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Research Article

Analysis of Surrounding Rock Pressure of Deep Buried Tunnel considering the Influence of Seepage

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This paper takes the surrounding rock of deep tunnel as the research object and considers the action mechanism under the influence of seepage. Based on the Mohr-Coulomb criterion, the stress mechanism of surrounding rock of deep buried tunnel is analyzed by a convergence constraint method. Based on the elastic-plastic solution, the nonlinear elastic-plastic solution of the interaction between surrounding rock and lining structure considering the effect of seepage force is proposed, and the radius of surrounding rock plastic zone is obtained. The relationship between surrounding rock stress and displacement, radial deformation of lining, and support reaction force was observed. At the same time, considering the effects of seepage, strain softening, and intermediate principal stress, the surrounding rock is divided into a plastic residual zone, plastic softening zone, and elastic zone, and the stress distribution expressions of the plastic zone and each zone of surrounding rock of circular tunnel are derived. The results show that with the change of nonuniform permeability coefficient, the seepage shows anisotropy in different directions, and the closer to the horizontal or vertical direction, the more obvious the influence of nonuniform permeability coefficient on pore water pressure distribution. Seepage and material softening have different effects on the distribution of surrounding rock stress field and the size of plastic zone. Material softening is more unfavorable to the stability of surrounding rock than seepage. The intermediate principal stress coefficient has a significant impact on the tangential stress and plastic zone of surrounding rock. When the intermediate principal stress effect is not considered, the calculation results are relatively conservative and cannot give full play to the strength of surrounding rock effectively. The research conclusion can provide a theoretical reference for studying the stability of surrounding rock in tunnel excavation under water-bearing rock.

1. Introduction

With the rapid development of economy, domestic infrastructure construction is in full swing, and all kinds of transportation are gradually expanding to remote mountain areas. As one of the key points of traffic construction, tunnel can give full play to the characteristics of high rock compressive strength when crossing mountainous areas, so it has been widely used [1–4]. However, the change of stress and displacement of tunnel surrounding rock caused by the coupling of seepage field and stress field after tunnel excavation and the interaction between surrounding rock and lining are unavoidable problems in current tunnel construction. During tunnel excavation under water-rich conditions [5–7], the tunnel is mainly affected by water in two aspects. One is that the existence of water reduces the mechanical parameters of rock mass and changes the strength of rock mass. On the other hand, the tunnel excavation leads to the redistribution of surrounding rock seepage field and changes the pore water pressure. The coupling between the two fields of the stress field and seepage field is the key reason for aggravating formation deformation. Therefore, considering the influence of seepage, the research on the stress and stress distribution of surrounding rock of deep tunnel has become a hot spot in academic and engineering circles.

On the stress and displacement of tunnel surrounding rock, some scholars mainly use elastic-plastic constitutive to analyze such problems. The main research methods include theoretical analysis method [6–15] and numerical simulation method [16–20]. In terms of theoretical analytical method, Liu et al. [6] substituted the Hoek-Brown criterion into the equilibrium differential equation considering the influence of seepage volume force to solve it. By solving the transcendental equation, the numerical solution of stress around the tunnel in the plastic zone and the numerical solution of rock mass stress in the elastic zone are obtained; Liu et al. [7] applied the seepage force to the stress field in the form of volume force without considering the lining and obtained the analytical expressions of elastic displacement and stress and then applied the Mohr-Coulomb yield criterion to obtain the analytical expressions of plastic stress and plastic radius; Lee and Pietyuszczak [8] used a simplified numerical method to calculate the stress and displacement distribution of circular caverns in Mohr-Coulomb and Hoek-Brown surrounding rock media considering strain softening. In terms of numerical simulation, Dou et al. [16] set the surrounding rock and lining as elastic-plastic materials and gave the calculation method of safety factor of tunnel surrounding rock, primary support, and secondary lining by using the method of limit analysis; Wu et al. [17] used the analysis program based on the rock elastic-plastic stress seepage damage coupling model, used the coupling model to inverse the damage parameters according to the field monitoring displacement, and then analyzed the distribution law of the tunnel surrounding rock stress field, seepage field, damage field, and stress characteristics of lining structure. On the other hand, there are many achievements in the research on the interaction between displacement and lining [21]. Based on Drucker-Prager yield criterion, Wang et al. [21] deduced the elastic-plastic analytical solution of the interaction between surrounding rock and lining structure under the influence of seepage effect and expounded the application of the above analytical results in determining the load of lining structure; Wu et al. [22] used the Mohr-Coulomb yield criterion and bilinear constitutive model to put forward the analytical expression of elastic-plastic solution of interaction system between surrounding rock and lining in deep buried circular roadway under generalized load; Zhu et al. [23] analyzed and studied the bearing water pressure of railway tunnel composite lining by the finite element method with the help of a load structure model.

In recent years, many scholars have studied on the basis of the classical elastic-plastic solution of the tunnel and deduced the tunnel elastic-plastic solution considering the influence of the seepage field. In Reference [7], by simplifying the deep buried circular tunnel and solving it by using the Mohr-Coulomb strength criterion, the analytical solution of surrounding rock stress considering seepage field is obtained, and the influence of seepage field on surrounding rock stress is discussed. In Reference [24], based on the study of reference [7], the stress adjustment coefficient is introduced to obtain the elastic-plastic analytical solution of deep buried circular permeable tunnel considering the influence of in situ stress redistribution. In Reference [24], the theoretical solution of the stability of roadway surrounding rock is derived by using the theory of elastic-plastic damage mechanics. By summarizing four rock strength criteria including the Mohr-Coulomb criterion and Hoek-Brown criterion, literature [25] obtains the unified form of the surrounding rock yield equation under plane strain. On this basis, the unified solutions of surrounding rock stress field and displacement field under seepage are derived. Reference [6] improved the elastoplastic stress solution of surrounding rock derived based on Hoek-Brown criterion so that it does not contain integral term. It can be seen that most scholars choose the Mohr-Coulomb criterion [24] and Hoek-Brown criterion [26] without considering the intermediate principal stress in the elastic-plastic calculation and analysis of surrounding rock, resulting in conservative calculation results. In the past, isotropic calculation was mostly used in the calculation of the seepage field; that is, the permeability coefficients in all directions were equal. This is different from the engineering practice. Only by mastering the distribution law of surrounding rock seepage field after tunnel excavation can we analyze the influence of seepage on surrounding rock stress and plastic zone.

On the basis of previous studies, based on the unified strength theory criterion and considering the effects of strain softening, seepage, and intermediate principal stress, this paper divides the tunnel surrounding rock considering the influence of seepage into plastic residual zone, plastic softening zone, and elastic zone, deduces the analytical expressions of stress and half diameter of each zone of surrounding rock, and considers the differences of permeability coefficients in horizontal and vertical directions. By defining the nonuniform permeability coefficient, its influence on the distribution law of pore water pressure in all directions of surrounding rock is analyzed. Finally, an example is given to analyze the effects of surrounding rock softening, seepage, and intermediate principal stress coefficient on the tangential stress and plastic zone radius of the tunnel. The research results can provide reference for further study on the stability of surrounding rock in tunnel excavation under water-bearing rock.

2. Analysis of Surrounding Rock Pressure and Displacement considering Seepage

2.1. Basic Assumptions and Mechanical Models. After the tunnel is excavated in water-rich mountain area, the surrounding rock and lining structure near the tunnel will be affected by seepage force. According to the basic theory of seepage mechanics, the seepage water pressure after tunnel excavation and seepage stability is calculated as the boundary condition for the analysis of the interaction between surrounding rock and lining structure under the influence of equivalent seepage effect. In order to facilitate the subsequent calculation, the following assumptions and simplifications are made of the characteristics of seepage in high head mountain tunnel and the theory of plastic mechanics:

(1) The tunnel is deeply buried and the excavation surface is circular

(2) The surrounding rock is isotropic, homogeneous, and continuous ideal elastic-plastic medium

(3) Lateral pressure coefficient of rock around tunnel is 1, and the tunnel bears equal pressure in all directions

(4) Groundwater is incompressible, and obey the seepage law under a steady state

(5) Because of the axial length of the tunnel, the value of the transverse dimension is very large, ignoring the influence of both ends of the tunnel; the tunnel is simplified as a plane strain problem

Based on the above assumptions, the analysis model of tunnel surrounding rock and lining seepage field is determined as shown in Figure 1 (the circular area from inside to outside in the figure is lining and surrounding rock, respectively; the circle with a distance of R from the center of the tunnel is the far-field seepage boundary, and its stable seepage head is H (corresponding water pressure is p_{w0})). The inner diameter of tunnel lining is r_0, and the outer diameter is r_1. In addition, the permeability coefficients of surrounding rock and lining are k_r and k_l, respectively.

2.2. Analysis of Steady Seepage Field. The seepage flow Q at each position of surrounding rock and lining has the following relationship with seepage water pressure:

$$Q = \frac{2k\pi r dp_w}{\gamma_w dr}, \tag{1}$$

where k is the permeability coefficient of surrounding rock or lining at the calculated section; r is the radius of any surrounding rock or lining circular section with the tunnel center as the center; γ_w is the gravity of water; and p_w is the seepage water pressure.

The boundary conditions of equation (1) include the following: on the inner and outer diameter of tunnel lining, when $r = r_0$, the seepage water pressure $p_w = 0$; when $r = r_1$, seepage water pressure $p_w = p_{w1}$. At the far-field seepage stability radius $(r = R)$, the seepage water pressure $p_w = p_{w0}$. The continuity condition is as follows: assuming that the seepage flow at a radius section of surrounding rock is Q_r and the seepage flow at a radius section of lining is Q_1, considering that the tunnel lining and surrounding rock are in a stable seepage field, the seepage flow at the junction of surrounding rock and lining is continuous; that is, when $r = r_1$, $Q_r = Q_1$.

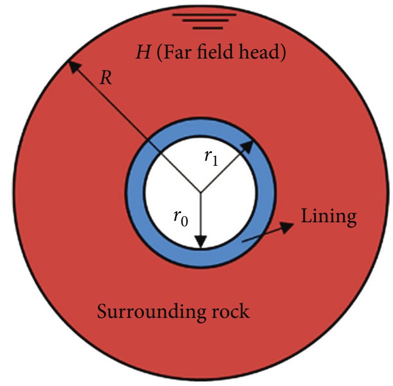

Figure 1: Tunnel seepage model.

By solving equation (1) with boundary conditions and continuity conditions, the expression of seepage water pressure p_w at any radius of lining and surrounding rock can be obtained:

$$P_w \begin{cases} P_{w0} = \dfrac{kr \ln(r/r_0)}{k1 \ln(R/r_1) + kr \ln(r_1/r0)}, & r_0 \leqslant r \leqslant r_{,1} \\ P_{w0} = \dfrac{kr \ln(r_1/r_0) + k1 \ln(r/r_1)}{k1 \ln(R/r_1) + kr \ln(r_1/r0)}, & r_1 \leqslant r \leqslant R. \end{cases} \tag{2}$$

Equation (2) is the seepage water pressure state after the tunnel seepage is in a stable state.

2.3. Theoretical Analysis of Stress and Displacement. The initial equilibrium of surrounding rock is disturbed by tunnel excavation. Due to the change of seepage state and in situ stress, the stress state around the tunnel will also change. Some surrounding rock will undergo nonlinear plastic deformation, not just elastic deformation. Displacement and stress of rock around tunnel such deep buried tunnel are suitable to be analyzed by elastic-plastic constitutive law.

2.3.1. Elastoplastic Mechanical Analysis Model. Based on the assumption, the elastic-plastic analysis model of tunnel surrounding rock is made, as shown in Figure 2. The inner diameter and outer diameter of tunnel lining are r_0 and r_1, respectively. The seepage water pressure at the far-field stable seepage is p_{w0}, the support reaction force of lining on surrounding rock after lining is applied is p_1, and the surrounding rock pressure uniformly distributed around the tunnel is q. Under the combined action of surrounding rock pressure, seepage water pressure, and support reaction force, the surrounding rock adjacent to the tunnel produces plastic deformation. The outer radius of the plastic zone is R_p, and the surrounding rock outside the plastic zone is still elastic. Take the rock mass element at any point a on the

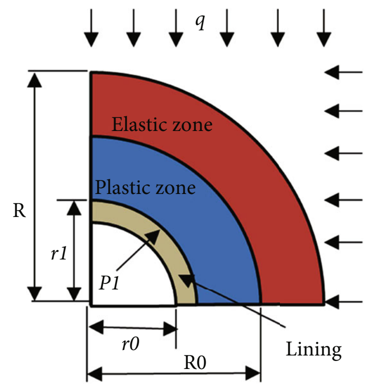

FIGURE 2: Elastoplastic analysis model.

surrounding rock in Figure 2 for stress analysis. The stress balance equation of point A is

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} - \beta \frac{dp_w}{dr} = 0, \quad (3)$$

where r is the distance from point A to the center of the tunnel; p_w is the seepage water pressure; and β refers to the reduction coefficient of seepage water pressure caused by fluid stress coupling during construction, and its value is related to the porosity of the material. For concrete, the value is generally 2/31, and for rock close to failure, the value is close to 1. If there is no special description in this paper, the value is 5/6 according to the actual project; σ_r is the circumferential effective stress, and σ_θ takes the tensile stress as positive and the compressive stress as negative. The stresses not specified below refer to the effective stress.

2.3.2. Stress Analysis of Surrounding Rock

(1) Analysis of the plastic zone of surrounding rock

Under the joint action of surrounding rock pressure, seepage water pressure, and support reaction force, part of the surrounding rock enters the plastic state, which is in the annular area between the lining and the radius R_p. After yielding, the rock mass in this area meets the Mohr-Coulomb criterion, that is:

$$\sigma_\theta^p = \frac{1 + \sin\varphi}{1 - \sin\varphi}\sigma_r^p + \frac{2c\cos\varphi}{1 - \sin\varphi}, \quad (4)$$

where σ_θ^p and σ_r^p are the radial effective stress and circumferential effective stress of the plastic zone element, respectively; c is the cohesion of rock mass; and φ is the friction angle in the rock body.

Substituting equations (2) and (4) into the equilibrium equation (3), the first-order nonlinear differential equation is obtained and solved with reference to [8].

$$\sigma_\theta^p = \frac{1 + \sin\varphi}{1 - \sin\varphi}(B - p_1)\left(\frac{r}{r_1}\right)^{2\sin\varphi/(1-\sin\varphi)} - \frac{B(1 + \sin\varphi)}{1 - \sin\varphi} + \frac{2\cos\varphi}{1 - \sin\varphi}. \quad (5)$$

(2) Elastic zone analysis of surrounding rock

It is assumed that the radius at the junction of the plastic zone and elastic zone of surrounding rock is R_p, and the radial stress of rock mass at this location is σ_{RP}. The elastic zone of surrounding rock of deep buried tunnel is regarded as a thick walled cylinder, and the earth pressure q and internal stress in the elastic zone can be expressed by Kirsch formula stress state of the elastic zone under σ_{RP}:

$$\sigma_r^E = \frac{R_p^2}{r^2}\sigma_{RP} + \frac{-q}{2}\left[(1+\lambda)\left(1 - \frac{R_p^2}{r^2}\right) + (1-\lambda)\left(1 - \frac{4R_p^2}{r^2} + \frac{3R_p^4}{r^4}\right)\cos 2\theta\right], \quad (6)$$

$$\sigma_\theta^E = -\frac{R_p^2}{r^2}\sigma_{RP} + \frac{-q}{2}\left[(1+\lambda)\left(1 + \frac{R_p^2}{r^2}\right) + (1-\lambda)\left(1 + \frac{3R_p^4}{r^4}\right)\cos 2\theta\right]. \quad (7)$$

In formulas (13)–(14), σ_r^E and σ_θ^E are the total radial stress and total circumferential stress of surrounding rock in the elastic zone, respectively; q is the surrounding rock pressure; λ is the lateral pressure coefficient; and θ is the angle between the calculated point and the tunnel center line and the vertical direction.

In this paper, the stresses under various equal pressure conditions are considered, so the lateral pressure coefficient λ takes 1. At the same time, considering the influence of seepage water pressure, it can be simplified as follows:

$$\sigma_r^E = -q\left(1 - \frac{R_p^2}{r^2}\right) + \frac{R_p^2}{r^2}\sigma_{RP} + \beta p_w, \quad (8)$$

$$\sigma_\theta^E = -q\left(1 + \frac{R_p^2}{r^2}\right) - \frac{R_p^2}{r^2}\sigma_{RP} + \beta p_w. \quad (9)$$

(3) Analysis of the plastic radius of surrounding rock

According to the stress continuity condition of surrounding rock at the elastic-plastic boundary ($r = R_p$), on the elastic-plastic boundary of surrounding rock, there are

$$\sigma_r^P + \sigma_\theta^P = \sigma_r^E + \sigma_\theta^E. \tag{10}$$

According to the calculation and simplification in Reference [14], the following can be obtained:

$$p_1 = B + \left(\frac{r_1}{R_p}\right)^{2\sin\varphi/(1-\sin\varphi)}(1-\sin\varphi) \cdot \left(q + \frac{c\cos\varphi}{1-\sin\varphi} + \frac{B}{1-\sin\varphi}\right), \tag{11}$$

$$R_p = \frac{r_1}{[(p_1 - B)/(q(1-\sin\varphi) + c\cos\varphi + B)]^{(91-\sin\varphi)/2\sin\varphi}}. \tag{12}$$

The relationship between support reaction force and surrounding rock plastic radius under different rock mass parameters and seepage conditions can be obtained from equations (11) and (12).

(4) Displacement analysis of surrounding rock

After the tunn5el is excavated and lined, the initial in situ stress and seepage field of surrounding rock will change, resulting in displacement and deformation of rock mass. The displacement of surrounding rock is divided into elastic zone displacement and plastic zone displacement.

In the elastic zone of surrounding rock, according to the theory of elasticity, the displacement of rock mass is

$$\begin{cases} u_e = \dfrac{1-\mu^2}{E_s} r\left(\Delta\sigma_\theta - \dfrac{\mu}{1-\mu}\Delta\sigma_r\right), & r_1 \leqslant r \leqslant R_p, \\ u_p = \dfrac{1-\mu^2 R_p^2}{E_s r}\left(\Delta\sigma_0 - \dfrac{\mu}{1-\mu}\Delta\sigma_r\right), & R_p \leqslant r \leqslant R. \end{cases} \tag{13}$$

3. Analysis of Surrounding Rock Pressure and Displacement considering Softening

3.1. Mechanical Model. In order to qualitatively study the stress distribution of surrounding rock of circular tunnel under seepage, the following assumptions are made for the practical problems: (1) the calculation process is considered as the plane strain problem under axisymmetry, (2) the water-bearing surrounding rock is regarded as a two-phase medium satisfying Darcy's law, and (3) for the convenience of coupling calculation, the compressive stress is positive and the tensile stress is negative.

Figure 3 shows the mechanical model of tunnel surrounding rock. According to the stress-strain state of surrounding rock, the tunnel surrounding rock is divided into three areas, namely elastic area, plastic softening area, and plastic residual area. The surrounding rock in the elastic area is in a complete state. When the surrounding rock stress exceeds the strength limit of rock mass, the surrounding rock is in a plastic softening state. With the gradual increase in deformation, the strength of rock mass decreases gradually. Finally, the residual strength is reached. At this time, the surrounding rock enters the plastic residual area. See formulas (1) and (2) for the values of corresponding mechanical parameters. The excavation radius of the tunnel is R_0 (m), the distance from the center of the tunnel to the outer boundary of the plastic residual zone is R_r (m), and the distance from the center of the tunnel to the outer boundary of the plastic softening zone is R_p (m). The water pressure of the stable seepage field outside the radius R (m) is the same as the water pressure P_i (Pa) outside the original seepage field, and R can be obtained from borehole test [27]. The tunnel support force is P_0 (Pa), the initial in situ stress is σ_0 (Pa), and the side pressure coefficient λ is 1; that is, the horizontal in situ stress is equal to the vertical in situ stress.

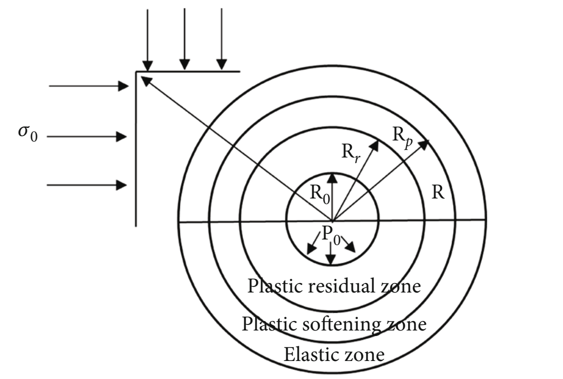

Figure 3: Mechanical model of tunnel surrounding rock.

3.2. Strain Softening Model. The strain softening of surrounding rock shows the opening and penetration of microcracks and the weakening of surrounding rock properties under stress. Macroscopically, it can be considered that the cohesion and internal friction angle of surrounding rock have changed, and the mechanical parameters of surrounding rock, cohesion c (Pa) and internal friction angle, have changed φ (°), respectively, η piecewise function:

$$c(\eta) = \begin{cases} c^p(\eta = 0), \\ c^p - (c^p - c^r)\dfrac{\eta}{\eta *}(0 < \eta < \eta *), \\ c^r(\eta \geq \eta *), \end{cases} \tag{14}$$

$$\varphi(\eta) = \begin{cases} \varphi^p(\eta = 0), \\ \varphi^p - (\varphi^p - \varphi^r)\dfrac{\eta}{\eta *}(0 < \eta < \eta *), \\ \varphi^r(\eta \geq \eta *). \end{cases} \tag{15}$$

where c^p and φ^p are the peak mechanical parameters of surrounding rock; c^r and φ^r are the residual mechanical parameter of surrounding rock; $\eta *$ is the plastic softening parameter, whose value is equal to the plastic shear strain. According to Reference [28], when $\eta = 0$, the surrounding rock is in the prepeak elastic stage. At this time, the c value of surrounding rock and the peak value φ is taken as the parameter. When $0 < \eta < \eta*$, the surrounding rock is in the plastic softening stage. With the η increase in the value, the c value and the φ value decrease gradually. When $\eta > \eta*$, the surrounding rock is in the plastic residual stage. At this time, the c value of surrounding rock and the φ value takes the residual parameter. The calculation formula obtained in Reference [29] is as follows:

$$\eta* = \left(\sigma_1^p - \sigma_1^r\right)\left(\frac{1}{E} + \frac{1}{M}\right)\left(1 + K_\psi\right), \tag{16}$$

$$K\psi = \frac{1 + \sin\psi}{1 - \sin\psi}, \tag{17}$$

where ψ is rock expansion angle (°); E is the elastic modulus of surrounding rock (Pa); and M is the slope of softening curve (Pa). Since there is little difference between the unloading slope and loading slope of the rock stress-strain curve, it is assumed that they are the same here.

3.3. Unified Strength Theory. The unified strength theory has many expressions. For rock materials, the internal friction angle of rock is used φ and rock cohesion c is expressed as follows:

When $\sigma_2 \leqslant ((\sigma_1 + \sigma_3)/2) - (\sigma_1 - \sigma_3/2)\sin\varphi$:

$$F = \sigma_1(1 - \sin\varphi) - \sigma_3'(1 + \sin\varphi) = 2C\cos\varphi, \tag{18}$$

$$\sigma_3' = \frac{b\sigma_2 + \sigma_3}{1 + b}. \tag{19}$$

When $\sigma_2 \geqslant ((\sigma_1 + \sigma_3)/2) - ((\sigma_1 - \sigma_3)/2)\sin\varphi$:

$$F' = \sigma_1'(1 - \sin\varphi) - \sigma_3(1 + \sin\varphi) = 2C\cos\varphi \tag{20}$$

$$\sigma_1' = \frac{\sigma_1 + b\sigma_2}{1 + b}, \tag{21}$$

where σ_1, σ_2, and σ_3 are the maximum principal stress, intermediate principal stress, and minimum principal stress, respectively; b is the influence degree of intermediate principal shear stress and normal stress on its action surface on material failure, and its value range is [0, 1].

3.4. Seepage Field Calculation. In the axisymmetric plane stable seepage field, considering that the rock mass has different permeability coefficients in the horizontal and vertical directions, Darcy's law in the two directions is expressed as

$$\begin{cases} V_x = -k_x \dfrac{\partial h}{\partial x}, \\ V_y = -k_y \dfrac{\partial h}{\partial y}, \end{cases} \tag{22}$$

where V is the seepage flow velocity (m/d); h is seepage potential (m); and k_x and k_y are permeability coefficient in horizontal and vertical directions (m/d).

The continuity equation of incompressible water flow in rock mass is

$$\frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} = 0. \tag{23}$$

Pore water pressure is $P_w = h \cdot \gamma_w$, and γ_w is the gravity of water (N/m^3). It is defined that the ratio of permeability coefficient in horizontal and vertical directions is uneven permeability coefficient ν, namely $k_x = \nu k_y$. When $\nu = 1$, the rock mass is isotropic.

The axisymmetric plane stable seepage differential equation is obtained by combining formulas (9) and (10) and expressed in the cylindrical coordinate system:

$$\frac{\partial^2 P_w(r)}{dr^2}\left(\nu\cos^2\beta + \sin^2\beta\right) + \frac{1}{r}\frac{dP_w(r)}{dr}\left(\nu\sin^2\beta + \cos^2\beta\right) = 0, \tag{24}$$

where β is the included angle between r and the horizontal coordinate axis $\delta = (\nu\cos^2\beta + \sin^2\beta)/(\nu\sin^2\beta + \cos^2\beta)$, so that the above formula becomes

$$\frac{\partial^2 P_2(r)}{dr^2} + \frac{1}{\delta r}\frac{dP_w(r)}{dr} = 0. \tag{25}$$

When $\nu = 1$, the above formula becomes an axisymmetric plane stable seepage field of isotropic rock mass. According to the boundary conditions $P_{w(r=R_0)} = 0$, $P_{w(r=R)} = P_i$, the distribution law of pore water pressure along the radius of tunnel surrounding rock is obtained as follows:

$$\begin{cases} P_{\omega(r)} = P_i \dfrac{\ln(r/R_0)}{\ln(R/R_0)} & (\nu = 1; R_0 \leqslant r \leqslant R), \\ P_{w(r)} = P_i\left(\dfrac{r^{(\delta-1)/\delta} - R_0^{(\delta-1)/\delta}}{R^{(\delta-1)/\delta} - R_0^{(\delta-1)/\delta}}\right) & (\nu \neq 1; R_0 \leqslant r \leqslant R). \end{cases} \tag{26}$$

4. Analysis of Surrounding Rock Pressure and Displacement considering Seepage and Softening

Because the surrounding rock of seepage tunnel is affected by internal water pressure, external water pressure, and in situ stress, under different working conditions, the first principal stress of surrounding rock may be either radial stress or tangential stress. When the tunnel is in construction or operation and the initial in situ stress is greater than the water pressure in the tunnel, if the initial in situ stress is greater than the water pressure in the tunnel or the tunnel is in construction, it is $\sigma_0 > P_0$; there are $\sigma_\theta > \sigma_r$ established. Because the research object of this paper is the deep buried diversion tunnel during the construction period, without considering the influence of internal water pressure, the first principal stress is tangential stress. For the plane strain problem, the intermediate principal stress is

$$s_2 = \frac{m}{2}(s_1 + s_3), \tag{27}$$

where m is the intermediate principal stress parameter, and for rock materials, $m = 1$.

4.1. Plastic Residual Zone. The equilibrium equation considering seepage is

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} + \alpha \frac{dP_{w(r)}}{dr} = 0, \tag{28}$$

where α is the action area coefficient of seepage water pressure. For the sake of safety, it is generally taken when studying the failure and stability of rock mass $\alpha = 1$ [21].

Since the maximum principal stress at the junction of plastic residual zone and plastic softening zone ($r = R_r$) is σ_{r1}, since the maximum principal stress under this working condition is tangential stress, that is, the stress boundary condition on the outer boundary of plastic residual area ($r = R_r$) is $\sigma_\theta^{pr} = \sigma_1^r$; the expression of radius Rr of the plastic residual area under this working condition is obtained:

$$R_r = R_0 \left[\frac{\sigma_1^r - A_1 B - A_2}{A_1 (P_0 - B)} \right]^{1/(A_1 - 1)}. \tag{29}$$

4.2. Plastic Softening Zone. The plastic softening zone also meets the strength criterion of the previous formula and the equilibrium equation of equation (28). The solution process of the stress equation is similar to that of the plastic residual zone. The stress expression of the plastic softening zone can be obtained through the stress boundary condition $\sigma_\theta^{ps} = \sigma_1^p$ on the inner boundary of the plastic softening zone ($r = R_r$):

$$R_p = R_r \left(\frac{\sigma_1^p - A_2 - A_1 B)}{\sigma_1^r - A_2 - A_1 B} \right)^{1/A_1}. \tag{30}$$

4.3. Elastic Zone. It is assumed that the radial stress σ_r^{gp} is caused by in situ stress on the contact surface between the plastic softening zone of surrounding rock and the elastic zone, and the elastic zone of surrounding rock can be regarded as the initial in situ stress at infinity σ_0. With radial compressive stress on the elastic-plastic contact surface for thick-walled cylinder under the combined action of σ_{rp} and seepage water pressure, the expression of stress in the elastic zone is

$$\begin{cases} \sigma_\theta^c = \sigma_0 \left(1 + \dfrac{R_p^2}{r^2} \right) - \sigma_r^{gp} \dfrac{R_p^2}{r^2} + \alpha P_w, \\[2ex] \sigma_r^e = \sigma_0 \left(1 - \dfrac{R_p^2}{r^2} \right) + \sigma_r^{gp} \dfrac{R_p^2}{r^2} + \alpha P_w. \end{cases} \tag{31}$$

From the sum of tangential stress and radial stress, which is continuous at the elastic-plastic interface of surrounding rock ($r = R_r$), it can be obtained that the peak stress completely caused by in situ stress without seepage is

$$\sigma_\theta^{gp} = \sigma_1^{gp} = \frac{2\sigma_0 A_1 + A_2}{1 + A_1}, \tag{32}$$

$$\sigma_r^{gp} = \sigma_3^{gp} = \frac{2\sigma_0 - \Lambda_2}{1 + A_1} \tag{33}$$

The stress expression of surrounding rock considering seepage and softening effect expressed by softening parameters and related rock parameters can be obtained.

5. Example Analysis

A tunnel in Yunnan is a separated tunnel. The starting and ending piles of the right line are K22+ 455-k22 + 711, with a length of 2256 m, and the starting and ending piles of the left line are zk22+ 437-zk24 + 680, with a length of 2243 m. It is a long highway tunnel. The maximum design speed of the whole tunnel is 100 km/h, the net width of the tunnel is limited to 14.5 m, and the net height is 5 m. The tunnel type is downhill tunnel, with the right longitudinal slope gradient of -2.5% and the right longitudinal slope gradient of -2.5%. The center line distance is 22 m-30 m, with a total length of 4499 m. The inlet section adopts biased end wall and end wall portal, the outlet section adopts open hole portal, and the lighting mode adopts photoelectric lighting. The ventilation mode is mechanical ventilation, and the mechanical parameters of tunnel surrounding rock are shown in Table 1. The construction site of tunnel entrance is shown in Figure 4.

5.1. Effect of Nonuniform Permeability Coefficient on Pore Water Pressure Distribution. It can be seen from the previous discussion that there are many factors affecting the distribution of pore water pressure in surrounding rock. The nonuniform permeability coefficient is analyzed below ν having influence on pore water pressure distribution in different directions of surrounding rock. Figure 5 shows different ν. The horizontal axis is the ratio of the distance r from the point to the center of the tunnel and the influence radius R of the external water pressure of the original seepage field. It can be seen from Figure 5 that the uneven permeability coefficient has an impact on the pore water pressure distribution in different directions. Figure 5(a) shows that when the external water pressure is constant, with the increase in uneven permeability coefficient, the growth rate of pore water pressure along the 0° direction (horizontal direction) gradually slows down and finally tends to the external water pressure. The variation trend of pore water pressure along the 30° direction is similar to that in the 0° direction (Figure 5(b)), but different ν. The difference of growth rate under value is not as obvious as the latter; Figures 5(c) and 5(d) show that the change trend of pore water pressure in the 60° direction is the same as that in the 90° direction (vertical direction), and the growth rate of pore

Table 1: Mechanical parameters of surrounding rock of the example project.

Mechanical parameters	Value	Mechanical parameters	Value
Elastic modulus E (Gpa)	7.8	Crustal stress σ_0	22
Strength attenuation modulus M (GPa)	7.8	External water pressure P_i	6
Peak internal friction angle φ^p (°)	35	Density ρ (g·cm^{-3})	2.6
Residual internal friction angle φ^r (°)	30	Poisson's ratio μ	0.25
Peak cohesion C^p (MPa)	1.5	Dilatancy angle ψ (°)	10
Residual cohesion C^r (MPa)	0.9	Critical plastic softening coefficient η^*	0.007

Figure 4: Construction site drawing of tunnel entrance.

water pressure gradually accelerates with the increase in uneven permeability coefficient, which is different in the 90° direction ν. The difference of growth rate under the value is more obvious than that in the 60° direction. This shows that with ν the change of value, the seepage shows anisotropy in different directions, and the closer to the horizontal or vertical direction, the more vulnerable the pore water pressure distribution is to the influence of uneven permeability coefficient. Specifically, when $\nu < 1$, the closer to the horizontal direction, the faster the change speed of pore water pressure. When $\nu > 1$, the closer to the vertical direction, the faster the pore water pressure changes. This shows that when $\nu < 1$, that is, when the permeability coefficient in the horizontal direction is less than that in the vertical direction, the reduction range of pore water pressure in the horizontal direction is much less than that in the vertical direction in the surrounding rock deep away from the tunnel free face. Because the boundary condition of hydraulic force at the tunnel free face is a certain value, therefore, the calculation results show that the hydraulic gradient increases sharply in the horizontal direction near the free face.

5.2. Influence of Seepage on Different Zoning Ranges and Stress Distribution of Surrounding Rock. In order to reveal the effects of strain softening and seepage on the stress and plastic zone radius of surrounding rock, it is compared with the elastic-plastic solutions of the following three cases: no strain softening, no seepage, and neither softening nor seepage. In order to simplify the calculation, the attenuation of rock material parameters in the softening zone is regarded as a linear change.

When the intermediate principal stress coefficient $b = 0$, the unified strength criterion degenerates to the Mohr-Coulomb strength criterion. The relationship curve between P_0 and plastic zone radius R_p in 4 cases is calculated, as shown in Figure 6. It can be seen from Figure 6 that P_0 has a significant impact on the radius R_p of the surrounding rock plastic zone. When B is constant, with the increase in P_0, the radius R_p of the plastic zone gradually decreases until there is no plastic zone (because the excavation radius $P_0 = 2$ m, $R_p < 2$ m indicates that the plastic zone does not exist). When $P_0 = 4$ MPa, the R_p corresponding to the four cases is 27.2%, 15.5%, 19.7%, and 12.8% lower than that corresponding to $P_0 = 2$ MPa. When $P_0 = 10$ MPa, the R_p corresponding to the four cases is only 10.5%, 6.3%, 8.9%, and 5.2% lower than that when $P_0 = 8$ MPa, indicating that the support force can reduce the plastic zone of surrounding rock within a certain range, and the smaller the support force, the more significant the impact. When considering seepage and softening, the obtained R_p is greater than that when only a single factor is considered or neither is considered. Therefore, during tunnel construction, the rock mass should be reinforced by grouting treatment to reduce the softening degree and permeability coefficient of surrounding rock, reduce the range of plastic zone of surrounding rock and improve the stability of surrounding rock. The calculated R_p only considering softening is greater than that only considering seepage, and R_p in both cases is greater than that calculated without considering both, indicating that both seepage and material softening will affect the stability of surrounding rock, which should not be ignored in calculation, and material softening is more unfavorable to the stability of surrounding rock than seepage.

Figure 7 shows the distribution of tangential stress of surrounding rock under 4 conditions when $P_0 = 2$ MPa and $b = 0$, and the first principal stress generally controls the surrounding rock. Under this working condition, the tangential stress is the first principal stress, so only the influence of various parameters on tangential stress is analyzed. It can be seen from Figure 7 that near the free face of the tunnel, the tangential stress value considering material softening is less than that without considering material softening. This is because the mechanical parameters of surrounding rock in the plastic zone deteriorate, resulting in the decline of the bearing capacity of surrounding rock and the transmission of stress to the depth of surrounding rock, resulting in the continuous expansion of the plastic zone. At the same time, the tangential stress distribution of surrounding rock considering the influence of seepage field and not considering the influence of seepage field is basically the same near the

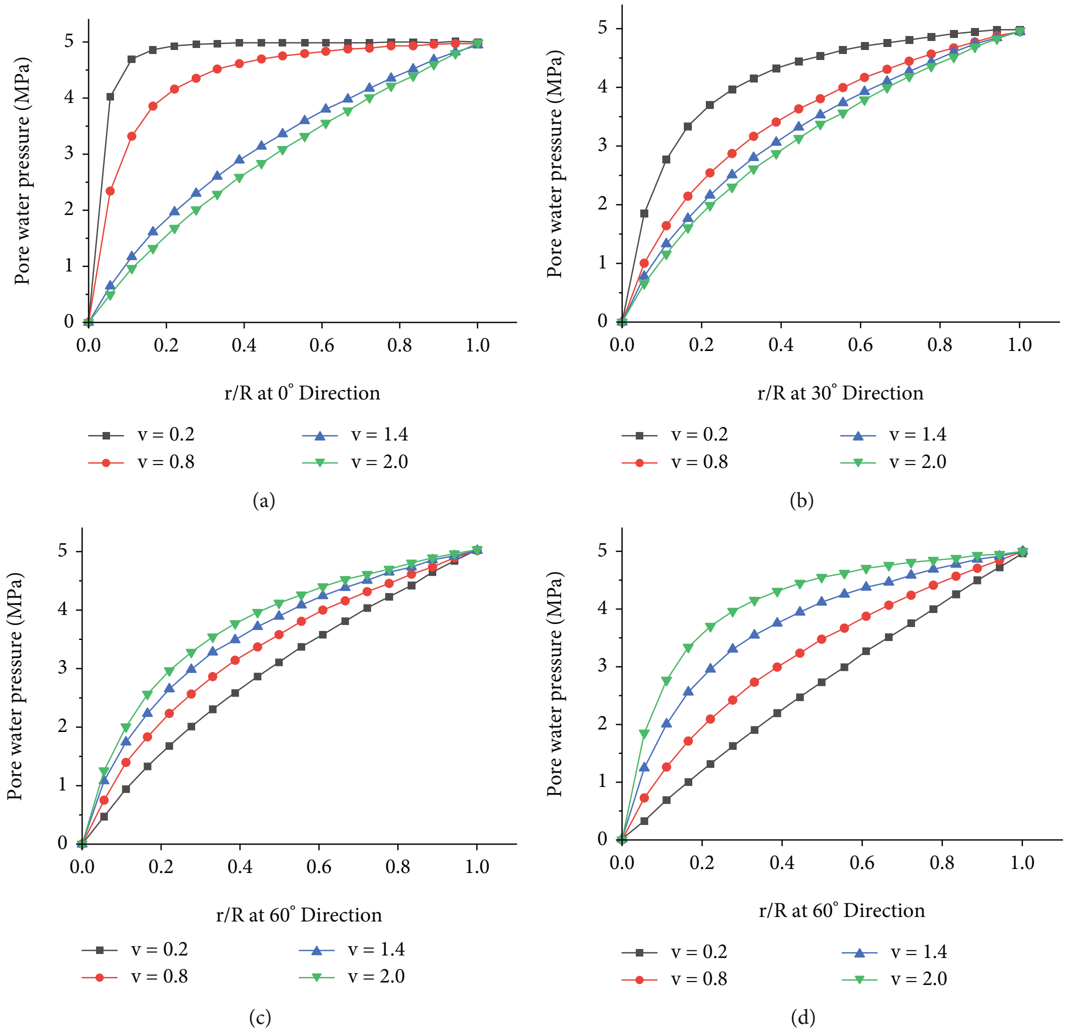

FIGURE 5: Different v pore water pressure distribution of surrounding rock in different directions under the condition of value.

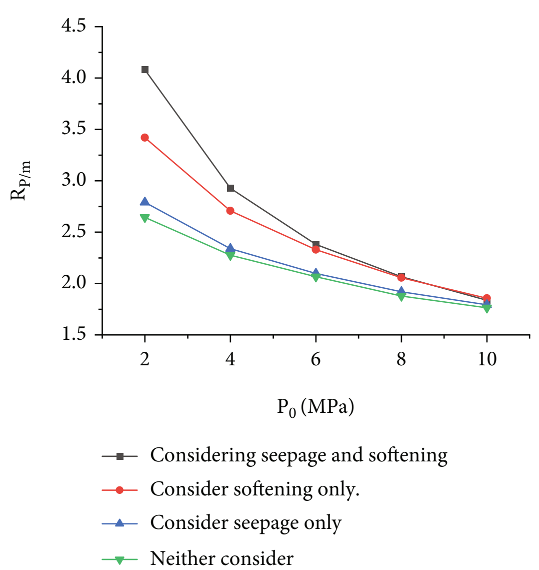

FIGURE 6: Relationship curve between P_0 and R_p in 4 cases ($b = 0$).

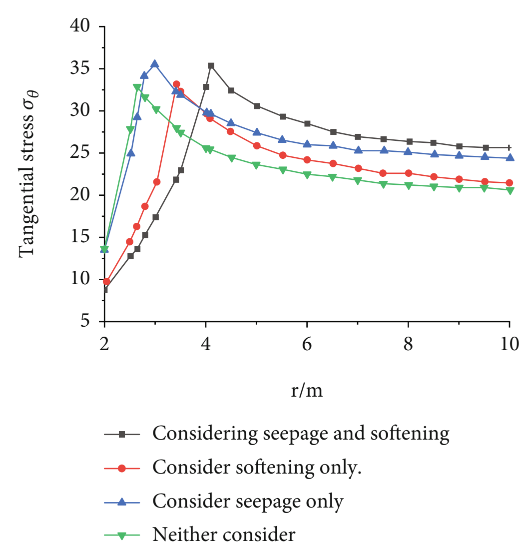

FIGURE 7: Tangential stress distribution of surrounding rock in 4 cases ($P_0 = 2$ MPa, $b = 0$).

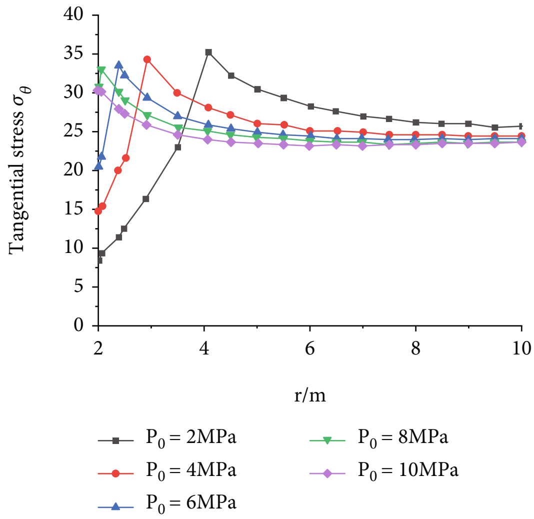

Figure 8: Tangential stress distribution of surrounding rock corresponding to different P_0 (considering seepage and softening, $B = 0$).

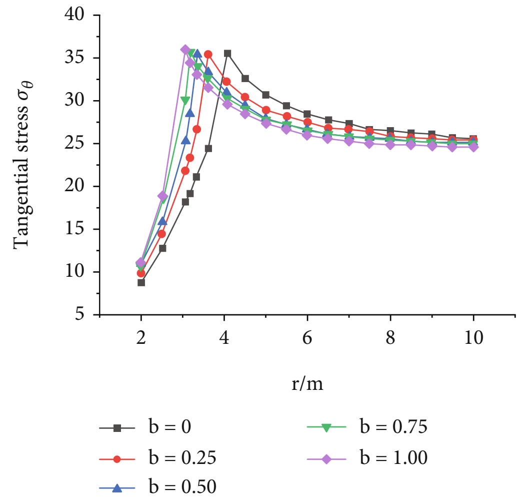

Figure 9: Tangential stress distribution of surrounding rock corresponding to different b values ($P_0 = 2$ MPa).

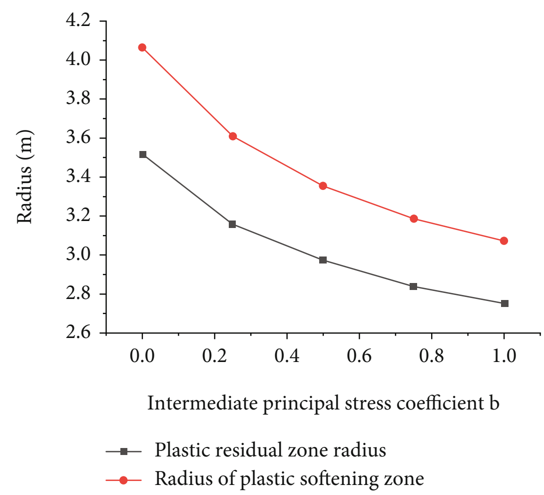

Figure 10: Radius of the plastic zone corresponding to different b values.

free face, but near the depth, the tangential stress of surrounding rock considering the influence of seepage field is gradually greater than that without considering the effect of seepage field. This is because when the seepage field is not considered, the support reaction acting on the tunnel wall is considered surface force, and its action range is limited. The seepage pressure is actually a volume force acting on any point of the stress field.

Figure 8 shows the tangential stress distribution of surrounding rock corresponding to different P_0 when $b = 0$ considering seepage and softening conditions, and the radius of plastic zone is shown in Figure 6. It can be seen from Figure 8 that with the increase in P_0, the tangential stress of surrounding rock near the free face gradually increases. P_0 increases from 2 MPa to 10 MPa, the tangential stress at the inner wall of the tunnel increases from 8.6 MPa to 30.4 MPa, an increase in 2.5 times, while the peak tangential stress decreases from 35 MPa to 30.6 MPa, a decrease in 14.4%.

5.3. Influence of Intermediate Principal Stress Coefficient on Different Zoning Ranges and Stress Distribution of Surrounding Rock. Based on the unified strength theory criterion, the elastic-plastic analytical solution of tunnel surrounding rock is obtained by comprehensively considering the influence of seepage field and surrounding rock strain softening. It is known that this series of solutions can be transformed into known solutions by changing the intermediate principal stress coefficient b. For example, when b is equal to 0, 0.25, 0.50, 0.75, and 1.00, respectively, this solution can be transformed into a special solution satisfying Mohr-Coulomb, double shear strength criterion, and other strength criteria. By changing the intermediate principal stress coefficient b, the influence of the intermediate principal stress on the tangential stress of surrounding rock under seepage and softening effect is analyzed. Figure 9 shows the tangential stress distribution of surrounding rock corresponding to different b values when $P_0 = 2$ MPa, and Figure 10 shows the radius of the plastic zone corresponding to different b values. It can be seen from Figures 9 and 10 that b value has a significant impact on the tangential stress of surrounding rock and the range of plastic zone. With the increase in b value, the radius of plastic zone gradually decreases, while the tangential stress of surrounding rock near the free face gradually increases. When considering the intermediate principal stress effect ($b = 0.25$, 0.50, 0.75, and 1.00), the peak tangential stress of surrounding rock increases by 0.2%, 0.8%, 0.9%, and 1.8%, respectively, compared with that without considering the intermediate principal stress effect ($b = 0$), and the radius of the plastic zone decreases by 10.8%, 17.2%, 21.3%, and 24.2%, respectively, which shows that compared with considering the intermediate principal stress effect, without considering the effect of intermediate principal stress, the calculation results are relatively conservative and cannot give full play to the strength of surrounding rock effectively.

6. Conclusions

(1) In order to study the interaction between surrounding rock and lining of deep buried tunnel considering the influence of seepage, based on Mohr Coulomb criterion, the interaction between surrounding rock and lining of deep buried tunnel is analyzed by convergence constraint method. Based on the elastic-plastic solution, the nonlinear elastic-plastic solution of the interaction between surrounding rock and lining structure considering the effect of seepage force is proposed, and the radius of surrounding rock plastic zone is obtained The relationship between surrounding rock stress and displacement, radial deformation of lining and support reaction force. At the same time, the seepage, strain softening and intermediate principal stress of surrounding rock are analyzed as parameters, and the calculation results are combined with the properties of surrounding rock. The surrounding rock is divided into plastic residual area, plastic softening area, and elastic area. The stress distribution expression of plastic area and surrounding rock in each area of circular tunnel is deduced

(2) The radial displacement around the tunnel is mainly related to the nonlinear deformation of surrounding rock lining. After the tunnel is excavated and lined, under the action of surrounding rock earth pressure and seepage water pressure, the surrounding rock and lining deform, and the support reaction provided by the lining gradually increases. The two coordinate deformation, making the tunnel tend to be stable. The final radial displacement around the tunnel depends on the convergence curve of surrounding rock and the support characteristic curve

(3) Define the coefficient of nonuniform permeability ν to quantitatively analyze the influence of two-way unequal permeability coefficient on pore water pressure distribution in all directions of surrounding rock: with ν with the change of value, the seepage shows anisotropy in different directions. When $\nu < 1$, the closer to the horizontal direction, the faster the change speed of pore water pressure. When $\nu > 1$, the closer to the vertical direction, the faster the pore water pressure changes

(4) Through the comparison of numerical examples, seepage and material softening have varying degrees of influence on the distribution of surrounding rock stress field and the size of plastic zone: due to the deterioration of surrounding rock mechanical parameters, the tangential stress when considering softening is smaller than when not considering softening, and the radius of plastic zone is larger. At the same time, The softening characteristic of the studied material has a greater influence on the stability of surrounding rock than that of seepage

(5) The intermediate principal stress coefficient b has a significant impact on the tangential stress of surrounding rock and the range of plastic zone. With the increase in b value, the radius of the plastic zone decreases gradually, while the tangential stress of surrounding rock near the free face increases gradually. In the calculation example in this paper, when $b = 1$, the peak tangential stress of surrounding rock increases by 1.8% compared with $B = 0$, and the radius of plastic zone decreases by 25%, which shows that compared with considering the intermediate principal stress effect, the calculation results are relatively conservative without considering the intermediate principal stress effect and cannot give full play to the strength of surrounding rock effectively

Data Availability

The experimental data used to support the findings of this study are included within the article.

Conflicts of Interest

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

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