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# Analysis of mechanical behavior of soft rocks and stability control in deep tunnels

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ABSTRACT

Due to the weakness in mechanical properties of chlorite schist and the high in situ stress in Jinping II hydropower station, the rock mass surrounding the diversion tunnels located in chlorite schist was observed with extremely large deformations. This may significantly increase the risk of tunnel instability during excavation. In order to assess the stability of the diversion tunnels laboratory tests were carried out in association with the petrophysical properties, mechanical behaviors and water-weakening properties of chlorite schist. The continuous deformation of surrounding rock mass, the destruction of the support structure and a large-scale collapse induced by the weak chlorite schist and high in situ stress were analyzed. The distributions of compressive deformation in the excavation zone with large deformations were also studied. In this regard, two reinforcement schemes for the excavation of diversion tunnel bottom section were proposed accordingly. This study could offer theoretical basis for deep tunnel construction in similar geological conditions.



## 1. Introduction

The West ends of diversion (high pressure) tunnels #1 and #2 of Jinping II hydropower station were located in the chlorite schist stratum with the length of about 400 m. This stratum is characterized with complex geological conditions, e.g. high in situ stress, and large overburden depth. The main characteristics of chlorite schist are related to the weakness in the mechanical properties, water-weakening effects and significant creep strain of rocks. Extremely large deformation was observed during construction due to the inadequate support measures, such as delayed support and low-strength support, after excavating the top section of tunnels. The significant interference of primary support with original lining section contributed to the continuously increasing deformation of rocks. This considerably increases the risk of instability of surrounding rock mass when excavating the bottom section of tunnels, resulting to a problem in the power generation capacity due to reduction of the tunnel cross-section.

Many definitions and/or concepts regarding soft rocks have been proposed (Fan, 1995; Guo, 1996; Lin, 1999). According to the work of Sciotti (1990), soft rocks, i.e. sandstone (Nickmann et al., 2006) and mudstone (Yoshinaka et al., 1997), have the main characteristics such as large deformability, strong dependence of resistance on degree of saturation or temperature, and susceptibility to alteration. For simplicity, soft rocks have been classified into two sets (Clerici, 1992; Russo, 1994): geological soft rock and engineering soft rock. The set of the geological soft rock has the intrinsic properties of weakness, looseness and expansibility, while the engineering soft rock generates significant plastic strain and creep strain subjected to engineered effect. The chlorite schist of Jinping II hydropower station can be viewed as a geological soft rock due to its weakness in mechanical properties, but also as the engineering soft rock due to high in situ stress at depth of approximately 1500 m.

Excavating tunnel in soft rock stratum usually will cause accident due to the complex geological conditions and mechanical behaviors of soft rocks. Many methods of support techniques have been proposed consequently. For example, the New Austrian Tunneling Method (NATM) (Han, 1987) which is also known as sequential excavation method (SEM) is a popular method in modern tunnel design and construction. Salamon (1970) studied the support system in terms of energy. The support structure and

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surrounding rock simultaneously generate compatible deformation and the support structure can absorb part of dispersed energy from surrounding rock mass. The combined support method (Feng, 1990) proposed that increasing the thickness of support was not the optimum method for tunneling in soft rocks, and the method of pre-flexibility and post-stiff was of priority. He (1994) suggested that the support technique for deep tunneling should be carried out in two steps: first flexible support and a coupling support followed for the critical parts. Dong et al. (1994) also proposed support theory for the loose circle in surrounding rock mass.

The support system for tunneling in soft rock is still a challenging issue due to the special mechanical behaviors of soft rocks. Therefore, the stability of the diversion tunnels passing through the chlorite schist stratum of Jinping II hydropower station should be analyzed. Based on this, laboratory tests were conducted to study the petrophysical properties, mechanical behaviors and water-weakening properties of chlorite schist. In addition, the creep deformation of surrounding rock mass, destruction of support structure and large-scale collapse induced by high in situ stress associated with mechanical properties of chlorite schist were analyzed. Accordingly, the excavation and reinforcement schemes for the diversion tunnels construction were proposed.

## 2. Project overview

Jinping II hydropower station is located at the Yalong River, Sichuan Province, China. The Jinping II hydropower station is well known as a super-large hydropower project due to the ultra-long, deep-buried and large-scale tunnel section. The four parallel diversion tunnels of Jinping II hydropower station are 16.7 km in length and the spacing between two adjacent tunnels is about 60 m. The tunnels 2 and 4 were excavated by drilling-and-blasting method in two steps, while tunnels #1 and #3 by TBMs. Full-section reinforced concrete tunnel support structure with thickness of 50–140 cm was used. The overburden depth along the tunnel is basically 1500–2000 m and the maximum depth is 2525 m.

The West ends of the diversion tunnels #1 and #2 were located in chlorite schist stratum in the stakes of (1) 1 + 537 and (2) 1 + 613 at the depth of about 1500 m (see Fig. 1). The chlorite schist is complex for geological conditions and frequent changes in lithology can be observed. The rocks are mainly green sandstone, green sandstone–marble interbedding, chlorite schist–marble interbedding and chlorite schist. The accumulated lengths of chlorite schist–marble interbedding and chlorite schist in the diversion tunnels #1 and #2 are 264 m and 314 m, respectively. The existence of chlorite schist–marble interbedding and chlorite schist strata caused the major collapse and destruction of primary support due to large deformation of surrounding rock mass in large area. Moreover, large deformation of surrounding rock mass in a large area was also observed in the second excavation (enlarged) and some local large deformation once again caused the destruction of support structure.

## 3. Physical properties of chlorite schist

Creep deformation, destruction of support structure and large-scale collapse were observed during excavation due to the weakness of mechanical properties of chlorite schist and high in situ field stress. These problems should be addressed in order to maintain the progress of project construction. Therefore, related petrophysical tests were carried out to analyze the physical properties and mineral compositions of chlorite schist, which could be helpful to investigate the mechanical behavior and the stability assessment of the diversion tunnels. All the samples are from the stake of (1) 1 + 760.

### 3.1. Mineral composition

Four typical samples of chlorite schist were used to identify the mineral composition and the results are shown in Table 1. It clearly shows that the main mineral component is chlorite, which has an evident property of water-argillization. This property is one of the most important factors that cause water-weakening and large deformation of soft rocks. The second main component is amphibole, whose Mohs mineral hardness is twice of the one of chlorite. Some talc is found in all the four samples. Talc has the minimum Mohs mineral hardness among all the mineral components; the sample with the maximum composition of talc thus has the minimum elastic modulus and strength. Moreover, small quantity of calcite is also observed in the sample of IRSM #1. The sample of IRSM #4 contains a little more calcite and some dolomite. This phenomenon can be explicated by the inclusion of marble in the samples of chlorite schist.

### 3.2. Microstructures of rock samples

Scanning electron microscopy (SEM) was used to observe the microstructures of the four samples in order to study the deformation mechanism of chlorite schist and the results are shown in Fig. 2. Except for the sample of IRSM #3, the other three samples have similar mineral arrangement and porous structure. The mineral grains are arranged in orientated layered sheet, resulting in tight porosity structure and low porosity. The sample of IRSM #3 presents disordered mineral arrangement and its porosity is formed by the connection of mineral grains in a dense type. The microstructure of chlorite schist revealed by SEM shows that the samples have low porosity and low permeability.

### 3.3. Expansibility

The mineral composition in Table 1 shows that no hydrophilic expansion mineral is observed in the samples; the expansibility of chlorite schist is thus not significantly different from the first estimation. The following expansion tests were performed to investigate the free expansion ratio, axial expansion ratio with lateral restraint and expansion pressure. The results are shown in

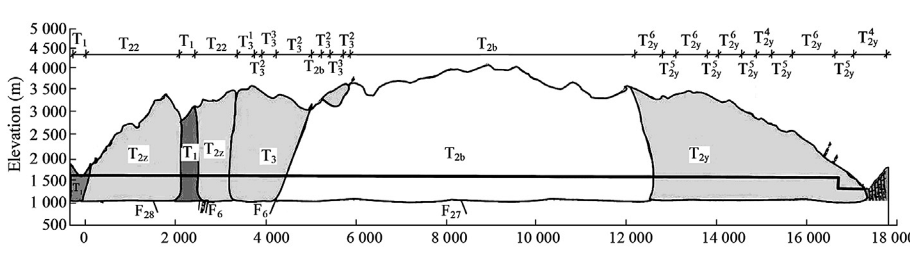



**Fig. 1.** Geological profile along diversion tunnel of Jinping II hydropower station. $T_1$: chlorite schist, $T_{2z}$: marble, $T_3$: sandy slate, $T_{2b}$: marble, $T_{2y}$: marble.

**Table 1**
Mineral identification results of chlorite schist.

| Sample No. | Chlorite (%) | Talc (%) | Amphibole (%) | Calcite (%) | Dolomite (%) |
|---|---|---|---|---|---|
| IRSM #1 | 39 | 14 | 39 | 8 | 0 |
| IRSM #2 | 47 | 23 | 30 | 0 | 0 |
| IRSM #3 | 39 | 36 | 25 | 0 | 0 |
| IRSM #4 | 25 | 22 | 25 | 12 | 16 |

Table 2. The average values of axial expansion ratio, lateral expansion ratio, volumetric expansion ratio, axial expansion ratio with lateral restraint, and expansion pressure are 0.077%, 0.117%, 0.311%, 0.102% and 15.35 kPa, respectively. This confirms the primary estimation of low expansibility of chlorite schist.

## 4. Mechanical behaviors and water-weakening properties of chlorite schist

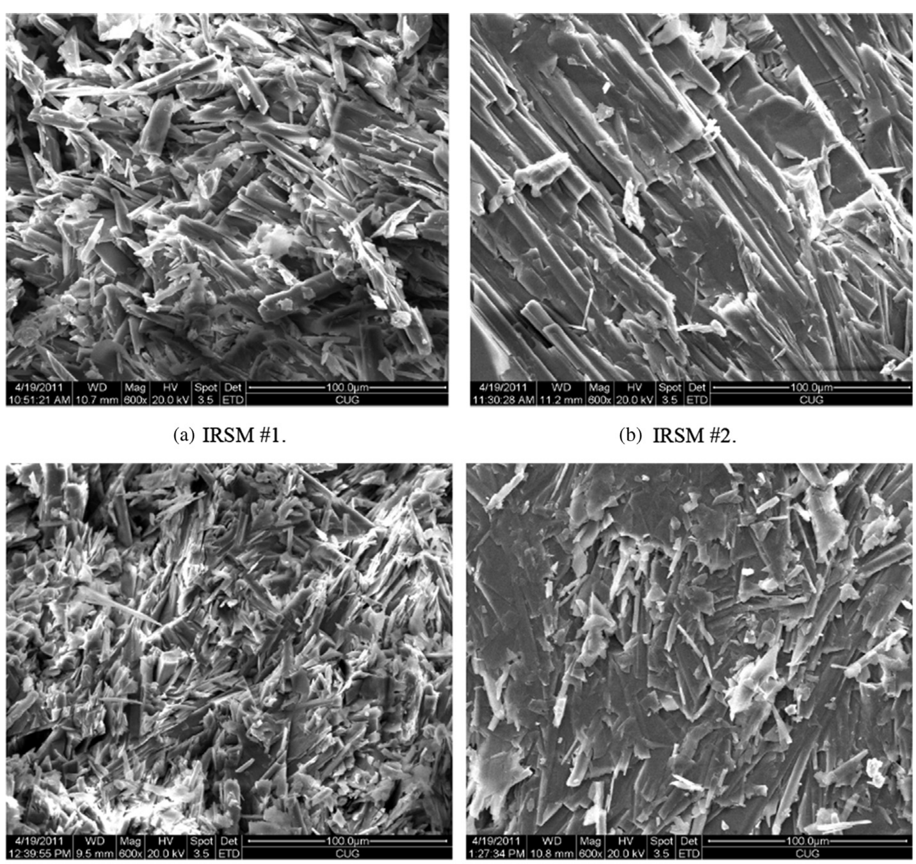
In order to investigate the shrinkage of diversion tunnel cross-section area and determine the excavation and support program, a series of tests, including classical triaxial compression tests and direct shear tests, were carried out on dry and saturated samples. The standard cylinder rock samples with size of $\phi$50 mm × 100 mm were prepared and at least three samples were used at each loading condition. The mechanical behaviors of the two kinds of samples were compared based on the effect of water on the deformation and strength.

### 4.1. Classical triaxial compression tests

The results of classical triaxial compression tests of chlorite schist for samples #28, #35, #45, #47 and #51 are shown in Fig. 3. The studied samples present significant strain hardening and softening behaviors, in association with confining pressure when considered. The axial stress undergoes a sharp drop under low confining pressure. With the increasing confining pressure, this axial stress drop disappears gradually and perfect plasticity is even observed under the confining pressure of 40 MPa.

Fig. 4 shows the evolutions of triaxial compressive strength and elastic modulus with confining pressure. It clearly shows that the failure surface coincides with Mohr–Coulomb linear yield function. The mechanical parameters of saturated samples are lower than those of dry samples. Moreover, the elastic modulus increases with confining pressure due to compaction of the samples during the loading of confining pressure.

The degradation ratio is defined as the ratio of triaxial compressive strength or elastic modulus of saturated sample to the one of the dry sample. The degradation ratios of triaxial compressive strength and elastic modulus between saturated and dry samples are illustrated in Fig. 5. The water-weakening effect is not obvious with the increasing confining pressure. The confining pressure near the tunnel is reduced after excavation, thus the significant degradation in strength and increase in deformation may occur once the surrounding rock in this zone encounters water. After construction of support structure and grout injection, the confining pressure may recover to some extent, which is helpful for the stability of surrounding rocks.




(c) IRSM #3. (d) IRSM #4.

**Fig. 2.** Microstructure of the four samples by SEM.

**Table 2**
Expansion test results of chlorite schist.

| Sample number | Free expansion ratio (%) | | Axial expansion ratio with lateral restraint (%) | Expansion pressure (kPa) |
|---|---|---|---|---|
| | Axial | Lateral | | |
| IRSM #11 | 0.034 | 0.193 | 0.062 | 11.078 |
| IRSM #12 | 0.154 | 0.101 | 0.120 | 19.436 |
| IRSM #13 | 0.043 | 0.057 | 0.123 | 15.538 |

It is noted that the triaxial stress was imposed on saturated samples without water drained, thus excess pore pressure may be generated within the samples when applying confining pressure. However, water within the cracks can be drained in surrounding rocks, the water-weakening effect is thus significantly reduced and the compaction caused by confining pressure is more obvious.

### 4.2. Direct shear tests

Water mainly exists in original cracks and joints as well as induced cracks due to low porosity and low permeability of intact chlorite schist. These cracks and joints are caused by the shear fracture and then affect the strength of rock mass structure. Therefore, the direct shear tests of single fracture were performed for both dry and saturated samples in order to compare their mechanical responses and to estimate water-weakening effect on the mechanical property of single shear fracture.

First, the direct shear tests of the original samples were conducted to generate shear fracture. Then the direct shear tests of single fracture were performed. The average shear strength parameters of dry and saturated samples are given in Table 3. It clearly shows that the water-weakening effect results in a significant decrease in cohesion by about 55.58%. This can be attributed to softening effect of water on the roughness of fracture surface, and the uplift part of fracture surface has less resistance in presence of water. The cohesion of samples is thus significantly decreased and increases the collapse risk of jointed rock mass. Fig. 6 shows the relationship between the degradation ratio of shear strength and normal stress. The degradation ratio is gradually increased with the increasing normal stress.

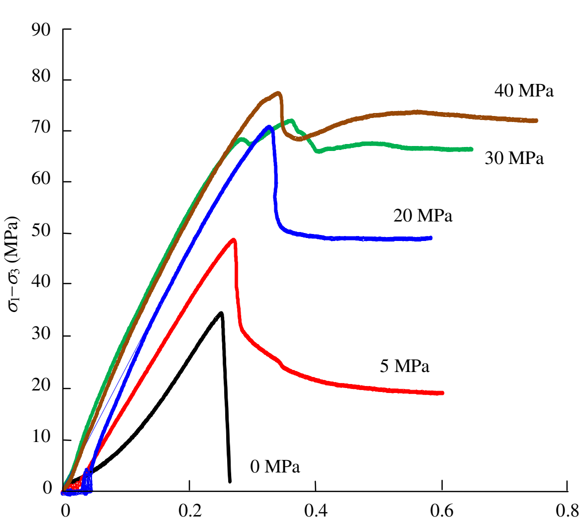



**Fig. 3.** Typical stress–strain curves of classical triaxial compression tests for dry samples #28, #35, #45, #47 and #51 under confining pressure of 0 MPa, 5 MPa, 20 MPa, 30 MPa and 40 MPa.

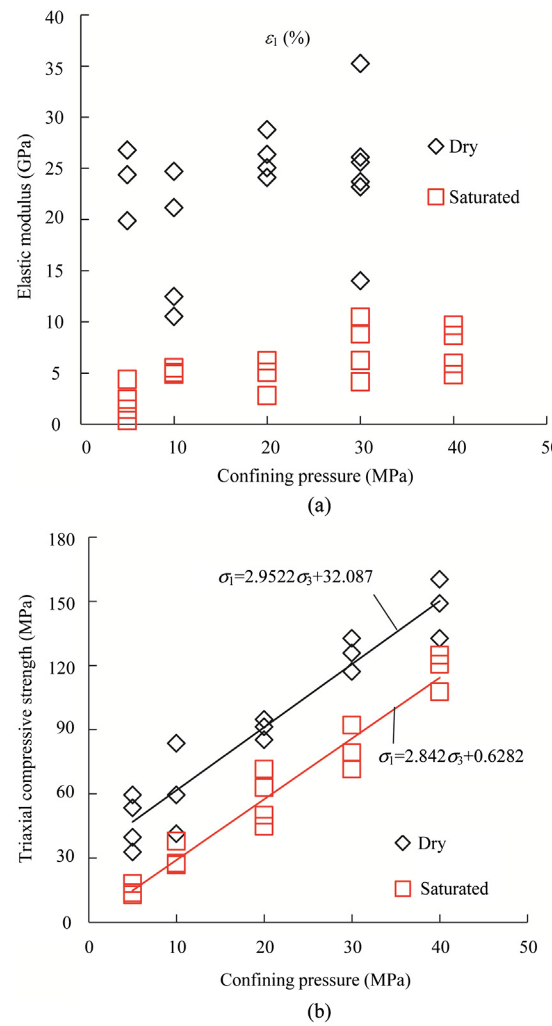



**Fig. 4.** Evolutions of elastic modulus and triaxial compressive strength with confining pressure. (a) Relationship between elastic modulus and confining pressure. (b) Relationship between triaxial compressive strength and confining pressure.

### 4.3. Summary of engineering mechanical properties of chlorite schist

The mechanical behavior of chlorite schist has a close relation with confining pressure. The triaxial compressive strength and elastic modulus both increase with confining pressure. The in situ stress of surrounding rocks is decreased during field stress evolution (redistribution) after excavation, the elastic modulus is thus significantly decreased and large elastic deformation is generated. In this instance, large deformation of the diversion tunnels in association with the plastic deformation was caused by large-scale excavation damaged zone. Therefore, the support structure is critically important for cracked surrounding rocks. Moreover, the existing water has a significant effect on the degradation in strength and the increase in deformation, but a desirable confining pressure can reduce the water-weakening effect and facilitate the discharge of water within the cracks.

## 5. Large deformation of diversion tunnels in Jinping II hydropower station

The maximum deformation of many sections after the primary support reached 0.5–0.7 m, which is 7.6%–10.6% of the radius of

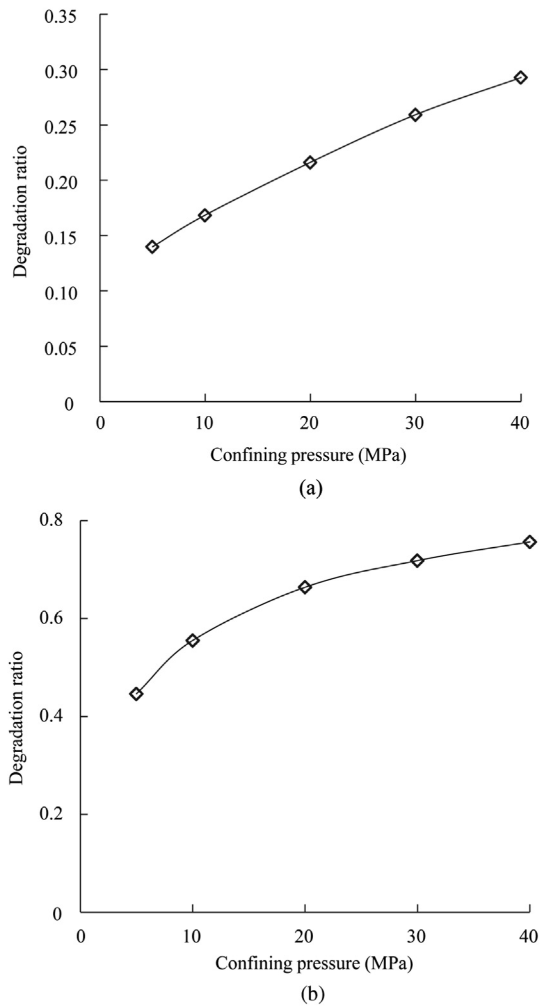



**Fig. 5.** Evolution of degradation ratios of elastic modulus and triaxial compressive strength with confining pressure. (a) Degradation ratio of elastic modulus. (b) Degradation ratio of triaxial compressive strength.

tunnel. Hoek and Brown (1980) and Barla (1995) have proposed assessment methods (see Table 4) of compressive deformation of rock mass based on empirical statistical data of large deformation of soft rocks. The method proposed by Hoek and Brown (1980) is adopted in the present study due to its advantage in classification.

Fig. 7 presents a chart of percentage of compressive deformation in the zone of the diversion tunnels #1 and #2. For two tunnels, the sectional percentages of severe and extremely severe compressive deformation are up to 74.36% and 58.93%, respectively, while the ones of moderate compressive deformation are 23.08% and 37.5%, respectively. Therefore, excavation in chlorite schist stratum generated severe compressive deformation according to the method proposed by Hoek and Brown (1980). Insufficient pre-support and delay of support structure may not efficiently control the development of plastic zone, and result in large deformation and even collapse.

**Table 3**
Average shear strength parameters of dry and saturated samples.

| Sample | Cohesion $c$ (MPa) | Friction coefficient | Internal friction angle $\varphi$ (°) |
|---|---|---|---|
| Dry | 4.21 | 0.35 | 19.30 |
| Saturated | 1.87 | 0.31 | 17.23 |
| Degradation ratio (%) | 55.58 | 11.43 | 10.71 |

Note: Degradation ratio = (dry − saturated)/dry × 100%.

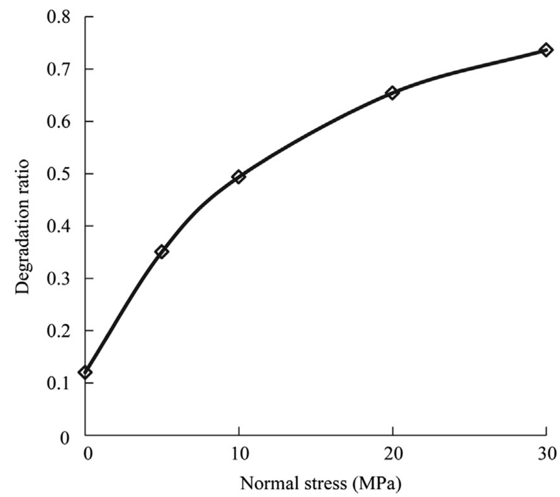



**Fig. 6.** Evolution of degradation ratio with the normal stress.

**Table 4**
Assessment of compression extent of rock mass.

| Assessment method | Large deformation extent (%) | | | |
|---|---|---|---|---|
| | Slight | Moderate | Severe | Extremely severe |
| Hoek and Brown (1980) | 1–2.5 | 2.5–5 | 5–10 | >10 |
| Barla (1995) | 1–3 | 3–5 | >5 | – |

## 6. Reinforcement schemes of top section before excavating bottom section

Tunnels #1 and #2 were excavated by drilling-and-blasting method and applied by reinforced concrete lining. After the excavation and introducing the support of the tunnel top sections in

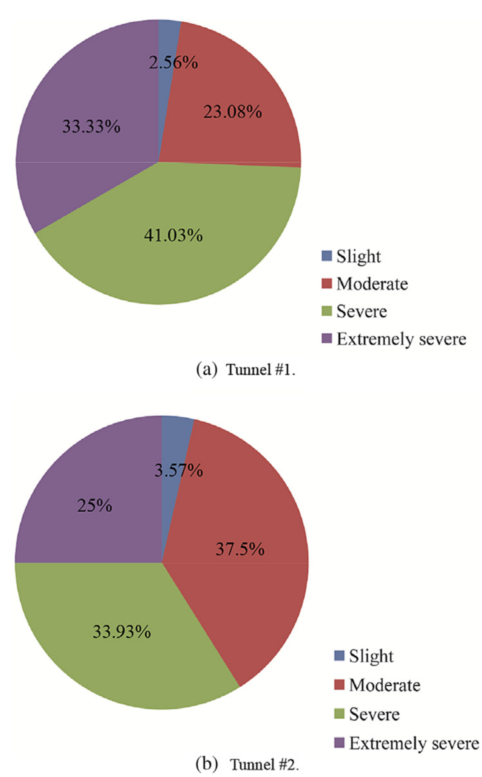



**Fig. 7.** Percentage of different levels of compressive deformation.

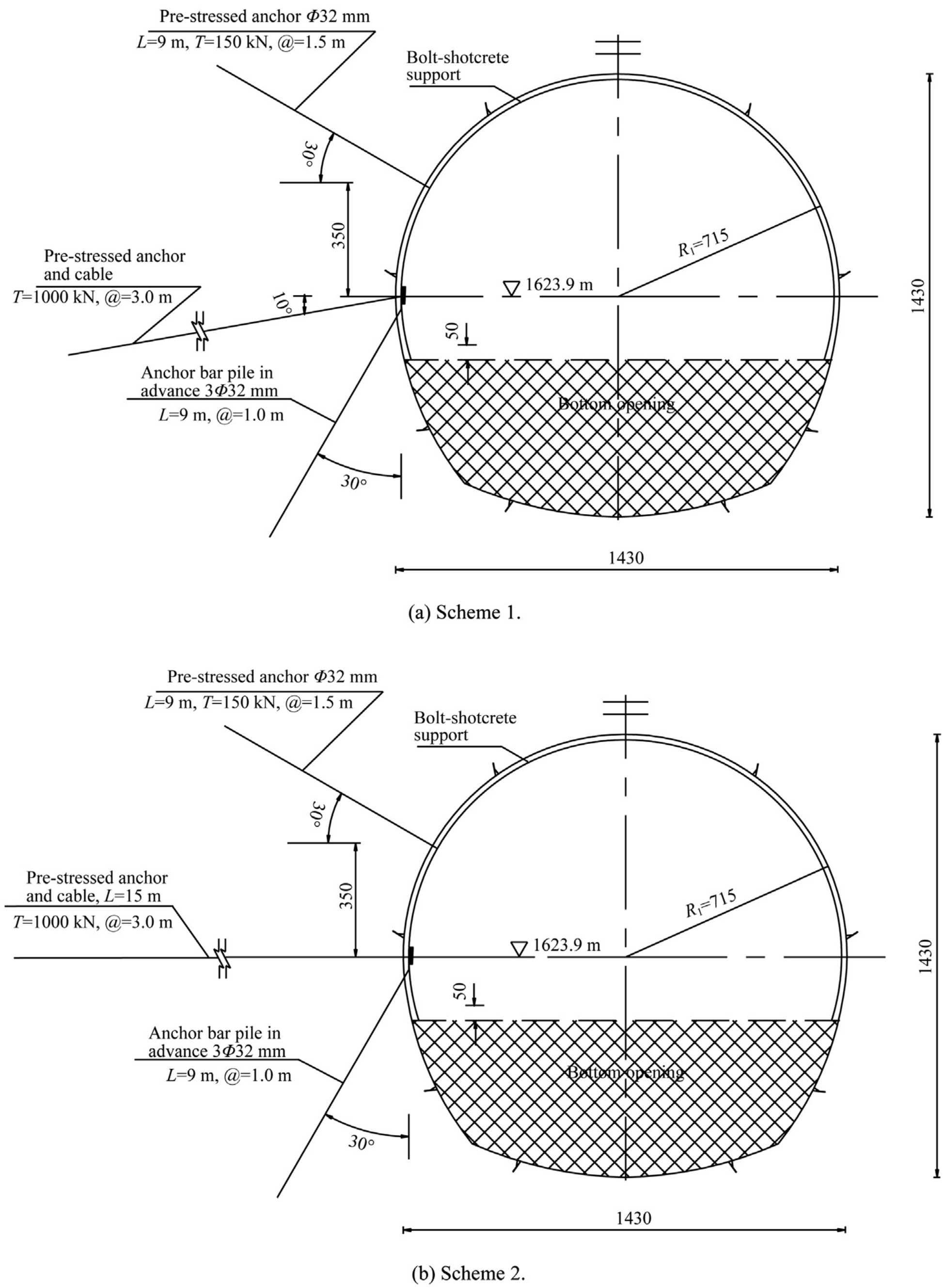


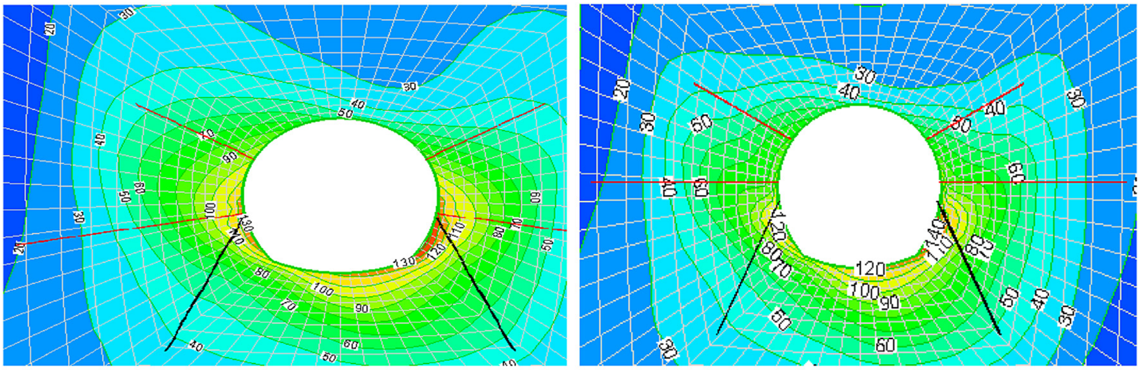

**Fig. 8.** Layout of reinforcement schemes of tunnel bottom (unit: mm).

chlorite schist stratum, the stresses obtained in the anchors were considerably high at the chlorite formations, and some of the anchors may reach their design strength. If there is no more support, larger deformation may occur in the top section of surrounding rocks and primary support will lose vertical reinforcement function at the foot section. This may even induce collapse due to the loss of vertical reinforcement. Therefore, the support principle of the top section before excavating the bottom section is to keep the top section stable and control floor heave.

Two design schemes were accordingly proposed to control the deformation of top section before excavating bottom section (see Fig. 8). The two schemes adopted anchor bar pile in advance at arch foot (3$\phi$32 mm, $L = 9$ m and @ = 1.0 m) and pre-stressed anchor at spandrel ($\phi$32 mm, $L = 9$ m, $T = 150$ kN and @ = 1.5 m), where $\phi$, $L$, $T$ and @ denote the diameter, length, pre-stress and spacing of anchors, respectively. Scheme 1 used downward pre-stressed anchor cable at side wall of central section ($L = 15$ m, $T = 1000$ kN and @ = 3.0 m), while scheme 2 used horizontal pre-stressed anchor cable at side wall of central section ($L = 15$ m, $T = 1000$ kN and @ = 3.0 m). The prediction of deformation was obtained using FLAC$^{3D}$ models. The in situ stresses determined of surrounding

**Table 5**
In situ stresses of surrounding rocks (MPa).

| $\sigma_x$ | $\sigma_y$ | $\sigma_z$ | $\tau_{xy}$ | $\tau_{yz}$ | $\tau_{xz}$ |
|---|---|---|---|---|---|
| −32.76 | −39.23 | −37.64 | −4.21 | −4.68 | −2.84 |



(a) Scheme 1. (b) Scheme 2.

**Fig. 9.** Deformation distribution under two reinforcement schemes (unit: mm).

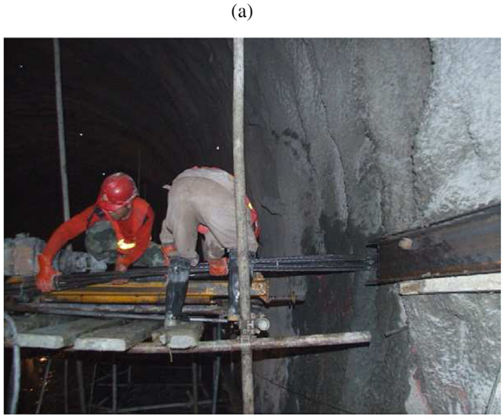
**Table 6**
The predicted deformations of the tunnel section after excavating the bottom section with two reinforcement schemes.

| Scheme | Maximum deformation and position | | | |
|---|---|---|---|---|
| | Zone with extremely severe compressive deformation less than 50 cm | | Zone with extremely severe compressive deformation more than 50 cm | |
| | Above mid part of side wall | Below mid part of side wall | Above mid part of side wall | Below mid part of side wall |
| 1 | 10 cm, mid part of the two side walls | 12 cm, mid part of the right arch foot | 11 cm, mid part of the two side walls | 14 cm, mid part of the right arch foot |
| 2 | 5 cm, mid part of the two side walls | 10 cm, mid part of the right side wall | 9 cm, mid part of the two side walls | 14 cm, mid part of the right side wall |

rocks are listed in Table 5. The predicted deformations of the tunnel section after excavating the bottom section with two reinforcement schemes are shown in Fig. 9 and Table 6. The deformation of surrounding rocks is 5–15 cm after the excavation of the tunnel bottom, and the distribution of deformation is anisotropic but follows an obvious law. The largest displacement occurs at the lower section of the side wall, and significant upheaval is observed at the side wall of right bottom and bottom plate, so immediate support is necessary to form a closed-loop system. The deformation of mid and top sections is smaller than the one of bottom section. Anchor bar pile in advance at arch foot is helpful for the stability of surrounding rocks. In this case, the lower section of the side wall can be reinforced and it plays an important role in the connection and support of the upper section arch.

The suitable anchor bar pile spacing is about 1.0 m. With the grouting pressure at the zone of cracked surrounding rock mass, pre-support stripe at the arch foot ensures the stability of temporary side wall at the bottom. Pre-stressed anchor and cable decrease the upward displacement induced by excavation of tunnel bottom section. The predicted results show that scheme 2 with horizontal arch cable is better than scheme 1. Therefore, scheme 2 was used to reinforce the top section at construction site (see Fig. 10). The observational results after excavating tunnel bottom section reveal that the deformation of surrounding rock mass can be controlled within an acceptable level when considering scheme 2.

(a)



(b)

**Fig. 10.** Scheme 2 at the construction site. (a) Anchor steel-stake in advance at arch foot. (b) Pre-stressed anchor and cable at mid part of side wall.

## 7. Conclusions

Laboratory tests on chlorite schist were carried out in order to understand the deformation distribution of diversion tunnels and to select a proper reinforcement scheme for the deep tunnels in Jinping II hydropower station crossing the chlorite soft rocks. The following conclusions could be drawn:

(1) Petrophysical tests show that chlorite schist has tight porous structure and low porosity, resulting in low porosity and low permeability. No hydrophilic expansion mineral was found and the expansibility of chlorite schist was not significant.
(2) Mechanical tests indicate that the triaxial compressive strength and elastic modulus increase with confining pressure. The presence of water has a significant effect on the degradation in

strength and the increase in deformation, thus a desirable confining pressure can reduce the water-weakening effect and facilitate the discharge of water within the cracks.

(3) The distributions of compressive deformation based on empirical statistical data show that excavation in the formation of chlorite schist can generate large compressive deformation. The insufficient pre-support and/or delay of support structure may not efficiently control the development of plastic zone, resulting in large deformation and even collapse.

(4) Two reinforcement schemes of the top section excavation were proposed to control the deformation induced by the bottom excavation. Anchor bar pile in advance at arch foot is helpful for the stability of surrounding rocks. Scheme with horizontal arch cable is prior to the one with downward arch cable.

## Conflict of interest

We wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

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