Research Article

The Influence of Moisture Content on the Time-Dependent Characteristics of Rock Material and Its Application to the Construction of a Tunnel Portal

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Uniaxial compression creep experiments were carried out for low-grade metamorphic slate samples (located in the southeastern area of Guizhou province, China) with different moisture contents, using an Instron electric-fluid servo-compression machine. Based on the experimental results, a detailed analysis was made of the effect of moisture content on the strength and deformation behaviour of the slate specimens. The three-parameter generalised Kelvin model was identified to describe the creep behaviour of the low-grade metamorphic slate with different moisture contents. There is an approximately linear negative correlation between the elastic modulus and the saturation degree, and the viscoelastic modulus and viscosity coefficient show a negative exponent correlation with the saturation degree. The Kelvin creep model considering the moisture degradation effect was established and a three-dimensional finite difference model was developed with the software FLAC^{3D} to validate the creep model. A three-dimensional numerical analysis was then performed to simulate the tunnel excavation process. The results show that the influence of moisture and creep of the surrounding rocks is needed for estimating the deformation of complex tunnel portals.

1. Introduction

The influence of moisture on the strength and deformability of rock has been analysed by many researchers [1–3]. Dyke and Dobereiner [4] demonstrated the variation in uniaxial compressive with moisture content for three quartz arenites ranging in dry strength from approximately 34 to 70 MPa. They concluded that in general the weaker the rock is, the more sensitive it is to changes in moisture content. Atkinson and Meredith [5] investigated experimentally the effect of water on subcritical microcrack growth parameters of granitic rocks. They found that the increase of humidity does not have a significant influence on the subcritical microcrack growth index n but results in a large increase in microcrack growth velocity. Golshani et al. [6] checked the effect of moisture conditions on the time-dependent behaviour of Inada granite. The experimental and theoretical results have shown that the time-dependent behaviours of rocks are seriously affected by water [7, 8].

Tunnel entrance construction presents challenges and because of its complexity it could easily result in landslides or collapse [9–13]. Generally, the cover depth at the tunnel entrance is shallow as it often moves through a mountain surface layer whose rocks are usually broken and seriously weathered. So, the surrounding rocks at a tunnel portal are easily affected by surface water [14], which makes the rheological properties of surrounding rocks more obvious and involves many different complicated problems due to the uncertainty associated with the response of the rocks to the process of excavation and construction [15].

In this study, uniaxial compression creep experiments were carried out for the low-grade metamorphic slate samples (the surrounding rock of the Ruipo tunnel portal, which is located in the southeastern area of Guizhou province, China) with different moisture contents using an Instron electric-fluid servo-compression machine. A detailed analysis was made of the effect of moisture content on the strength and deformation behaviour of slate specimens. Then

FIGURE 1: X-ray diffraction figures of low-grade metamorphic slate: (a) Specimen 1; (b) Specimen 2.

a three-dimensional finite difference analysis was executed on expressway tunnel portal areas while considering tunnel excavation and the moisture degradation effects on creep characteristics of surrounding rocks. In addition, the results were compared with those obtained by common elastoplastic models.

2. Compression and Creep Tests

2.1. Specimen Preparation and Experimental Procedure. Low-grade metamorphic slate, a quintessential rock of the Shui-Du expressway that was taken from the Ruipo tunnel portal, has a cataclastic texture. X-ray diffraction analysis reveals that, apart from the most common quartz, kaolinite and muscovite are present in the collected low-grade metamorphic slate (see Figure 1). Kaolinite has a rough surface and perfect cleavage, strong water absorption, no water swelling, great plasticity in wet conditions, and a great influence on rock strength. There were 21 cylindrical test specimens, namely, S-1 to S-21, all 50 mm in diameter and 100 mm in length, which was in accordance with the tested sample size suggested by the ISRM [16]. Three degrees of saturation of specimens were considered for experiments and 6 specimens were prepared at each degree of saturation.

3 specimens, S-1, S-2, and S-3, were used to obtain the degree of saturation at different times. Nature dry specimens were considered as the first degree of saturation, specimens submerged in water for 24 hours represented the second degree, and those that remained submerged in water for 240 hours represented the third degree of saturation. The specimen preparation process was as follows. First, all specimens were nature-dried for 48 hours, the weights of S-1, S-2, and S-3 were measured, and S-4 to S-9 were selected as the specimens at the first degree of saturation. Second, all specimens except S-4 to S-9 were submerged in water, and the weights of S-1, S-2, and S-3 were measured every 24 hours. After having been submerged in water for 24 hours, S-10 to S-15 were selected as the specimens at the second degree of saturation, and S-16 to S-21 were specimens at the third degree of saturation after having been submerged in water for 240

FIGURE 2: Specimen sealed up with wax.

hours. In order to avoid water loss from the specimens during compression testing, S-10 to S-21 were sealed up with wax (see Figure 2). Then, the dried weights of S-1, S-2, and S-3 were obtained after having been oven-dried for 24 hours at 105°C. Finally, S-1, S-2, and S-3 were saturated using the vacuum saturation method [17]. The samples were placed in the vacuum saturation apparatus and a sufficient amount of water at approximately 25°C was added to cover the sample. Entrapped air was removed from the samples by applying a residual manometer pressure of 100 kPa. The pressure was maintained for 4 hours and then the saturated weights of samples were measured.

FIGURE 3: Curves showing relation between saturation degree of specimens and time submerged in water.

Uniaxial creep experiments were performed on an Instron 1346 electronic hydraulic servo-controlled testing machine, the loading capacity of which is 2 MN. Before the creep tests, uniaxial compression tests were performed on two specimens at each degree of saturation, and triaxial compression experiments were also carried out at each degree of saturation for specimens under three different confining pressures σ_3 , that is, 2, 5, and 10 MPa. The instantaneous failure strengths and corresponding peak strains were obtained at failure of these specimens, and creep tests were planned from a knowledge of these values. The multilevel test was used: the load was applied at low levels, rising to a high level on the same specimen. The duration of tests was dependent on the displacement rates. In the case when the rate of displacement was less than 0.001 mm/h, the creep under this load level was considered to be basically stable [18]. The next load level could be executed afterwards, and the loading rating was 500 N/s.

2.2. Experimental Results and Discussion

2.2.1. Relationship between Saturation Degree and Time Submerged in Water. The degree of saturation of specimens at different times can be defined as

$$S_t = \frac{m_t - m_d}{m_s - m_d} \times 100, \quad (1)$$

where m_t is the weight of the specimen after being soaked for a period of time; m_d and m_s are the dry weight and saturated weight of the specimen, respectively.

The experimental results showed that the degree of saturation increased with the time submerged in water (see Figure 3), and the average degrees of saturation of specimens were 41.5%, 80.1%, and 93.5% when submerged in water for 0, 24, and 240 hours, respectively.

2.2.2. Mechanical Parameters Obtained from Uniaxial and Triaxial Compression Tests. Table 1 lists the mechanical parameters of specimens at three saturation degrees. The uniaxial

compression strength of specimens with saturation degrees of 80.1% and 93.5% is, respectively, 69.85 and 58.74 MPa, which decreases by 24.2% and 36.2% compared with nature dry specimens, as well as the fact that elastic modulus has a reduction of 64.4% and 83.8% correspondingly. As a conclusion, a great decrease could be found in elastic modulus when compared with strength, and the increase of moisture causes an increase in Poisson's ratio.

For the triaxial test, the linear Mohr-Coulomb criterion, which can be expressed simply as a linear relationship between peak axial stress σ_1 and confining pressure σ_3 , that is, $\sigma_1 = \{(1 + \sin \varphi)/(1 - \sin \varphi)\}\sigma_3 + 2c \cos \varphi/(1 - \sin \varphi)$, is selected here to determine the peak strength parameters of cohesion c and internal friction angle φ [19]. The values of c and φ for the specimens in these tests decreased with the increase of saturation degree (see Figure 4). Several studies [20, 21] concluded that there was usually an exponential relation between the strength of rock material and moisture. The values of c and φ of the slate obtained by Li et al. [20] also had an exponential dependency on the rock moisture, and so the exponential function was chosen to describe the relationship between strength parameters and degree of saturation:

$$\begin{aligned} c &= 32.12e^{-0.0055S}, \\ \varphi &= 39.68e^{-0.0052S}, \end{aligned} \quad (2)$$

where S is the saturation degree of specimens (%).

2.2.3. Creep Behaviour. Uniaxial experimental creep results for specimens are shown in Figure 5. Overall, transient and creep axial strains of the specimens increase with degree of saturation. At $\sigma_1 = 25.5$ MPa, the transient axial strain of the three specimens was, respectively, 0.073% ($S = 41.5\%$), 0.074% ($S = 80.1\%$), and 0.413% ($S = 93.5\%$), and the axial strain of specimens increased by 0.003%, 0.026%, and 0.066% correspondingly after creeping for 12 hours. At $\sigma_1 = 35.7$ MPa, the transient axial strain of the three specimens was, respectively, 0.092% ($S = 41.5\%$), 0.137% ($S = 80.1\%$), and 0.582% ($S = 93.5\%$), and the axial strain of specimens increased by 0.003%, 0.080%, and 0.041% correspondingly after creeping for 12 hours. When reaching the failure stress level, the specimens showed tertiary creep behaviour and succumbed to creep failure. The failure stress level of specimens was the highest while the saturation degree was 41.5% (66.2 MPa) and then decreased when the moisture content was higher (56.1 and 38.2 MPa for specimens with $S = 80.1\%$ and 93.5%).

From the compression and creep tests results, it can be seen that the moisture has a great effect on the strength and deformability of the low-grade metamorphic slate. Hawkins and McConnell [3] studied the sensitivity of sandstone strength and deformability to changes in moisture content. They concluded that the sensitivity increases progressively with higher proportions of clay minerals and rock fragments.

TABLE 1: Mechanical parameters of specimens at three degrees of saturation.

Submerged time/hours	Degree of saturation/%	Uniaxial compressive strength/MPa	Poisson's ratio	Elasticity modulus/GPa	Triaxial compressive strength Confining pressures/MPa	Peak strength/MPa	Cohesion <i>c</i> /MPa	Internal friction angle φ /($^{\circ}$)
0 (nature dry)	41.5	88.80	0.30	50.56	2	97.12		
		95.42	0.32	54.83	5	106.61	25.44	31.84
		Average	Average	Average	10	124.53		
24	80.1	68.92	0.41	17.64	2	73.84		
		70.78	0.41	19.86	5	82.69	21.32	26.91
		Average	Average	Average	10	96.21		
240	93.5	58.34	0.46	8.32	2	62.18		
		59.13	0.47	8.73	5	69.95	18.91	24.07
		Average	Average	Average	10	82.40		

FIGURE 4: Relationships between the peak strength parameters and saturation degree of specimens: (a) c - S ; (b) φ - S . FIGURE 5: Curves showing relation between strain and time of specimens with different saturation degrees: (a) $S = 41.5\%$; (b) $S = 80.1\%$; (c) $S = 93.5\%$.

The low-grade metamorphic slate studied in this paper has 31% sericite and 10% siliceous cement, which have similar characteristics with clay minerals. Another important reason lies in the fact that the well-developed fissures might make it much easier for water to get into the rock specimens.

3. Creep Model and Validation

3.1. Creep Model considering Moisture Degradation Effect. From Figure 5, the deformation of specimens reached stability after a certain period of time and the strain rate reduced to zero. Therefore, the three-parameter generalised Kelvin

TABLE 2: Creep parameters of specimens with different moisture contents.

Submerged time/hours	Degree of saturation/%	Elastic modulus E_1 /GPa	Viscoelastic modulus E_2 /GPa	Viscosity coefficient η /GPa·h
0 (nature dry)	41.5	39.01	1536.43	2339.84
24	80.1	20.46	148.60	229.78
240	93.5	6.20	76.24	161.62

FIGURE 6: The three-parameter generalized Kelvin model.

model (see Figure 6), which connects in series an elastic component (Hookean body) and a Kelvin model, can be used to describe the deformation characteristics [22]. The one-dimensional creep equation of the Kelvin model is

$$\varepsilon = \frac{\sigma}{E_1} + \frac{\sigma}{E_2} \left(1 - e^{-(E_2/\eta)t}\right), \quad (3)$$

where E_1 , E_2 , and η are the elastic modulus, the viscoelastic modulus, and the viscosity coefficient, respectively.

For the creep test, in general, the least square method is used to statistically process the test data [23]. The estimated creep parameters using the least square method are shown in Table 2. There is an approximately linear negative correlation between the elastic modulus E_1 and the saturation degree S (see Figure 7(a)):

$$E_1 = -0.5975S + 64.732. \quad (4)$$

The viscoelastic modulus E_2 and viscosity coefficient η show a negative exponent correlation with saturation degree S (see Figures 7(b)-7(c)):

$$\begin{aligned} E_2 &= 17028e^{-0.0584S}, \\ \eta &= 20285e^{-0.0533S}. \end{aligned} \quad (5)$$

It can be seen that all the Kelvin model parameters, E_1 , E_2 , and η , decrease with the saturation degree. Therefore, the damage to the rock induced by water can be considered as the damage of each creep parameter. The damage variable of the elastic modulus ($D_1(S)$) is defined as follows:

$$D_1(S) = \frac{E_1(0) - E_1(S)}{E_1(0)}, \quad (6)$$

where $E_1(S)$ is the elastic modulus of rock with saturation degree S and $E_1(0)$ is the elastic modulus of rock with saturation degree $S = 0$. Substituting (4) into (6), we have

$$D_1(S) = 0.00923S. \quad (7)$$

In the same way, the damage variables of the viscoelastic modulus ($D_2(S)$) and viscosity coefficient ($D_3(S)$) can be described as follows:

$$\begin{aligned} D_2(S) &= \frac{E_2(0) - E_2(S)}{E_2(0)} = 1 - e^{-0.0584S}, \\ D_3(S) &= \frac{\eta(0) - \eta(S)}{\eta(0)} = 1 - e^{-0.0533S}. \end{aligned} \quad (8)$$

Substituting (7)-(8) into (3), the Kelvin creep model considering the moisture degradation effect can be expressed as

$$\varepsilon(t, S) = \frac{\sigma}{E_1(0)[1 - D_1(S)]} + \frac{\sigma}{E_2(0)[1 - D_2(S)]} \left(1 - e^{-(E_2(0)[1 - D_2(S)]/\eta(0)[1 - D_3(S)]t}\right). \quad (9)$$

3.2. Validation of the Creep Model. To validate the Kelvin creep model considering the moisture degradation effect, a three-dimensional finite difference model is developed with FLAC^{3D} [24]. In classical fluid mechanics there are the following three hypotheses [25]: (1) the creep deformation of rock material results from the partial stress tensor, but the spherical stress tensor does not cause the creep deformation; that is, no volume flow occurs during the creep deformation; (2) rock is a kind of isotropic material, and the short-term stress-strain curve and creep curve in the tensile and compressive stress states are very similar; and (3) the Poisson's ratio of rock material is not dependent on time during the creep deformation. Based on the above hypotheses, we can derive the three-parameter generalised Kelvin model creep model in the three-dimensional stress state, which is described as follows:

$$\varepsilon_{ij} = \frac{\sigma_m \delta_{ij}}{3K} + \frac{S_{ij}}{2G_1} + \frac{S_{ij}}{2G_2} \left(1 - \exp\left(-\frac{G_2}{\eta}t\right)\right), \quad (10)$$

where ε_{ij} is the strain tensor, σ_m is the spherical stress tensor, δ_{ij} is the Kronecker delta, S_{ij} is the partial stress tensor, K is the bulk modulus, G_1 is the elastic shear modulus, and G_2

FIGURE 7: Parameters of generalized Kelvin model with the degrees of saturation: (a) E_1 - S ; (b) E_2 - S ; (c) η - S .

is the viscoelastic shear modulus. In the three-dimensional space, we can obtain the following equations:

$$\begin{aligned}
 \sigma_m &= \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3), \\
 S_{ij} &= \sigma_{ij} - \sigma_m, \\
 K &= \frac{E_1}{3(1-2\mu)}, \\
 G_1 &= \frac{E_1}{2(1+\mu)}, \\
 G_2 &= \frac{E_2}{2(1+\mu)}.
 \end{aligned} \tag{11}$$

Due to the limitations of the Kelvin model in dealing with the plastic characteristic of rock material, a Mohr-Coulomb model is then used. The viscoelastic and plastic strain rate components are assumed to act in series. The viscoelastic constitutive law corresponds to the Kelvin model, and the plastic constitutive law corresponds to the Mohr-Coulomb model. The creep model is developed by using a user subroutine, which is an option provided by FLAC^{3D}. The shape and size of the finite difference model are the same as for the tested specimens, and the mesh contains 1,000

elements and 1,111 nodes. The parameter values in the model are obtained from the laboratory test.

The calculated results for the specimens are shown in Figure 8. Although there is a certain deviation, the calculated curves coincide basically with the experimental curves with the same general rules, which demonstrates the validity of the creep model when considering the moisture degradation effect.

4. An Application of Three-Dimensional Analysis of the Ruipo Tunnel Portal

4.1. Project Background. The Shui-Du (Shuikou to Duyun city in Guizhou province, China) expressway is one of the most difficult sections in the construction of the Xiamen to Chengdu highway, which is the number 16 east-west expressway of the National Trunk Highway System. 58 tunnels were designed on the Shui-Du expressway, with a total length of 71,988 m (34.6% of the whole line) and more than 200 tunnel portals. Almost all of the rocks surrounding the tunnel portals are broken and have poor stability. About half of the portals are shallow buried and unsymmetrically loaded, such as the Laozhai tunnel portal [26] and the Ruipo tunnel portal (see Figure 9). Another challenge was the rainy season that set in during the excavation of tunnel portals. The heavy and constant rainfall would lead to a serious decrease in

FIGURE 8: Comparison between numerical data and laboratory test data of specimens with different saturation degrees: (a) $S = 41.5\%$; (b) $S = 80.1\%$; (c) $S = 93.5\%$.

FIGURE 9: Ruipo tunnel portal on the Shui-Du expressway.

surrounding rock quality. In order to overcome the difficulties in excavation, many prereinforcements (such as retaining wall and antislip pile) were built before excavation. Also,

many complicated excavation methods, for example, the cross diaphragm (CRD) method [27] and ring-like drift heading method [28], were adopted. The excavation was slow (less than 2 m/d), but even then collapse occurred in many tunnel portals.

The Ruipo tunnel is located in AT11 section of the Shui-Du expressway, with starting and ending milestones of ZK49+865 and ZK51+395. The surrounding rocks of the tunnel portal section (ZK49+865 and ZK50+000) were composed mainly of strong-weathered slate and completely weathered slate, which have very poor stability. The tunnel portal section was seriously shallow buried (the depth was 0–38 m) and unsymmetrically loaded (the transversal gradient was about 40°). According to design, the excavation in the tunnel portal section was to be performed by the CRD method. Before excavation the surrounding rocks were reinforced by a grouting method and a 108 mm diameter steel pipeline shed was installed along the tunnel outline, for which the intervals of steel were 50 cm. In order to mitigate the

FIGURE 10: The support system and excavation sequence: (a) support system at ZK49+899; (b) excavation sequence with CRD method (length unit: cm).

influence of an unsymmetrical load, seven antislidie piles were installed in the shallow buried side. The support system and excavation sequence are shown in Figure 10.

During excavation rain began to fall and lasted for nearly 3 days, and then the portal suffered severe damage. Cracks first appeared in the slope, initial support, and landfills behind the antislidie piles, and more cracks appeared and spread in the next few days (see Figure 11). For the reason that damage happened shortly after the tunnel portal excavation, only the forces of antislidie piles were measured. The monitoring results are shown in Figure 12 (many measurements were not picked up for the reason that the earth pressure cells were destroyed). As can be seen in Figure 12, the maximum value of the forces was only 55 kPa; the low forces showed that antislidie piles had less reinforcement effect. The reason for this is that the rainfalls reduced the loading capacity of the surrounding rocks and landfills in the shallow buried side, which could not transmit the unsymmetrical pressure from the deep buried side to the antislidie piles.

4.2. Elements and Boundary Conditions. In order to analyse the moisture degradation effect on creep characteristics of surrounding rocks, a three-dimensional finite difference analysis was performed by using the program $FLAC^{3D}$. The analysis model includes information from the horizontal direction, 5.5 times the tunnel span from the tunnel centre to the model boundary, and from the vertical direction, 3 times the tunnel height from the tunnel bottom to the base boundary, and from the height above the tunnel from the tunnel crown to the nature ground surface. The model dimensions (x , y , and z) were approximately 143 m, 71 m,

and 72 m. The boundary conditions include a free ground surface, fixed left and right boundaries in the x -axis direction, fixed front and rear boundaries in the z -axis direction, and a fixed base boundary in the y -axis direction. The tunnel liners, steel pipeline shed, horizontal support, and median septum are modelled with shell elements, and the antislidie piles are modelled with solid elements. There are 63,519 elements in this numerical model (see Figure 13).

4.3. Simulation Phases and Rock Mass Physical and Mechanical Properties. Before excavation, the system was brought to elastic equilibrium (time 0 of modelling) under gravity. Boundary and initial conditions were set. Referring to Figure 9, the excavation sequence can be defined for the CRD method as follows.

Stage 1. Part A was excavated once with a length of 2 m.

Stage 2. Parts A and B were excavated once with lengths of 2 m.

Stage 3. Parts A, B, and C were excavated once with lengths of 2 m.

Stage 4. Parts A, B, C, and D were excavated once with lengths of 2 m, until the tunnel excavation was completed.

Based on the actual situation of tunnel excavation and support implementation, no matter which part was excavated, the lining and anchors corresponding to that part should be activated before the creep time of 12 hours had

FIGURE 11: Damage in Ruipo tunnel portal: (a) cracks in slope at the beginning; (b) cracks expanded and instability of the slope; (c) cracks in landfills behind the antislade pile.

FIGURE 12: Measurement results of pressure between landfills and antislade piles: (a) 2 # antislade pile; (b) 4 # antislade pile; (c) 6 # antislade pile.

passed. And the next excavation stage was performed 24 hours after the support implementation.

For comparison, another condition, where the influences of rainfall and creep were not considered, was also simulated. The lining and anchors corresponding to the excavated part

should be activated simultaneously to simulate the support of the tunnel.

The constitutive models used in the numerical calculation are as follows: elastic model for all of the support system, including the tunnel liners, steel pipeline shed, horizontal

TABLE 3: Physical and mechanical properties of the rock mass and support.

Properties	Strong-weathered slate	Completely weathered slate	Initial lining	Second lining	Antislide pile	Landfills
Bulk volume $\gamma/(\text{kN}/\text{m}^3)$	23	18	22	25	25	22
Elastic modulus/MPa	$2.28E+03$	$9.62E+02$	$2.40E+04$	$3.25E+04$	$3.25E+04$	$2.10E+04$
Poisson's ratio	0.35	0.4	0.2	0.2	0.2	0.2
Cohesion c/MPa	0.25	0.10	—	—	—	—
Friction angle $\varphi/^\circ$	30	20	—	—	—	—
Elastic bulk modulus K/MPa	327.1	32.3	—	—	—	—
Elastic shear modulus G_1/MPa	158.5	15.2	—	—	—	—
Viscoelastic shear modulus G_2/MPa	6108.1	593.4	—	—	—	—
Viscosity coefficient $\eta/(\text{MPa}\cdot\text{h})$	9072.3	889.0	—	—	—	—

FIGURE 13: Three-dimensional analysis model.

support, median septum, and antislide piles; the Kelvin-MC model established in Section 4.1 for surrounding rocks; and the Mohr-Coulomb model for surrounding rocks under the condition that the influences of rainfall and creep were not considered.

The evaluation of the rock mass quality has been carried out through detailed geomechanical field surveys according to the ISRM suggested method [29]. The geological strength indexes of the strong-weathered slate and completely weathered slate are 25 and 10, respectively. The results obtained by field surveys and laboratory tests allowed us to apply the Hoek-Brown failure criterion and estimate the rock mass strength and elastic properties [30]. The decreases in the rock mass uniaxial compression strength were used to estimate the creep properties of rock mass [31]. The rock mass and support physical and mechanical properties are listed in Table 3.

During excavation, a heavy rainfall lasted for 3 days. Based on the relationship between saturation degree and time submerged in water (Section 2.2.1), the saturation degree of rock mass was estimated to be 84.0%.

4.4. Simulation Results. The excavation section ZK49+883, which was in the middle portion of the freshly constructed tunnel, was chosen as the subject for the research. Large surface subsidence happened after ZK49+883 was excavated (see Figure 14); the maximum value for the vertical displacement was -791 mm . The unsymmetrical surface subsidence squeezed the tunnel support structure, which made downward vertical displacements on the deep buried side and upward vertical displacements in the arch foot and haunch of shallow buried side. The maximum value for the horizontal displacement of surrounding rocks near the tunnel was 36 mm , which lies in the haunch of the deep buried side. The horizontal displacement of landfills behind the antislide piles was also very large. The simulation results show an acceptable agreement with the real situation.

The vertical and horizontal displacements at section ZK49+883, where the influences of rainfall and creep were not considered, are shown in Figure 15. The maximum values for the vertical and horizontal displacements were 7.2 mm and 2.1 mm , respectively. The low displacements indicate that the tunnel excavation is safe under this condition.

5. Conclusions

In this study, laboratory experiments were performed to understand the compression and creep characteristics of the low-grade metamorphic slate with different moisture states. Then a three-dimensional analysis on the Ruipo tunnel portal was performed while considering tunnel excavation and the moisture degradation effect on creep characteristics of surrounding rocks. The laboratory tests showed that the moisture has a great effect on the strength and deformability of the low-grade metamorphic slate, and the mathematical relation between the creep parameters and degree of saturation was established. The simulation results show that the tunnel excavation is safe without considering the influence of rainfall and creep, whereas with these being considered large surface subsidence and displacement would appear. Therefore, a three-dimensional analysis that considers the

FIGURE 14: Displacements at section ZK49+883 when the influences of rainfall and creep were considered: (a) vertical displacement; (b) horizontal displacement.

FIGURE 15: Displacements at section ZK49+883 when the influences of rainfall and creep were not considered: (a) vertical displacement; (b) horizontal displacement.

influence of moisture and creep of the surrounding rocks is needed for estimating the deformation of complex tunnel portals.

Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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