Numerical Study on Rock Stability and Mechanical Characteristics of Supporting Structure of Water Tunnel with Different Buried Depths

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Abstract. To explore the suitability of the supporting scheme of horseshoe tunnel affected by different buried depths in phyllite rock, the 3D numerical tunnelling simulations were conducted. The plastic zone distribution, deformation characteristics and stress field of surrounding rock and mechanical characteristics of supporting structures during the excavation process were analyzed. It was shown that the plastic zone depth, volume and deformation development of the surrounding rock are greatly limited by the shotcrete-bolt support under different buried depths. Meanwhile, the restriction proportion to the development of the plastic zone and rock deformation is slightly affected by the buried depth of the tunnel. With the increase of the buried depth of the tunnel, the reasonable shortest distance that should be reserved between the constructed lining and tunnel face also gradually increases. Combined with the calculated tunnel deformation and the classification standard proposed by Hoek, it is recommended to adopt full-section shotcrete-bolt support, and add steel arch support as the necessary supplement for the existing tunnel supporting scheme. The research results contribute to design and construction guidance for tunnels under similar geological environments.

1. Introduction
There are two main kinds of factors that affect the deformation and stability of surrounding rock during the underground engineering excavation. One is the geological factors, including the surrounding rock integrity and its physical and mechanical properties, the structural plane occurrence, groundwater, buried depth and lateral pressure coefficient, etc. The other is the engineering factors, including tunnel diameter, lining form, lining structural parameters and construction methods, etc. Among them, the buried depth is an important consideration basis for the design and construction of the tunnel supporting structure.

The tunnel depth is the main factor affecting the initial geostress of surrounding rock, besides the tectonism. Under the high buried depth, the surrounding rock is usually characterized by large deformation and poor stability after excavation, and rock pressures imposed on the lining are more complex, which may bring about engineering disasters like the supporting concrete cracking, steel arch distortion, bolt yield fracture and secondary lining failure. The buried depth has become an important index to evaluate the rockburst risk of hard rock or large deformation risk of soft rock under high ground stress. With the development of computer technology and numerical simulation theory, it is feasible to analyze the influence of buried depth and other factors on the surrounding rock stability and optimize the design of tunnel structure using numerical simulation method.
Tian et al. [1] studied the loose zone of the tunnel surrounding rock with the FEM, orthogonal test and field measurement methods, and the research shows that the main factor affecting the loose zone of surrounding rock is the buried depth, which was positively related to the loose zone size. Through the numerical simulation method, Wu et al. [2] compared the influences of tunnel span, buried depth, height span ratio, tunnel excavation method and mechanical excavation size on the surrounding rock stability during tunnel excavation in the upper soft and lower hard strata. The results show that the tunnel span and buried depth are the most critical factors that affect the stability of the tunnel. With five typical factors, including tunnel diameter, buried depth, strata inclination, thickness ratio of slate to sandstone and side pressure coefficient selected, Guo et al. [3] researched the influence rules of above different factors to surrounding rock stability through the orthogonal tests based on 3DEC numerical simulation software. Zhao [4] analyzed the influences of the buried depth, excavation radius and side pressure coefficient on the loose zone area with the numerical simulation method. In above researches, the application of the numerical method helps to obtain the influence rule of various factors on the deformation and stability of the tunnel surrounding rock.

Based on the previous survey and design data, the study carried out the suitability analysis of the excavation and supporting scheme of horseshoe tunnel in phyllite rock of class IV affected by different tunnel buried depths using the 3D numerical simulation method.

2. Project overview
A water tunnel with a total length of 23 km is designed to be excavated by drilling and blasting method. The terrain along the tunnel fluctuates greatly, the surface elevation varies from 2125 m to 3300 m, and the maximum buried depth of the tunnel is 1236 m. The bedrock along the tunnel in the north section of the project area is mainly phyllite of the lower devonian system, occasionally sandwiched with thin limestone or marble. The dissolution degree of limestone section is slight to medium. The bedrock along the south section is mainly sandstone of the upper triassic system. There are about 50 m thick slope proluvial or alluvial proluvial boulders, pebbles and debris accumulated near the tunnel entrance. The designed tunnel passes through the overburden.

There are metamorphic strata and magmatic rocks formed by magmatic activities in the project area. The strata are generally metamorphosed, with complex lithology and alternating soft and hard rocks. The proportion of surrounding rock in class II, III and IV is 4.67%, 51.54% and 43.79%, respectively. The geological exploration results show that the surrounding rock of phyllite (metamorphic rock) tunnel section is mainly in slightly weathered state, belonging to soft to medium hard rock. The tunnel axis intersects the schistosity trend with a small angle, which may leads rib spalling or rock falling. In addition, due to high buried depth, enough attention should be paid to the engineering geological problems caused by high ground stress during the tunnel excavation.

3. Numerical model and result analysis method

3.1. Fast Lagrangian analysis of continua method
In this study, FLAC3D software is used for numerical simulation. The method is based on an explicit finite difference method proposed by Cundall P.A., and has the following characteristics: 1) the continuous medium is divided into several interconnected solid elements, and the forces are concentrated on the nodes; 2) the first derivative of variables about space and time is approximated by finite difference; 3) the dynamic relaxation method is used to solve the particle motion equation, the motion of the system is attenuated to equilibrium by damping [5-6].

In the calculation, the formation of stiffness matrix is not needed. Both the dynamic and static problems can be solved by the explicit method of motion equation, which makes it easy to simulate nonlinear problems and dynamic problems such as vibration, instability and large deformation. Therefore, the software is suitable to deal with complex nonlinear geotechnical problems and the simulation of unloading effect of tunnel excavation in this study.
3.2. Tunnel excavation and supporting model

According to the geological profile along the tunnel, this study selects phyllite tunnel section to study the influence of different buried depths on surrounding rock deformation and supporting structure stress. The surrounding rock is in class IV, and the buried depth of the tunnel is 600 m, 900 m and 1200 m respectively. Figure 1 shows the designed section size of the horseshoe tunnel and the corresponding supporting measures. According to the geological survey report, the physical and mechanical parameters of rock mass are taken as the table 1. Meanwhile, according to the test and calculation results, the initial horizontal ground stresses under different buried depths are shown in Table 2.

![Figure 1. Horseshoe tunnel section and its supporting scheme (unit: cm).](image).

| Rock type | Deformation modulus (GPa) | Poisson's ratio | Friction coefficient | Cohesion (MPa) | Density (kg/m³) |
|-----------|--------------------------|----------------|---------------------|---------------|----------------|
| Phyllite  | 1.5                      | 0.30           | 0.60                | 2.5           | 2700           |

![Table 1. Mechanical parameters of rock mass.](image)

| Buried depth/ m | Initial geostress components/ MPa | σx | σy | τxy | σz |
|-----------------|----------------------------------|----|----|-----|----|
| 600             | 14.36                            | 19.64 | 3.14 | 15.88 |
| 900             | 21.58                            | 29.42 | 4.67 | 23.80 |
| 1200            | 28.72                            | 38.88 | 6.05 | 31.75 |

![Table 2. Initial geostress components under different buried depths.](image)

According to the section design scheme of horseshoe tunnel, the tunnel excavation model is established as shown in figure 2. The model dimensions are all 200 m in length, width and height with 154396 units and 158179 nodes in total. In the model, the direction perpendicular to the tunnel axis is set as the X direction, the direction along the tunnel axis is the Y direction, and the Z direction coincides with the vertical direction.

Figure 3 shows the initial supporting measures in which the bolt is simulated with the cable element, the sprayed concrete is simulated with the shell element, and the lining structure is simulated with the solid element. Mohr Coulomb constitutive model is adopted for surrounding rock element and elastic constitutive model for lining element.
3.3. Excavation process simulation

This study aims to analyze the influence of different buried depths on the stability of surrounding rock in the excavation process of the horseshoe tunnel with supporting measurements. The supporting scheme contains the initial shotcrete-bolt support and the permanent lining. Figure 4 shows the construction method of the initial shotcrete-bolt support and the permanent lining support with the continuous advance of the tunnel face. When applying the initial support, the bolt and shotcrete should be conducted in time after rock excavation, lagging behind the tunnel face by $S = 2$ m, so as to limit the surrounding rock deformation and the plastic zone development, and ensure the rock stability during the tunnel excavation period. The closer the construction site of permanent lining is to the tunnel face, the lower the release rate of surrounding rock deformation is, the larger the load bearing proportion of the lining due to rock excavation is, and the larger the reinforcement ratio is. In order to ensure that the lining stress is in a reasonable interval and control the reinforcement ratio, different lagging distances of the permanent lining are selected for comparative analysis, i.e. $t=12$ m, 16 m, 20 m, 24 m, 28 m and 32 m.

The method of continuous advance of the tunnel face is used to simulate the rock excavation process. Each excavation step is 2 m, that is, the rock mass within the current excavation area is removed firstly, and the initial support and permanent lining are respectively conducted at the corresponding location, and then the mechanical response of the surrounding rock and the supporting structure is calculated. The next excavation begins after the current calculation arrives balance state. The whole calculation continues until the model grid in tunnelling area is excavated. Meanwhile, the tunnel excavation simulation under the same buried depth condition without any supports, i.e. the unlined tunnel, is conducted to analyze the supporting effect.

The following provisions are made about the relative distance between the tunnel face and the monitoring section. When the tunnel face has not passed the monitoring section, their relative distance is defined as the negative value. After the tunnel face passes the monitoring section, their relative
distance is defined as the positive value. The stress sign in calculation results is consistent with that in the elastic mechanics.

Figure 4. Construction timing of primary support and secondary lining.

3.4. Results analysis method

To conduct comparative analysis, the mid points of the vault, side wall and floor of the tunnel contour in the middle section of the excavation model are selected as the monitoring points of surrounding rock deformation. The plastic zone, deformation, stress field of surrounding rock on the monitoring section, stress state of bolt and internal force of lining structure are analyzed. Meanwhile, the internal force calculation sections of lining are selected as figure 5 and lining reinforcements are calculated, so as to determine whether the lagging distance of lining construction is reasonable.

Figure 5. Internal force calculation section of lining.

4. Calculation results and analysis

To conduct the comparison of the surrounding rock deformation and the supporting structure stress under different buried depths, it is necessary to determine the corresponding lagging distance of the permanent lining, so the internal force of the lining and the reinforcement ratio can be ensured in the reasonable interval. The reasonable lagging distances of permanent lining corresponding to buried depths 600 m, 900 m and 1200 m of phyllite tunnel are 16 m, 24 m and 28 m respectively, through the comparing calculation and analysis.

4.1. Plastic zone of surrounding rock

Figure 6 shows the comparison of plastic zone distribution of surrounding rock between the unlined tunnel and the lined tunnel with the buried depth of 900 m. Tables 3 and 4 respectively show the comparison of the plastic zone depth and plastic zone volume around the tunnel under different buried depths.

It can be found that the plastic zone depth of surrounding rock of the lined tunnel is significantly lower than that corresponding to unlined tunnel condition. With the increase of the buried depth, the plastic zone depth and volume of the tunnel surrounding rock increase significantly. Meanwhile, it can be found that under different buried depths, supporting measures at different parts around the tunnel have different restrictions on the development of surrounding rock plastic zone. The plastic zone depths of surrounding rock of lined tunnel at the vault, side wall and floor are respectively 43%, 64% and 86% of those of the unlined tunnel. As for the restriction effect of volume development of rock plastic zone
induced by supporting measurements, the volumes of shear plastic zone, tensile plastic zone and whole plastic zone are respectively 83%, 74% and 82% of those corresponding to the unlined tunnel. Above proportions have small differences with different buried depths. It is shown that under the same supporting scheme, the restriction proportion of the plastic zone development of surrounding rock is slightly affected by the tunnel depth.

![Figure 6. Distribution comparison of surrounding rock plastic area (buried depth of 900 m of class IV phyllite).](image)

| Buried depth | Condition | vault | side wall | floor |
|-------------|-----------|-------|----------|-------|
|              | plastic zone depth (m) | percentage | plastic zone depth (m) | percentage | plastic zone depth (m) | percentage |
| 600 m        | unlined   | 3.9   | 100%     | 4.2   | 100% | 4.7 | 100% |
|              | lined     | 1.8   | 46.2%    | 2.7   | 64.3% | 4.2 | 89.4% |
| 900 m        | unlined   | 4.7   | 100%     | 5.2   | 100% | 5.7 | 100% |
|              | lined     | 2.1   | 44.7%    | 3.3   | 63.5% | 4.7 | 82.5% |
| 1200 m       | unlined   | 6.1   | 100%     | 6.5   | 100% | 6.5 | 100% |
|              | lined     | 2.4   | 39.3%    | 4.2   | 64.6% | 5.6 | 86.2% |

Table 3. Comparison of the plastic zone depths of surrounding rock around the tunnel under different buried depths.

| Buried depth | Condition | plastic zone volume ($10^4$ m$^3$) | percentage | plastic zone volume ($10^4$ m$^3$) | percentage |
|-------------|-----------|-------------------------------------|------------|-------------------------------------|------------|
| 600 m       | unlined   | 3.84                               | 100%       | 0.59                                | 100%       |
|             | lined     | 3.12                               | 81.3%      | 0.44                                | 74.6%      |
| 900 m       | unlined   | 4.95                               | 100%       | 0.69                                | 100%       |
|             | lined     | 4.18                               | 84.4%      | 0.51                                | 73.9%      |
| 1200 m      | unlined   | 7.50                               | 100%       | 0.81                                | 100%       |
|             | lined     | 6.14                               | 81.8%      | 0.60                                | 74.1%      |

Table 4. Volume comparison of rock plastic zone around the tunnel under different buried depths.

4.2. Deformation of surrounding rock

Figure 7 shows the comparison of surrounding rock deformation distribution between the unlined tunnel and the lined tunnel with the buried depth of 900m. Tables 5 shows the comparison of surrounding rock deformations around the tunnel under different buried depths.

It can be found that the surrounding rock deformation of the lined tunnel is significantly lower than that corresponding to unlined tunnel condition. It means that the supporting measures can effectively restrict the deformation development of surrounding rock and reduce the possibility of rock instability.
With the increase of the buried depth, the surrounding rock deformation around the tunnel increase significantly. Under different buried depths, the reduction ratio of surrounding rock deformation before and after supporting conduction at the same part around the tunnel is similar. The plastic zone depths of surrounding rock of lined tunnel at the vault, side wall and floor are respectively 46%, 61% and 82% of those of the unlined tunnel. The above proportions have small differences with different buried depth. It is shown that under the same supporting scheme, the restriction proportion of the deformation development of surrounding rock is slightly affected by the tunnel depth.

![Figure 7. Deformation comparison of surrounding rock (buried depth of 900 m of class IV phyllite).](image)

| Buried depth | Condition | vault deformation (mm) | percentage | side wall deformation (mm) | percentage | floor deformation (mm) | percentage |
|-------------|-----------|------------------------|------------|---------------------------|------------|------------------------|------------|
| 600 m       | unlined   | 169.3                  | 100%       | 177.4                     | 100%       | 185.3                  | 100%       |
|             | lined     | 79.72                  | 47.1%      | 105.6                     | 59.5%      | 147.5                  | 79.6%      |
| 900 m       | unlined   | 282.9                  | 100%       | 310.9                     | 100%       | 322.5                  | 100%       |
|             | lined     | 135.9                  | 48.0%      | 194.6                     | 62.6%      | 275.4                  | 85.4%      |
| 1200 m      | unlined   | 493.5                  | 100%       | 543.7                     | 100%       | 550.2                  | 100%       |
|             | lined     | 214.7                  | 43.5%      | 333.1                     | 61.3%      | 452.7                  | 82.3%      |

4.3. Stress of surrounding rock

Figures 8 and 9 show the comparisons of the first and the third principal stress of the surrounding rock between the unlined tunnel and the lined tunnel under the buried depth of 900 m, respectively. Table 6 shows the deformation comparison of the surrounding rock around the tunnel under different depths.

It can be found that under the buried depth of 900 m, the first principal stress of the surrounding rock increases significantly by 0.6−8.6 MPa, and the third principal stress changes from the small compressive stress to the significant compressive stress. The increase of compressive stress at the vault is the most significant and effectively restricts the unloading effect of tunnel excavation. It is shown that the mechanism of supporting measures to improve the stability of surrounding rock is to reduce the stress release degree and unloading area of surrounding rock and further improve the bearing capacity of surrounding rock.

With the increase of the tunnel depth, the growth degrees of main stress of the surrounding rock in different parts of the tunnel are quite different between the unlined tunnel and the lined tunnel. For the buried depths of 600 m, 900 m and 1200 m, compared with the unlined tunnel, the first principal stress at the vault of the lined tunnel increases by 5.1 MPa, 8.6 MPa and 11.6 MPa, respectively. That at the side wall increases by about 1.1 MPa, and that at the floor merely increases by about 0.7 MPa. It is the result of the intensive bolts at the vault in the supporting scheme, and the deformation gradient is
significant there. Therefore, the bolts restrict the release of the surrounding rock stress well. Due to no bolts at the floor, the deformation and stress release of surrounding rock there are barely affected. For the tunnel with high depth, the rock deformation at the floor is obvious, which may result in the failure of the floor heave. The inverted arch should be conducted in time, and the steel arch can be added if necessary.

Shotcrete-bolt support is only conducted at the side wall and vault under the phyllite surrounding rock in class IV. Due to the instability possibility induced by the local large deformation of surrounding rock, it is recommended to adopt full-section shotcrete-bolt support for the tunnel section with large buried depth, so as to avoid the excessive plastic zone and floor deformation of the tunnel and to ensure the surrounding rock stability of the whole section.

![Figure 8](image1.png)  
(a) unlined tunnel  
(b) lined tunnel  
Figure 8. Comparison of the first principal stress of surrounding rock (buried depth of 900 m of class IV phyllite).

![Figure 9](image2.png)  
(a) unlined tunnel  
(b) lined tunnel  
Figure 9. Comparison of the third principal stress of surrounding rock (buried depth of 900 m of class IV phyllite).

Table 6. Comparison of main stress of surrounding rock around the tunnel under different buried depths.

| Buried depth | Condition | vault (MPa) | side wall (MPa) | floor (MPa) |
|--------------|-----------|-------------|----------------|-------------|
|              |           | σ₁  | σ₃  | σ₁  | σ₃  | σ₁  | σ₃  |
| 600 m        | unlined   | -7.17 | -0.12 | -7.09 | -0.06 | -7.08 | -0.07 |
|              | lined     | -12.26 | -4.88 | -8.19 | -1.38 | -7.73 | -0.66 |
|              | unlined   | -9.11 | -0.20 | -8.86 | -0.07 | -8.85 | -0.08 |
| 900 m        | lined     | -17.72 | -6.92 | -9.91 | -0.83 | -9.45 | -0.54 |
|              | unlined   | -8.9 | -0.13 | -8.9 | -0.08 | -8.8 | -0.07 |
| 1200 m       | lined     | -20.5 | -8.05 | -10.0 | -0.88 | -9.61 | -0.64 |
4.4. Bolt stress

Figure 10 shows bolt stresses in lined tunnel under the buried depth of 900 m, and table 7 shows the comparison of extreme values of bolt stress in lined tunnel under different buried depths.

It can be seen that with the increase of the tunnel buried depth, the minimum stress of bolts increases continuously, while the maximum stress of the bolt reaches or approaches 300 MPa, which is close to the yielded state, and its safety margin is small. The yielding failure of bolt may lead to the overall instability of the surrounding rock from point to surface and underground engineering accidents. Adopting hollow grouting bolt to improve the mechanical properties of surrounding rock and enhance the bolting force of bolt, or using yielding bolt to enhance the allowable deformation of rod body are possible ways to solve the problem of insufficient bolting capacity of bolts [7-8].

Figure 10. Bolt stresses in lined tunnel (MPa, buried depth of 900 m of class IV phyllite).

Table 7. Comparison of extreme values of bolt stress in lined tunnel under different buried depths.

| Buried depth | Minimum bolt stress (MPa) | Maximum bolt stress (MPa) |
|--------------|--------------------------|--------------------------|
| 600 m        | 136                      | 299                      |
| 900 m        | 241                      | 303                      |
| 1200 m       | 259                      | 305                      |

4.5. Internal force and reinforcement of lining

Considering the spatial effect of tunnel excavation, the closer the construction site of permanent lining is to the tunnel face, the lower the release rate of surrounding rock deformation is, the larger the load bearing proportion of the lining due to rock excavation is, and the larger the reinforcement ratio of the lining is. Therefore, sufficient distance should be reserved between the construction site of the permanent lining and the tunnel face to control the stress and reinforcement of the lining.

For the phyllite tunnel section with the buried depth of 900 m, when the lining construction lags behind the tunnel face by 20 m, the reinforcement ratios of D-D and F-F sections are large. When the lining construction lags behind the tunnel face by 24 m, the surrounding rock deformation and reinforcement ratio of the lining reduce greatly. Therefore, the reasonable shortest lagging distance of lining corresponding to the buried depth of 900 m is 24 m. In the same way, those corresponding to the buried depths of 600 m and 1200 m can be determined to be 16 m and 28 m, respectively. Table 8 shows the relationship comparison between the internal force and reinforcement ratio of horseshoe tunnel lining and its displacement release rate under the reasonable shortest lagging distance of lining corresponding to different buried depths.

It should be pointed out that this study mainly reflect the spatial effect of the tunnel excavation on the supporting structure, while the influence of the time-dependent deformation of surrounding rock on the stress of permanent lining structure has not been considered. In fact, the surrounding rock deformation induced by tunnel excavation in phyllite may show some time-dependent characteristics, and the surrounding rock deformation can converge slowly. Therefore, the difference of construction conditions should be considered in the actual project, and the lagging distance of lining construction
should be adjusted dynamically based on the convergence deformation of surrounding rock and the monitoring results of supporting structure stress during tunnel excavation.

Table 8. Comparison of internal force and reinforcement ratio of horseshoe tunnel lining and their relationship with displacement release rate.

| Buried depth | 600 m | 900 m | 1200 m |
|--------------|-------|-------|--------|
|               | Lining lagging 16 m behind | Lining lagging 24 m behind | Lining lagging 28 m behind |
| Sections      | M/ kN·m | N/ kN | ρ /% | M/ kN·m | N/ kN | ρ /% | M/ kN·m | N/ kN | ρ /% |
| A-A          | -19 | 2369 | constructional | -40.0 | 1825 | constructional | -88.4 | 2052 | constructional |
| B-B          | 78.3 | 3566 | constructional | 64.6 | 2606 | constructional | 44.4 | 3893 | constructional |
| C-C          | -79.3 | 4204 | constructional | -30.6 | 3739 | constructional | -7.2 | 4463 | constructional |
| D-D          | 355 | 5952 | 1.43 | 258.7 | 4727 | constructional | 270.0 | 6169 | 1.27 |
| E-E          | -250 | 4417 | constructional | -208.8 | 3644 | constructional | -258.8 | 4386 | constructional |
| F-F          | 290 | 6990 | 2.11 | 221.3 | 5546 | 0.49 | 253.9 | 7261 | 2.21 |
| G-G          | -72.2 | 4039 | constructional | 29.0 | 3793 | constructional | -1.0 | 5074 | constructional |
| H-H          | 95.6 | 3203 | constructional | 70.7 | 3063 | constructional | 54.2 | 3934 | constructional |
| Deformation release rate | 91.4%~92.8% | 95.1%~96.0% | 97.8%~98.2% |

5. Classification of surrounding rock deformation

Hoek and Marinos [9] proposed the tunnel deformation classification standard based on the relationship between the relative strain of the tunnel (the ratio of the tunnel closure to tunnel diameter) and rock strength-stress ratio (the ratio of rock mass strength to the in-situ stress), as shown in Figure 11. Based on numerous field practices, the recommended supporting schemes corresponding to different tunnel deformation degrees are given as shown in Table 9.

Figure 11. Tunnel deformation classification standard based on the relationship between the relative strain of the tunnel and rock strength-stress ratio [9].
Table 9. Recommended supporting schemes corresponding to different tunnel deformation degrees [9].

| Grade | Squeezing classification | Strain ε% | Supporting types |
|-------|--------------------------|-----------|------------------|
| A     | Few support problems     | Less than 1 | Very simple tunnelling conditions, with rockbolts and shotcrete typically used for support. |
| B     | Minor squeezing problems | 1 to 2.5  | Minor squeezing problems which are generally dealt with by rockbolts and shotcrete; sometimes with light steel sets or lattice girders are added for additional security. |
| C     | Severe squeezing problems | 2.5 to 5  | Severe squeezing problems requiring rapid installation of support and careful control of construction quality. Heavy steel sets embedded in shotcrete are generally required. |
| D     | Very severe squeezing problems | 5 to 10 | Very severe squeezing and face stability problems. Forepoling and face reinforcement with steel sets embedded in shotcrete are usually necessary. |
| E     | Extreme squeezing problems | More than 10 | Extreme squeezing problems. Forepoling and face reinforcement are usually applied and yielding support may be required in extreme cases. |

According to the calculation results of rock deformation around the tunnel under different buried depths in this study, the classification of surrounding rock deformation degree is obtained as shown in table 10. For the surrounding rock with large buried depth in this study, considering its large surrounding rock deformation, steel arch support should be properly used as the necessary supplement of initial shotcrete-bolt support according to the recommendation of Hoek. The necessity and feasibility of advance support should also be studied based on the time-space characteristics of surrounding rock deformation and the application of conventional shotcrete-bolt support and steel arch support. Meanwhile, considering the decrease of surrounding rock grade and the increase of buried depth, the value of surrounding rock deformation increases significantly. It is suggested to reserve some surrounding rock deformation allowances according to its deformation degree, and prevent too large rock deformation from occupying the tunnel clearance area.

Table 10. Statistics of deformation value of surrounding rock of tunnel under various working conditions and classification of surrounding rock extrusion deformation (class IV surrounding rock).

| Buried depth | Lining lagging distance/m | Deformation value (mm) | Strain ε% | Squeezing classification |
|--------------|--------------------------|------------------------|-----------|-------------------------|
| 600 m        | 16                       | 147.5                  | 2.7%      | Severe                  |
| 900 m        | 24                       | 275.4                  | 5.1%      | Very severe             |
| 1200 m       | 28                       | 452.7                  | 8.4%      | Very severe             |

6. Conclusions

To explore the suitability of the supporting scheme of horseshoe tunnel affected by different buried depths in phyllite rock, the 3D numerical tunnelling simulations were conducted. The plastic zone distribution, deformation characteristics and stress field of surrounding rock and mechanical characteristics of supporting structures during the excavation process were analyzed. The main conclusions are as follows:

(1) The surrounding rock stability and deformation of the unlined and lined tunnels under different buried depths are analyzed and compared. It was shown that the plastic zone depth, volume and deformation development of the surrounding rock are greatly limited by the shotcrete-bolt support under different buried depths. Meanwhile, the restriction proportion to the development of the plastic zone and rock deformation is slightly affected by the buried depth of the tunnel.

(2) The increase of compressive stress at the vault is the most significant and effectively restricts the unloading effect of tunnel excavation. It is shown that the mechanism of supporting measures to improve
the stability of surrounding rock is to reduce the stress release degree and unloading area of surrounding rock and further improve the bearing capacity of surrounding rock.

(3) Sufficient distance should be reserved between the construction site of the permanent lining and the tunnel face to control the stress and reinforcement of the lining. With the increase of the buried depth of the tunnel, the reasonable shortest distance also gradually increases.

(4) With the increase of the tunnel buried depth, the minimum stress of bolts increases continuously, while the maximum stress of the bolt reaches the yielded state and its safety margin is small. For the surrounding rock with large buried depth in this study, considering its large surrounding rock deformation, steel arch support should be properly used as the necessary supplement of initial shotcrete-bolt support.

Acknowledgements
This work was funded by China Postdoctoral Science Foundation [grant numbers 2019M662568] and CRSRI Open Research Program [grant numbers CKWV2019741/KY].

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