rock and primary support

### 4. Comparison between deformation and confined pressure

**4.1. Spatial comparison**

Initial support steel arches have excessive local load and loss of bearing capacity due to the long-term continuous deformation of the surrounding rock. The extrusion deformation of the surrounding rock directly acted on the secondary lining, causing the secondary lining of some sections to crack and block (see in Figure 7(d)). The real-time monitoring results of the contact pressure of surrounding rocks in Figure 7 indicated that the rock pressure will not only increase in quantity but also show unsymmetrical distribution due to the anisotropy of the rock shown in Figure 7(a). Under the anisotropic control of surrounding rock deformation (Figure 7(b)), the support force after tunnel excavation was extremely uneven. Moreover, the partial pressure was too large, causing the lining to break.

![Figure 7. Comparison of spatial distribution in DK187 + 895, 0 of Muzhailing Tunnel](image)

(a) carbonaceous slate in Muzhailing tunnel; b. deformation in 3DEC; c. in-situ monitoring results; d. failure phenomenon in tunnel

**Figure 7.** Comparison of spatial distribution in DK187 + 895, 0 of Muzhailing Tunnel

Based on the current surrounding rock pressure calculation formula, the vertical uniform pressure is

\[ q = \gamma H \]
where $S$ is the surrounding rock level, $\omega$ is the width influence coefficient, $\omega = 1 + i(B-5)$, $B$ is the tunnel width, and $i$ is the surrounding rock pressure increase and decrease rate. When $B > 5$ m, take $i=0.1$. Taking the calculation of the maximum buried depth of the tunnel, the statistical analysis of the existing construction showed that the variation range of the bulk density as maintained at 26–29 kN/m$^3$, and the average bulk density was calculated as 27 kN/m$^3$. Given that $q$ is the force per unit length, that is, the same as the pressure value, the vertical uniform pressure was calculated to be 0.466 MPa. The horizontal uniform pressure was $(0.3–0.5)\cdot q$, which was 0.233 MPa based on the maximum value. However, during actual monitoring, the actual value of multiple points was greater than the calculated value, and the force of different monitoring parts significantly differed, as shown in Figure 7(c).

4.2. Temporal comparison

For the railway tunnel deformation monitoring of the new Austrian tunneling method, if the displacement curve is in a stable state, then it is effective in installing the secondary lining. On the basis of the deformation monitoring curve in Figure 4, the secondary lining of the section DK187 + 895, 0 was set up after 35 days of excavation. However, the monitoring curve of the confined pressure in the primary support in Figure 8(a) showed sustainable growth in 60 days of excavation. The curve gradually became stable, and the final stability appeared after 120 days of excavation.

![Figure 8. Confining pressure–time curves at the section DK187 + 895, 0](image)

Remarkably, both Point 0-1 and 0-5 curves dropped sharply and then increased slowly to achieve stability. Compared with the monitoring curves of the secondary lining in Figure 8(b), the curves simultaneously went straight up. The steel arches of the primary support became unstable, and rock pressure was exerted on the secondary lining, especially on the crown and left spandrel of the section. Observations of rock pressure on the secondary lining proved that the contact pressure reached over 1.2 MPa, which was much higher than our own experiences.

5. Conclusions

Some lessons can be learned from a comparison of deformational behavior and confined pressure in Muzhailing tunnel of Lan-Yu railway regarding the current technical standards.

(1) The special property of slate and schistose rock with weak cement should be considered in the classification of rock. The current classification takes the rock as isotropic and with invariable mechanical property without considering any changes caused by engineering disturbances. However, the schistose rock with weak cement by flaky minerals like carbonaceous films can be easily broken into anisotropic and soft medium when excavated. There should be some rules to reflect the softening and anisotropy of rock due to changes in conditions.

(2) The deformation pressure should be considered in the rock pressure calculation for deep buried and soft rock tunnel. The current standards for tunnel design mainly consider the deformation of the primary support. However, the rock pressure for deep buried and soft rock tunnel will be much higher than the upper limit mentioned above. A big element from deformation was not involved into the rock pressure calculation, so we could not predict the large deformation and failure of surrounding rock of tunnels.
(3) An asymmetric supporting system should be carried out in the design of a tunnel lining system. Evenly distributed bolts and symmetrical supporting systems are not appropriate for controlling the deformation in schistose rock tunnels because they do not match the distribution of deformation and do not work efficiently. For instance, a bolt parallel to the schistose plane could never strengthen the connection of the slices, but one across the planes will be much more effective. This matter must be considered in the current standards.

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References
[1] Dalgãc Suleyman. Tunneling in squeezing rock, the Bolu tunnel, Anatolian Motorway, Turkey[J]. Engineering Geology, 2002, 67(2): 73-96.
[2] Zhang, G.H., Jiao, Y.Y., Wang, H., Outstanding issues in excavation of deep and long rock tunnels: a case study[J]. Canadian Geotechnical Journal. 2014, 51(9): 984-994.
[3] Cao C, Shi C, Lei M, et al. Squeezing failure of tunnels: A case study[J]. Tunnelling and Underground Space Technology, 2018, 77(2018): 188-203.
[4] Rettighieri M, Triclot J, Mathieu E, et al., Difficulties associated with high convergences during excavation of the Saint Martin La Porte access adit. In: Building underground for the future: Proceedings of the AFTES international congress, Monaco, Monte Carlo, Oct 2008. AFTES, Limonest, France, pp 395-403.
[5] Bewick, R. P., Eng, P., & Se, M. A. Influence of Rock Mass Anisotropy on Tunnel Stability. Rockeng09: Proceedings of the 3rd CANUS Rock Mechanics Symposium, Toronto, May 2009, 1-12.
[6] Zhang, C., Feng, X.T., Zhou, H., Estimation of in situ stress along deep tunnels buried in complex geological conditions[J]. International Journal of Rock Mechanics and Mining Sciences, 2012, 52(6): 139-162.
[7] Sha P, Wu F Q, Guo Q L. Anisotropic deformation in Muzhailing Railway Tunnel, Gansu, China[C] //Proceedings of the 3rd Isem Sinorock Symposium, Tongji University, Shanghai, China. Modelling and Engineering Design Methods: Rock Characterization, London: Taylor & Francis Group, 2013: 855-860.
[8] Xu D P, Feng X T, Chen D F, et al. Constitutive representation and damage degree index for the layered rock mass excavation response in underground openings[J]. Tunnelling and Underground Space Technology, 2017, 64: 133-145.
[9] Saroglou, Charalamos, Qi, Shengwen, Guo, Songfeng. ARMR, a new classification system for the rating of anisotropic rock masses[J]. Bulletin of Engineering Geology and the Environment, 2019, 78(5): 3611-3626.
[10] Nasseri M H B, Rao K S, Ramamurthy T. Anisotropic strength and deformation behaviour of Himalayan schists[J]. International Journal of Rock Mechanics and Mining Sciences, 2003, 40(1): 3-23.
[11] Tien Y M, Kuo M C, Juang C H. An experimental investigation of the failure mechanism of simulated transversely isotropic rocks[J]. International Journal of Rock Mechanics and Mining Sciences, 2006, 43(8): 1163-1181.
[12] Wang S Y, Sloan S W, Tang C A, et al. Numerical simulation of the failure mechanism of circular tunnels in transversely isotropic rock masses[J]. Tunnelling and Underground Space Technology, 2012, 32: 231-244.
[13] Chen Y F, Wei K, Liu W, et al. Experimental Characterization and Micromechanical Modelling of Anisotropic Slates[J]. Rock Mechanics and Rock Engineering, 2016, 49(9): 3541-3557.
[14] Xu G. He C. Su A., et al., Experimental investigation of the anisotropic mechanical behavior of phyllite under triaxial compression[J]. International Journal of Rock Mechanics and Mining Sciences, 2018, 104: 100-112.
[15] Cui Zhendong, Liu Da’an, Wu Faquan. Influence of Dip Directions on the Main Deformation Region of Layered Rock around Tunnels[J]. Bulletin of Engineering Geology and the Environment, 2013, 73: 441-450.
[16] Liu Gao, Zhang Fanyu, Li Xinzhao, et al. Research on Large Deformation and Its Mechanism of
Muzhailing Tunnel[J]. Chinese Journal of Rock Mechanics and Engineering, 2005, 24(S2): 5521-5526.

[17] Sha P, Wu F Q, Li Xiang, et al. Controlling Structural Deformation of Surrounding Rock Passed by a Deep-Buried Tunnel[J]. Modern Tunnelling Technology, 2018, 55(3): 112-120.

[18] Raymond L.A, Terranova T. 1984. Prologue: the mélange problem-a review[J]. Geol Soc Am Special Paper, 198: 1-5