

Research Article

Application of Modified Hoek–Brown Strength Criterion in Water-Rich Soft Rock Tunnel

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When a tunnel is excavated in the water-rich soft rock stratum, the strength of the soft rock is greatly reduced due to the seepage of groundwater. The condition may result in engineering accidents, such as large deformation, limit invasion, and even local collapse of the tunnel. Therefore, it is very important to research the stability of the surrounding rock in the water-rich soft rock tunnel. The water-rich disturbance factor considering the seepage influence of groundwater and blasting disturbance is proposed, and the generalized Hoek–Brown strength criterion is modified on the basis of the immersion softening test of soft rock. In accordance with the classical elastic–plastic mechanics theory, the stress, strain, and displacement calculation formulas of the tunnel surrounding rock are derived. The displacement of tunnel surrounding rock is analyzed using the derived formula and the modified Hoek–Brown strength criterion and then compared with the measured value. Results show that the displacement of surrounding rock, which is calculated by modified Hoek–Brown strength criterion considering water-rich disturbance factor and the displacement calculation formula, is close to the measured deformation of surrounding rock in water-rich soft rock tunnel, and the error is small. Therefore, the modified Hoek–Brown strength criterion can be applied to the water-rich soft rock tunnel, and the derived displacement calculation formula can accurately calculate the deformation of tunnel surrounding rock. It is of great significance to the study of surrounding rock stability of water-rich soft rock tunnel.

1. Introduction

With the rapid development of highway tunnels and railway tunnels in China, more and more tunnels need to be built in water-rich soft rock strata [1]. Water is an important factor affecting the deformation and failure mode of soft rock [2]. Usually, water negatively affects the mechanical properties, long-term strength, and stability of soft rock [3]. In most cases, due to the disturbance of tunnel excavation, the state of groundwater changes, thereby significantly changing the strength of the surrounding rock. It is an important reason for affecting the stability of surrounding rock in soft rock tunnel [4, 5]. Therefore, studying the influence of groundwater on the stability of surrounding rock in the water-rich soft rock stratum is important.

In the past 30 years, many scholars have found that groundwater has a great influence on the stability of surrounding rock after tunnel excavation. Some scholars have conducted a number of studies on the softening effect of

water on the soft rock strength and deformation and have made considerable achievements. For example, Yang et al. [6] studied the microstructure and failure characteristics of argillaceous slate under different immersion time. They found that with the extension of immersion time, the internal pores of rock samples gradually increased, and the failure mode gradually developed from splitting failure to shear failure. Shakoor and Barefield [7] studied the change in sandstone strength under different water contents. They found that within the range of water content lower than 20%, the strength of surrounding rock decreased significantly with the increase in water content. However, when the water content was higher than 20%, the strength decreased insignificantly. Nara et al. [8] studied the microstructure of shale under different immersion time and found that the increase rate of internal cracks increased with the extension of immersion time. Azhar et al. [9] tested and analyzed the mechanical properties of clay-rich sandstone under different immersion time and found that the elastic modulus and compressive

strength decreased with the increase of immersion time. Zhang et al. [10] studied the failure characteristics of soft rock under different immersion time and analyzed the failure mode of tunnel when water gushing occurred. Sun et al. [11] studied the softening characteristics and failure mode of rock under different immersion time and analyzed the failure mode of tunnel under different water contents. Bian et al. [12] considered the shale of Huangjialing tunnel as the research object, studied the changes in mineral composition and microstructure under different immersion time, analyzed the failure modes under different immersion time, and predicted the failure modes of the tunnel under different water contents.

The above research results only analyzed the softening properties and failure modes of soft rock after immersion, which can be used to study the mechanism of large deformation of water-rich soft rock tunnel and predict the failure mode of the tunnel. However, it cannot be used to analyze the stability of tunnels. In tunnel construction, the stability of surrounding rock is often judged by monitoring the deformation of surrounding rock. If the cumulative deformation of surrounding rock is small, the surrounding rock of tunnel is considered to be stable; on the contrary, if the cumulative deformation of surrounding rock is large, it is considered that the stability of surrounding rock is poor, and corresponding measures should be taken to strengthen support, such as temporary inverted arch, biological improvement grouting reinforcement [13, 14], and other measures to strengthen support. The deformation of surrounding rock during tunnel excavation should be calculated for the analysis of tunnel surrounding rock stability. In the calculation of surrounding rock displacement, selecting the appropriate calculation method and strength criterion can obtain more accurate calculation results. Considering the deformation capacity of surrounding rock, Thirukumaran et al. [15] proposed a stability analysis method of tunnel vault rock mass, which can be used to analyze the stability of the surrounding rock. But there is still a large gap between the calculation results of simplifying the vault rock block to a rectangular block and the actual deformation. To make up for the deficiency that the Mohr–Coulomb strength criterion can only describe the linear failure characteristics of rock, Hoek and Brown [16] proposed Hoek–Brown strength criterion, which can reflect the nonlinear failure characteristics, on the basis of Griffith's theoretical research results. Hoek et al. [17] improved the Hoek–Brown strength criterion and proposed the generalized Hoek–Brown strength criterion with wider applicability. Meanwhile, the blasting disturbance factor (D) and the geological strength index (GSI) were introduced to calculate the parameters. Sonmez and Ulusay [18] comprehensively considered the geological strength index and the disturbance effect in actual construction and corrected the blasting disturbance factor, which can calculate the displacement of tunnel surrounding rock more accurately. However, due to the cumbersome calculation process, it is less used at present. Chen et al. [19, 20] based on the damage law of rock and the reduction effect of cohesion and internal friction on the stability of rock proposed the damage evolution equation of fractured rock mass and established the damage constitutive

equation of rock, which can be used for the stability analysis of fractured rock mass. Lee and Pietruszczak [21] proposed the method of numerical difference and considered the softening law of strength parameters, such as cohesion, internal friction angle, and dilatancy angle. They deduced the displacement analysis formula of tunnel surrounding rock by using the generalized Hoek–Brown strength criterion, which makes the calculation results more suitable for the actual tunnel deformation. Xia et al. [22] used the strength reduction method and catastrophe theory to study the stability of surrounding rock and applied the research theory to practical engineering, which achieved good results. Wu et al. [23] combined the Levenberg–Marquardt optimization technique with complex variable differential method and proposed a modified optimization technique for stress-seepage coupling problem, which can accurately and effectively estimate multiple rock mass parameters. Xue et al. [24] established a seepage model, which can simulate groundwater flow from aquifer to fault by coupling the Darcy flow model and the Fokheimer flow model and can be used to analyze the stability of tunnel when fractured rock mass gushes water.

The existing research results can accurately calculate the deformation of the tunnel surrounding rock in the natural state. However, the softening effect of groundwater on soft rock is not considered. There is still a large error in the calculation of the surrounding rock displacement in a water-rich soft rock tunnel. Thus, the stability of the surrounding rock of the water-rich soft rock tunnel cannot be accurately analyzed. Therefore, the variation law of physical and mechanical properties of soft rock is analyzed through the immersion softening test of soft rock. On this basis, the water-rich disturbance factor considering the influence of groundwater on the mechanical properties of soft rock is proposed. It is introduced into the generalized Hoek–Brown strength criterion for modification. The modified Hoek–Brown strength criterion can be applied to the displacement calculation and stability analysis of water-rich soft rock tunnel. The applicability of the modified Hoek–Brown strength criterion is verified by monitoring data in engineering practice.

2. Water Immersion Softening Test of Soft Rock Specimens

2.1. Preparation of Specimens. The rock samples are obtained from the face of the Xiejiaopo tunnel, which is a water-rich soft rock tunnel, and the lithology is mainly carbonaceous phyllite. In accordance with the requirements of “Test method standard for engineering rock mass” (GB/T50266-2013) and on the basis of the actual test scheme and conditions, the rock specimens were made into cylindrical blocks with a diameter of 50 mm and height of 100 mm, as shown in Figure 1. The height-diameter ratio is not less than 2.0, the diameter error is less than 3 mm, and the nonparallelism of the end face is less than 0.05 mm. The end face of the rock sample is perpendicular to the axis, and the maximum deviation is less than 0.25 degrees, which is in line with the provisions of the International Society of Rock Mechanics (ISRM) for rock samples.



FIGURE 1: Prepared phyllite specimens.

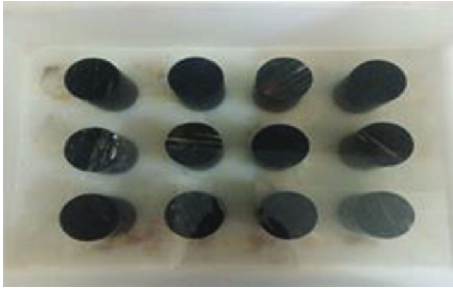


FIGURE 2: Phyllite specimen immersed in water.

The surface integrity of the initially processed rock samples was examined to prevent the large discreteness of the test data. The RSM-SY5(T) nonmetallic acoustic testing instrument was used to detect the wave velocity of the rock samples. The rock samples with no evident surface defects and similar wave velocity were selected for the test.

2.2. Test Scheme and Equipment. The prepared samples were divided into five groups with three blocks in each group. Five groups of test blocks were placed in a 50 cm × 45 cm × 30 cm tank, and water was added until the samples were completely submerged, as shown in Figure 2. Five groups of test blocks were immersed for 0, 30, 90, 180, and 270 days. The P-wave velocity test, porosity test, and uniaxial compression test of soft rock under different immersion time were carried out to study the softening characteristics of soft rock.

2.2.1. P-Wave Velocity Test. The P-wave velocity of the samples soaked for 0, 30, 90, 180, and 270 days was tested. The test equipment was RSM-SY5(T) acoustic wave tester, as shown in Figure 3. Three P-wave velocity tests were carried out on each sample, and the average value of the three tests was considered the P-wave velocity value of the sample.

2.2.2. Porosity Test. The porosity test was carried out when the samples were soaked for 0, 30, 90, 180, and 270 days. The test equipment was MesoMR23-060 H-I nuclear magnetic resonance instrument. Prior to the nuclear magnetic test, the specimen required vacuum saturation. The porosity of material was calculated accurately by analysing the relaxation behaviour of proton in rock under a magnetic field.

2.2.3. Uniaxial Compression Test. Uniaxial compression tests were carried out by YA-2000 digital pressure testing machine, as shown in Figure 4. Uniaxial compression tests were carried out on specimens immersed in water for 0, 30,



FIGURE 3: P-wave velocity test.



FIGURE 4: Uniaxial compression test.

90, 180, and 270 days. The load was applied at the loading rate of 0.5 kN per second and continuously collected until the test block was destroyed.

2.3. Mechanical Properties of Soft Rock under Different Immersion Time. The parameters of soft rock under different immersion time are obtained by analysing the test data, as shown in Table 1.

Generally, the lithology of surrounding rock in the same section is similar. Table 1 shows that the mechanical properties of the soft rock are the same under the same immersion time, therefore, it can be considered that the mechanical properties of the surrounding rock at the same section in water-rich soft rock tunnel are the same. Poisson's ratio of soft rock does not change significantly with the extension of immersion time. The change trend of elastic modulus and uniaxial compressive strength gradually reduced in the early stage and then stabilized. The experimental data were fitted by exponential function, power function, and logarithmic function, and the corresponding mathematical model was established. The functional relationship between immersion time and elastic modulus and compressive strength was obtained. The fitting formula and correlation coefficient are shown in Table 2.

Considering the correlation coefficient and relative error, the formula with the highest fitting degree was determined, as shown in Formula (1).

$$\begin{cases} E_t = 0.894E_0e^{-3 \times 10^{-3}t}, \\ \sigma_{ct} = 0.949\sigma_0e^{-1 \times 10^{-3}t}, \end{cases} \quad (1)$$

TABLE 1: Parameters of soft rock under different immersion time.

Test specimen number	Immersion time (t/d)	Compressive strength (σ_{ct} /MPa)	Poisson ratio (μ)	Elastic modulus (E/GPa)	P-wave velocity (V/m·s ⁻¹)	Porosity (n/%)
1-1	0	23.58	0.337	33.214	2650	0.467
1-2	0	23.40	0.348	33.383	2647	0.446
1-3	0	22.96	0.333	32.656	2539	0.421
2-1	30	18.12	0.336	30.007	2638	0.572
2-2	30	18.45	0.332	30.131	2609	0.568
2-3	30	18.32	0.337	29.952	2629	0.583
3-1	90	14.35	0.337	26.935	2578	0.762
3-2	90	15.13	0.330	27.917	2541	0.695
3-3	90	14.96	0.329	27.272	2558	0.707
4-1	180	10.92	0.336	25.029	2501	1.098
4-2	180	11.38	0.327	24.960	2458	0.974
4-3	180	11.10	0.336	25.510	2418	1.063
5-1	270	9.96	0.328	24.929	2449	1.218
5-2	270	9.82	0.334	24.137	2439	1.196
5-3	270	10.58	0.336	25.017	2321	1.101

TABLE 2: Fitting formula and correlation coefficient comparison table.

Form of fitting function	Fitting formula	Correlation coefficient
Exponential function	$E_t = 0.894E_0e^{-3 \times 10^{-3}t}$	0.998
	$\sigma_{ct} = 0.949\sigma_0e^{-1 \times 10^{-3}t}$	0.985
	$V_t = 1.007V_0e^{-3 \times 10^{-4}t}$	0.995
	$(1 - n_t) = (1 - n_0)e^{-3 \times 10^{-5}t}$	0.971
Power function	$E_t = 0.864E_0t^{-0.094}$	0.859
	$\sigma_{ct} = 0.944\sigma_0t^{-0.035}$	0.822
	$V_t = 0.994V_0t^{-0.008}$	0.681
	$(1 - n_t) = 1.455(1 - n_0)t^{0.0624}$	0.764
Polynomial function	$E_t = 0.0001t^2 - 0.0555t + E_0$	0.977
	$\sigma_{ct} = 0.0002t^2 + 0.0721t + \sigma_0$	0.977
	$V_t = -0.00002t^2 - 0.8588t + V_0$	0.809
	$(1 - n_t) = -0.000005t^2 + 0.004t + (1 - n_0)$	0.967

where σ_{ct} is the uniaxial compressive strength of soft rock when the soaking time is t ; σ_0 is the uniaxial compressive strength of the soft rock in the natural state, $\sigma_0 = 23.313$ MPa; E_t is the elastic modulus of soft rock when the soaking time is t ; E_0 is the elastic modulus of soft rock in the natural state, $E_0 = 33.084$ GPa; t is immersion time.

The change in P-wave velocity and porosity can reflect the change in mechanical properties. Therefore, the P-wave velocity and rock porosity in Table 1 are analyzed. The table shows that with the extension of soaking time, the P-wave velocity of soft rock decreased gradually in the early stage and stabilized in the later stage. On the contrary, with the

extension of soaking time, the porosity increased slowly in the early stage and then increased rapidly. The P-wave velocity and porosity of soft rock were fitted with the immersion time, and the formula with the highest fitting degree was selected, as shown in Formula (2), as follows:

$$\begin{cases} V_t = 1.007V_0e^{-3 \times 10^{-4}t}, \\ (1 - n_t) = (1 - n_0)e^{-3 \times 10^{-5}t}, \end{cases} \quad (2)$$

where V_t is the P-wave velocity of soft rock when the soaking time is t ; V_0 is the P-wave velocity of soft rock in the natural state, $V_0 = 2612$ m·s⁻¹; n_t is the porosity of soft rock when

the soaking time is t ; n_0 is the porosity of soft rock in the natural state, $n_0 = 0.4445\%$.

3. Modified Hoek–Brown Strength Criterion

3.1. Generalized Hoek–Brown Strength Criterion. In 1980, based on the theoretical research results of Griffith, Hoek and Brown [16] proposed Hoek–Brown strength criterion through a large number of rock tests, which can reflect the nonlinear failure characteristics of rock mass and make up for the deficiency of linear Mohr–Coulomb strength criterion. Then, Hoek et al. [17] improved the Hoek–Brown strength criterion and proposed a generalized Hoek–Brown strength criterion with wider applicability, such as Formula (3), as follows:

$$\sigma_1 = \sigma_3 + \sigma_c \left[m_b \frac{\sigma_3}{\sigma_c} + s \right]^\alpha, \quad (3)$$

where σ_c is the compressive strength of rock; m_b , s , and α are the parameters related to rock lithology. Hoek introduced the geological strength index (GSI) for the calculation to determine the three parameters. Then, considering the influence of blasting and other construction measures on the stability of surrounding rock, the blasting disturbance factor (D) is introduced. The specific calculation formula is shown in Formula (4), as follows:

$$\begin{cases} m_b = m_i \exp\left(\frac{\text{GSI} - 100}{28 - 14D}\right), \\ s = \exp\left(\frac{\text{GSI} - 100}{9 - 3D}\right), \\ \alpha = 0.5 + \frac{1}{6} \left[\exp\left(-\frac{\text{GSI}}{15}\right) - \exp\left(\frac{-20}{3}\right) \right], \end{cases} \quad (4)$$

where m_i is the m value of the rock block without joints and beddings; it can be obtained by referring to the relevant specifications through the lithology, hardness, and mineral composition of the rock; D is the blasting disturbance factor, which can be determined by the lithology and construction of surrounding rock; GSI is the geological strength index of rock.

3.2. Water-Rich Disturbance Factor. To apply the Hoek–Brown strength criterion to the water-rich soft rock tunnels, the softening effect of groundwater should be considered. To a certain extent, the change in the mechanical properties of soft rock after immersion can reflect the seepage influence of groundwater on the mechanical properties of surrounding rock in soft rock tunnel.

The elastic modulus of rock is an index used to describe the elastic deformation resistance of rock, so it can best reflect the change of mechanical properties of rock. According to the water immersion softening test of soft rock and the principle of damage mechanics, the change in elastic modulus under water-rich condition is used to characterise the influence of groundwater on soft rock, and the water-rich influ-

ence factor is proposed. The definition is shown in Formula (5), as follows:

$$D_\omega = 1 - \frac{E_\omega}{E_0}, \quad (5)$$

where D_ω is the water-rich influence factor, the value range is $0 \leq D_\omega \leq 1$; E_ω is the elastic modulus value after the influence of rich water that can be obtained by substituting the specific immersion time into E_t ; E_0 is the elastic modulus under natural state.

The elastic modulus can be obtained by uniaxial compression test of rock and analysis of stress-strain curve. The porosity and P-wave velocity of rock can be obtained only by simple nondestructive test, and the test is relatively simple and fast. Therefore, the P-wave velocity and porosity are selected to calculate the damage variable of rock. According to the propagation theory of elastic wave in rock, the P-wave velocity of rock is related to elastic modulus, Poisson's ratio, and rock density, such as Formula (6), as follows:

$$V_p = \sqrt{\frac{E(1-\mu)}{\rho(1+\mu)(1-2\mu)}}, \quad (6)$$

where V_p is the P-wave velocity, which is different under different conditions. The P-wave velocity can be obtained by substituting different soaking time into Formula (2). E is the elastic modulus, which is different under different conditions. Elastic modulus can be obtained by substituting different soaking time into Formula (1). μ is the Poisson's ratio, and ρ is the density.

According to the results of the water immersion softening test, the Poisson's ratio of the soft rock has no evident change after soaking and can be ignored. Substitute the equality transformation of Formula (6) into Formula (5). At the same time, the relationship between the density and porosity of rock mass under unit mass is substituted into the water-rich influence factor, and the expression is shown in Formula (7), as follows:

$$D_\omega = 1 - \frac{1 - n_\omega}{1 - n_0} \frac{V_\omega^2}{V_0^2}, \quad (7)$$

where n_ω is the porosity of soft rock affected by water-rich conditions; n_0 is the porosity of soft rock before experiencing the influence of water-rich conditions; V_ω is the P-wave velocity affected by water-rich conditions; V_0 is the P-wave velocity of soft rock before experiencing the influence of water-rich conditions.

The relationship among P-wave velocity, porosity, and immersion time is substituted into the expression of the water-rich influence factor to accurately calculate the water-rich influence factor under different working conditions. The water-rich influence factor with timeliness is obtained,

such as Formula (8), as follows:

$$D_{\omega t} = 1 - \frac{1 - n_t}{1 - n_0} \frac{V_t^2}{V_0^2} = 1 - 1.014e^{-6.3 \times 10^{-4}t}, \quad (8)$$

where $D_{\omega t}$ is a water-rich influence factor considering timeliness.

According to the results of the soft rock immersion softening test, after 180 days of soft rock immersion, the changes in various parameters are small and tend to be stable. It can be assumed that in the water-rich section of the soft rock tunnel, the t value greater than 180 can be selected to calculate the water-rich influence factor; in the natural state, $t=0$ can be selected for calculation.

For water-rich soft rock tunnels, the effects of blasting disturbance and water-rich on tunnels should be considered simultaneously. In this regard, a water-rich disturbance factor considering blasting disturbance and groundwater effect is established. According to the principle of strain equivalence, the blasting disturbance factor is the first disturbance effect, and the water-rich disturbance factor is the second disturbance effect. Combined with the damage coupling principle in damage mechanics theory, the second disturbance effect can only affect other parts except for the first disturbance effect. Then, the expression of water-rich disturbance factor is defined in Formula (9), as follows:

$$D_m = D + D_\omega - DD_\omega, \quad (9)$$

where D_m is the water-rich disturbance factor considering the blasting disturbance factor and the water-rich influence factor; D is the blasting disturbance factor; D_ω is the water-rich influence factor.

By substituting Formula (8) into Formula (9), the expression of water-rich disturbance factor with timeliness can be obtained, as shown in Formula (10), as follows:

$$D_{mt} = 1 + 1.014D e^{-6.3 \times 10^{-4}t} - 1.014 e^{-6.3 \times 10^{-4}t}, \quad (10)$$

where D_{mt} is a water-rich disturbance factor with timeliness. The blasting disturbance factor D value can be determined in accordance with the lithology and construction of the site. In the water-rich section of the soft rock tunnel, the t value greater than 180 can be selected to calculate the water-rich influence factor; in the natural state, $t=0$ can be selected for calculation.

3.3. Modified Hoek–Brown Strength Criterion. From the parameter solution Formula (4), obtaining the three parameters m_b , s , and α in the modified Hoek–Brown strength criterion, the geological strength index value is required in addition to the value of water-rich disturbance factor. Hashemi et al. [25] proposed the method of estimating the GSI value by RMR value. Barton [26] proposed the relationship between RMR_{89} and rock mass quality index Q . Q was also related to the P-wave velocity of the rock mass. The spe-

cific relationship is shown in Formula (11), as follows:

$$\begin{cases} \text{GSI} = \text{RMR}_{89} - 5, \\ \text{RMR}_{89} = 15 \lg Q + 50, \\ Q = 10^{V_p/1000-3.5}, \end{cases} \quad (11)$$

where RMR_{89} is the geological classification parameter of the RMR geomechanical classification method, $\text{RMR}_{89} > 23$; Q is the rock mass quality index, and V_p is the P-wave velocity.

After substituting Formula (2) into Formula (11), the geological strength index value of soft rock under water-rich conditions with timeliness can be obtained, as shown in Formula (12), as follows:

$$\text{GSI}_t = 39.45e^{-3 \times 10^{-4}t} - .75, \quad (12)$$

where GSI_t is a geological strength index with timeliness.

By substituting time-dependent water-rich disturbance factor and time-dependent geological strength index into the parameter solving Formula (4), the parameters in the modified Hoek–Brown strength criterion can be obtained using Formula (13), as follows:

$$\begin{cases} m_b = m_i \exp \left(\frac{39.454e^{-3 \times 10^{-4}t} - 107.5}{14(1 + 1.014e^{-6.3 \times 10^{-4}t} - 1.014e^{-6.3 \times 10^{-4}t}D)} \right), \\ s = \exp \left(\frac{39.454e^{-3 \times 10^{-4}t} - 107.5}{3(2 + 1.014e^{-6.3 \times 10^{-4}t} - 1.014e^{-6.3 \times 10^{-4}t}D)} \right), \\ \alpha = 0.5 + \frac{1}{6} \left[\exp(-2.630e^{-3 \times 10^{-4}t} + 0.5) - \exp\left(\frac{-20}{3}\right) \right]. \end{cases} \quad (13)$$

Combined with the water-rich condition of the tunnel, the water-rich time t of the tunnel is reasonably selected and substituted into Formula (13). Each parameter value in the Hoek–Brown strength criterion under different water-rich conditions can be obtained and then used to calculate the displacement of the tunnel surrounding rock.

4. Deformation Analysis of Surrounding Rock

The problem of deep tunnel excavation can be simplified as the “thick wall cylinder” problem in elastic–plastic mechanics. The inner diameter of “cylinder” is the diameter of tunnel excavation, and the outer diameter can be ideally regarded as infinite. The deformation of the corresponding position of the tunnel surrounding rock can be obtained by calculating the displacement of the inner wall of the “thick wall cylinder.”

4.1. Fundamental Assumptions. Prior to the elastic–plastic analysis of the deformation of tunnel surrounding rock, the rock mass is assumed to be continuous, homogeneous, and isotropic. The lithology of the surrounding rock in each part of the tunnel is consistent. At the same time, assuming the surrounding rock is an ideal linear elastic body, the plastic zone strain of the tunnel conforms to the Hoek–Brown

strength criterion. The excavation mechanical model of the tunnel is shown in Figure 5.

As shown in Figure 5, the excavation radius of the tunnel is r_0 ; the radius of the plastic zone of the surrounding rock of the tunnel is R_0 , and the elastic zone of the surrounding rock of the tunnel is infinite. P_0 is the initial stress of the surrounding rock during tunnel excavation, and P_i is the supporting resistance of the supporting structure.

4.2. Surrounding Rock Stress Analysis

4.2.1. Stress Analysis in Elastic Zone. The stress component of any point in the elastic zone of the surrounding rock is only related to the distance to the centre of the tunnel. It is independent of the position of the point. The stress of any point in the elastic zone of surrounding rock is only related to the radius and independent of the angle. The stress equilibrium equation and geometric equation at any point in the elastic zone of surrounding rock are shown in Formula (14), as follows:

$$\begin{cases} \frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0, \\ \varepsilon_r = \frac{dx}{dr}, \\ \varepsilon_\theta = \frac{x}{r}, \end{cases} \quad (14)$$

where σ_r is the radial stress at any point in the elastic zone; σ_θ is the tangential stress at any point in the elastic zone; r is the distance from any point in the elastic zone to the centre of the tunnel; x is the radial displacement of any point in the elastic zone; ε_r is the radial strain at any point in the elastic zone; ε_θ is the tangential strain at any point in the elastic zone.

On the boundary of the elastic zone and plastic zone of the surrounding rock, the stress of the elastic zone is the same as that of the plastic zone. Therefore, the stress expression of any point in the elastic zone of the surrounding rock can be obtained, as shown in Formula (15), as follows:

$$\begin{cases} \sigma_r = P_0 \left(1 - \frac{r_0^2}{r^2} \right) + \sigma_R \frac{r_0^2}{r^2}, \\ \sigma_\theta = P_0 \left(1 + \frac{r_0^2}{r^2} \right) - \sigma_R \frac{r_0^2}{r^2}, \end{cases} \quad (15)$$

where P_0 is the initial ground stress; σ_R is the radial stress at the boundary of the elastic zone and plastic zone of the surrounding rock; r_0 is the excavation radius of the tunnel.

4.2.2. Stress Analysis in Plastic Zone. In polar coordinates, the expression of Hoek–Brown strength criterion is shown in Formula (16), as follows:

$$\sigma_\theta = \sigma_r + \sigma_c \left(m_b \frac{\sigma_r}{\sigma_c} + s \right)^\alpha. \quad (16)$$

Formula (16) is substituted into the first formula of Formula (14), and the formula is integrally calculated. Considering the boundary condition $r = r_0$, $\sigma_r = P_i$, the stress

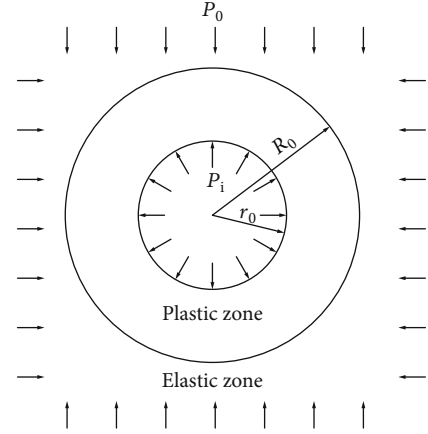


FIGURE 5: Mechanical model diagram of tunnel excavation.

expression of the plastic zone can be obtained, as shown in Formula (17), as follows:

$$\begin{cases} \sigma_r = \frac{\sigma_c}{m_b} \left[m_b(1-\alpha) \ln \left(\frac{r}{r_0} \right) + \left(m_b \frac{P_i}{\sigma_c} + s \right)^{1-\alpha} \right]^{1/1-\alpha} - \frac{\sigma_c}{m_b} s, \\ \sigma_\theta = \frac{\sigma_c}{m_b} \left[m_b(1-\alpha) \ln \left(\frac{r}{r_0} \right) + \left(m_b \frac{P_i}{\sigma_c} + s \right)^{1-\alpha} \right]^{1/1-\alpha} - \frac{\sigma_c}{m_b} s, \\ + \sigma_c \left[m_b(1-\alpha) \ln \left(\frac{r}{r_0} \right) + \left(m_b \frac{P_i}{\sigma_c} + s \right)^{1-\alpha} \right]^{\alpha/1-\alpha}, \end{cases} \quad (17)$$

where P_i is the supporting resistance of the supporting structure.

On the boundary of the elastic–plastic zone of the surrounding rock, where $r = R_0$, the stress in the elastic zone is the same as that in the plastic zone, where $\sigma_\theta^e = \sigma_\theta^p$, $\sigma_r^e = \sigma_r^p$. According to Formula (15), we can obtain the following:

$$\sigma_r^e + \sigma_\theta^e = 2P_0, \quad (18)$$

where σ_θ^e is the tangential stress in the elastic zone, σ_θ^p is the tangential stress in the plastic zone, σ_r^e is the radial stress in the elastic zone, σ_r^p is the radial stress in the plastic zone, and P_0 is the initial ground stress.

The radius of the plastic zone can be calculated by Formulas (17) and (18), as shown in Formula (19), as follows:

$$\begin{aligned} 2P_0 = & \frac{2\sigma_c}{m_b} \left[m_b(1-\alpha) \ln \left(\frac{r}{r_0} \right) + \left(m_b \frac{P_i}{\sigma_c} + s \right)^{1-\alpha} \right]^{1/1-\alpha} \\ & - \frac{2\sigma_c}{m_b} s + \sigma_c \left[m_b(1-\alpha) \ln \left(\frac{r}{r_0} \right) + \left(m_b \frac{P_i}{\sigma_c} + s \right)^{1-\alpha} \right]^{\alpha/1-\alpha}. \end{aligned} \quad (19)$$

Substituting the radius of the plastic zone into Formula (17), the radial stress of the elastic zone and plastic zone boundary of the surrounding rock can be calculated by

Formula (20), as follows:

$$\sigma_R = \frac{\sigma_c}{m_b} \left[m_b(1-\alpha) \ln \left(\frac{R_p}{r_0} \right) + \left(m_b \frac{P_i}{\sigma_c} + s \right)^{1-\alpha} \right]^{1/\alpha} - \frac{\sigma_c}{m_b} s. \quad (20)$$

4.3. Displacement Analysis of Surrounding Rock. The rock mass has an initial ground stress P_0 . Tunnel excavation causes the redistribution of surrounding rock stress, resulting in the stress increment, and then the displacement of tunnel surrounding rock.

When the surrounding rock is in the elastic zone, Formula (15) can deduce the stress increment of the surrounding rock by using Formula (21), as follows:

$$\begin{cases} \Delta\sigma_r = P_0 \left(1 - \frac{R_p^2}{r^2} \right) + \sigma_R \frac{R_p^2}{r^2} - P_0 = -\frac{R_p^2}{r^2} (P_0 - \sigma_R), \\ \Delta\sigma_\theta = P_0 \left(1 + \frac{R_p^2}{r^2} \right) - \sigma_R \frac{R_p^2}{r^2} - P_0 = \frac{R_p^2}{r^2} (P_0 - \sigma_R). \end{cases} \quad (21)$$

According to the stress–strain equation of the surrounding rock under polar coordinates, the stress–strain equation of surrounding rock under stress increment can be obtained using Formula (22), as follows:

$$\varepsilon_\theta = \frac{1-\mu^2}{E} \left(\Delta\sigma_\theta - \frac{\mu}{1-\mu} \Delta\sigma_r \right), \quad (22)$$

where μ is Poisson's ratio; E is the elastic modulus, ε_θ is the strain increment, $\Delta\sigma_\theta$ is the increment of tangential stress, and $\Delta\sigma_r$ is the increment of radial stress.

Substituting stress increment in Formula (21) into constitutive Formula (22) and combining geometric equation (14), the displacement of the elastic zone can be calculated using Formula (23), as follows:

$$x = \frac{1+\mu}{E} \frac{R_p^2}{r} (\sigma_R - P_0), \quad (23)$$

when $r = R_0$, the displacement of the elastic zone and plastic zone boundary of the surrounding rock can be calculated using Formula (24), as follows:

$$x_p = \frac{(1+\mu)(\sigma_R - P_0)}{E} R_p, \quad (24)$$

where x_p is the displacement of the boundary between the elastic zone and plastic zone, R_0 is the radius of the plastic zone; P_0 is the initial ground stress; σ_R is the radial stress at the boundary of the elastic zone and plastic zone; μ is Poisson's ratio; E is elastic modulus.

The plastic deformation of the tunnel surrounding rock mainly considers the shape change. Assuming the tunnel surrounding rock is an incompressible material, the deforma-

tion is shown in Formula (25), as follows:

$$\varepsilon_v = \varepsilon_r + \varepsilon_\theta + \varepsilon_z = 0. \quad (25)$$

In Formula (25), ε_v is the volumetric strain, ε_r is the radial strain, ε_θ is the tangential strain, and ε_z is the axial strain.

In elastic–plastic mechanics, “thick-walled cylinder” is a plane strain problem, considering $\varepsilon_z = 0$. According to the continuity of tunnel surrounding rock deformation, the plastic zone displacement of the tunnel surrounding rock can be calculated using Formula (26), as follows:

$$x = \frac{1+\mu}{E} (\sigma_R - P_0) \frac{R_p^2}{r}. \quad (26)$$

In the plastic zone and the tunnel excavation boundary, where $r = r_0$, combined with Formula (23) and considering Formula (17), the surrounding displacement of the tunnel surrounding rock can be obtained using Formula (27), as follows:

$$x = \frac{1+\mu}{E} \frac{R_p^2}{r_0} \left\{ \frac{\sigma_c}{m_b} \left[m_b(1-\alpha) \ln \left(\frac{R_p}{r_0} \right) + \left(m_b \frac{P_i}{\sigma_c} + s \right)^{1-\alpha} \right]^{1/\alpha} - \frac{\sigma_c}{m_b} s - P_0 \right\}. \quad (27)$$

The displacement of the surrounding rock of the water-rich soft rock tunnel can be obtained through the displacement formula of the tunnel surrounding rock and the solution formula of each parameter.

5. Engineering Application

5.1. Engineering Situation. The newly-built Xiejiapo tunnel is a typical water-rich soft rock tunnel in Ankang–Langao Expressway, located in Hanbin District, Ankang City, Shaanxi Province. The tunnel is a separate one-way two-lane tunnel. The entrance pile number of the left line of the tunnel is ZK 15 + 200, and the exit pile number is ZK18 + 090, which is 2870 m long. The entrance pile number of the right line of the tunnel is K15 + 210, the exit pile number is K18 + 090, and the length is 2880 m. The maximum buried depth of the tunnel is approximately 234.0 m. The lithology of the entire strata in the tunnel site area is relatively complex, mainly phyllite of Meiziya Formation of Lower Silurian, which is grey and Brown. The surrounding rock of the tunnel is a thin sheet structure with silk lustre, and the strength is low in wet state. It is easily softened in water. The surrounding rock mass in the tunnel is broken, and the joint fissure is developed. It has poor ability to resist weathering, poor self-stability, and easy to collapse. The surrounding rock grade is V.

Groundwater is developed in the tunnel site area; it is mainly composed of pore fissure water of quaternary loose rock and basic fissure water. The surrounding rock is under water-rich conditions for a long time. The tunnel has a net width of 11.77 m and a net height of 8.80 m. The internal radius of the arch wall is 6.05 m, and the internal radius is

17.0 m in the inverted arch. The tunnel is in the Qinling fold tectonic belt, and the Dabashan fault zone passes through it. Tunnel construction is prone to collapse and water inrush events.

5.2. Theoretical Analysis of the Displacement of Surrounding Rock of Xiejiapo. The water-rich section ZK17 + 730 – ZK 17 + 760 and the natural state section ZK17 + 500 – ZK17 + 530 of Xiejiapo tunnel are selected as the research objects. In the water-rich section, the surrounding rock is always in the state of immersion, and it can be considered that the surrounding rock is in the state of immersion for 300 days. The surrounding rock is assumed to be immersed for 300 days, and in the natural state, the surrounding rock is assumed to be immersed for 0 days. The displacement of the surrounding rock is calculated considering the actual construction situation.

5.2.1. Determining the Parameters. The surrounding rock of the tunnel is mainly carbonaceous phyllite. According to the lithology of carbonaceous phyllite observed on site, $m_i = 7$. According to the blasting evaluation results of field tunnel engineering, the blasting disturbance factor $D = 0.45$. According to the tunnel equivalent radius value method, $r_0 = 9.60$. At the same time, different t values are substituted into Formula (13) to obtain the parameters of Hoek–Brown strength criterion, as shown in Tables 3 and 4.

5.2.2. Initial Ground Stress. The initial vertical ground stress of the tunnel only considers the self-weight of the surrounding rock in Formula (28), as follows:

$$P_0 = \gamma h. \tag{28}$$

In Formula (28), P_0 is the initial ground stress, γ is the bulk density, and h is the buried depth of the tunnel. The average buried depth of the water-rich section and natural state section of soft rock tunnel is 130 m. Thus, the initial stress of the tunnel can be calculated, $P_0 = 3.63\text{MPa}$.

5.2.3. Support Resistance. According to the provisions of “Specification for design of highway tunnels” (JTG 3370.1-2018), the formula for calculating the vertical surrounding rock pressure at the arch of deep-buried tunnels is shown in Formula (29), as follows:

$$q = 0.45 \times 2^{s-1} \times \gamma \times [1 + i(B - 5)], \tag{29}$$

where q is the surrounding rock pressure of the tunnel arch, and the unit is kN/m^2 ; s is the level of surrounding rock; B is the tunnel width, the unit is m; i is related to tunnel width B , $i = 0.1$. Support resistance of the initial support in vertical direction can be calculated, $P_i = 256.20\text{kPa}$.

5.2.4. Surrounding Rock Deformation. According to the vertical initial stress and vertical support resistance, the parameters in Tables 2 and 3 are substituted into the calculation formula of surrounding rock displacement (20), and the range of plastic zone of the surrounding rock can be calculated. Then, the parameters are substituted into Formula

TABLE 3: Parameters of the generalized Hoek-Brown strength criterion.

t	D	GSI	m_b	s	α	σ_c
0	0.4500	31.815	0.30232	0.000135	0.51977	23.314
360	0.4500	31.815	0.30232	0.000135	0.51977	7.917

TABLE 4: Parameters of the modified Hoek-Brown strength criterion.

t	D_m	GSI_t	m_b	s	α	σ_c
0	0.4500	31.815	0.30223	0.000135	0.51977	23.314
360	0.5616	27.790	0.19400	0.000052	0.52592	7.917

TABLE 5: Calculation results of tunnel vault surrounding rock deformation.

Immersion time (d)	The displacement using generalized Hoek-Brown strength criterion (mm)	Displacement using Hoek-Brown strength criterion considering water-rich disturbance factor (mm)
0	43.51	43.51
360	44.33	54.53

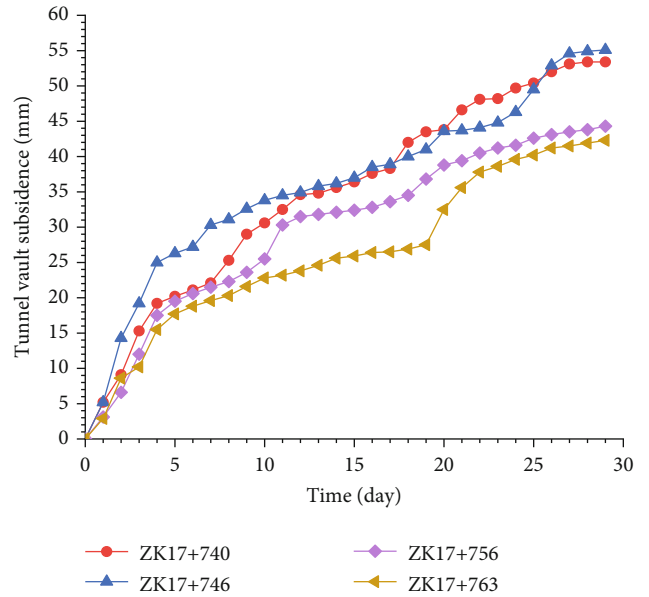


FIGURE 6: Temporal curve of vault subsidence monitoring section.

(27), and the vault subsidence of the tunnel surrounding rock can be calculated. The calculation results are shown in Table 5.

Table 4 shows that when the generalized Hoek–Brown strength criterion is used to calculate the vault settlement of the tunnel, the calculation results of the natural section and the water-rich section are similar. However, when the modified Hoek–Brown strength criterion is used, the vault settlement of the water-rich section is evidently larger than that of the natural section. When calculating the displacement

TABLE 6: Displacement of surrounding rock under different strength criterion.

Section number	Section	The deformation value of tunnel vault (mm)	The displacement using generalized Hoek-Brown strength criterion		Displacement using modified Hoek-Brown strength criterion	
			Theoretical value (mm)	Relative error (%)	Theoretical value (mm)	Relative error (%)
1	ZK17 + 740	53.6 mm	44.33 mm	-20.91%	54.53 mm	1.71%
2	ZK17 + 746	55.1 mm	44.33 mm	-24.30%	54.53 mm	-1.05%
3	ZK17 + 756	44.3 mm	43.51 mm	-1.81%	43.51 mm	-1.81%
4	ZK17 + 763	42.3 mm	43.51 mm	2.78%	43.51 mm	2.78%

of the tunnel vault in the natural state, the calculation results of the modified Hoek–Brown strength criterion are the same as those of the generalized Hoek–Brown strength criterion. This is mainly because the modified Hoek-Brown strength criterion in natural state can be transformed into generalized Hoek-Brown strength criterion, so the displacement calculation results are the same. When calculating the displacement of tunnel vault in the water-rich section, the modified Hoek-Brown strength criterion can comprehensively consider the softening effect of water on rock mass, so the calculated displacement value of the modified Hoek-Brown strength criterion is significantly larger than that of the generalized Hoek-Brown strength criterion.

5.3. Field Displacement of the Surrounding Rock of the Xiejiaopo Tunnel. According to the requirements of the “Technical specification for monitoring and measuring highway tunnels” (DB13/T 2177-2015), combined with the construction of the Xiejiaopo tunnel, ZK17 + 740 and ZK17 + 746 are selected in the water-rich section ZK17 + 730 – ZK17 + 760, and ZK17 + 515 and ZK17 + 521 are selected in the natural state section ZK17 + 500 – ZK17 530. The tunnel vault subsidence of the four sections is monitored and measured, and the monitoring measurement results are drawn into the vault subsidence temporal curve, as shown in Figure 6.

The figure shows that the vault subsidence values of the four sections are 53.6, 55.1, 44.3, and 42.3 mm. The deformation of surrounding rock in the natural section is obviously smaller than that in the water-rich section, and the stability time of surrounding rock in the natural section is obviously earlier than that in the water-rich section.

5.4. Comparative Analysis. The field monitoring measurement results are compared with the theoretical calculation results, and the error is shown in Table 6.

It can be seen from Table 6 that in the natural state, the displacement of surrounding rock calculated by the generalized Hoek-Brown strength criterion is close to the measured value, and the calculation error is small, so it has good applicability. As the generalized Hoek-Brown strength criterion fails to consider the softening effect of rock mass in the water-rich section of the tunnel, the calculation error of displacement is large when calculating the settlement of the vault. However, the modified Hoek-Brown strength criterion can better consider the disturbance of rich water on lithology

and can accurately calculate the displacement of surrounding rock. Therefore, the displacement calculation is closer to the field test results, and the error is small. This shows that the modified Hoek-Brown strength criterion is more suitable for water-rich soft rock tunnel, and the analysis of surrounding rock stability in water-rich soft rock tunnel is more accurate.

6. Conclusions

Through the immersion softening test, the softening law of the mechanical properties of soft rock under the groundwater seepage influence is obtained, and on this basis, the water-rich influencing factor is established. Combined with the disturbance characteristics when the water-rich soft rock tunnel excavated, the water-rich disturbance factor is introduced to modify the generalized Hoek-Brown strength criterion. Using the “thick wall cylinder” problem in elastic-plastic mechanics theory, the calculation formula of tunnel surrounding rock displacement is obtained. By comparing the monitoring value and theoretical calculation value of surrounding rock displacement in practical engineering, the applicability of the modified Hoek-Brown strength criterion is verified. The main conclusions are as follows:

- (1) Through the immersion softening test of soft rock, it was found that the Poisson’s ratio of soft rock did not change significantly with the extension of immersion time. The uniaxial compressive strength, elastic modulus, and P-wave velocity of soft rock decreased rapidly at the beginning of immersion and then gradually stabilized. On the contrary, the porosity of soft rock changes little at the beginning of immersion and then increases rapidly. On this basis, the variation formula of physical and mechanical indexes of soft rock with immersion time is fitted
- (2) According to the damage of elastic modulus of soft rock after immersion, the concept of water-rich influence factor is proposed. Combined with the disturbance characteristics of water-rich soft rock tunnel excavation, the water-rich disturbance factor considering both blasting disturbance and groundwater softening is established. Based on this, the generalized Hoek-Brown strength criterion is modified

- (3) In the water-rich section of Xiejiapo tunnel, the relative error between the vault subsidence displacement obtained by the modified Hoek-Brown strength criterion and the field monitoring measurement is small, so the modified Hoek-Brown strength criterion can be applied to the displacement calculation and the stability analysis of water-rich soft rock tunnel

Data Availability

The raw/processed data required to reproduce these findings cannot be shared at this time as the data also form part of an ongoing study.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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