Theoretical Study of Surrounding Rock Loose Zone Scope Based on Stress Transfer and Work–Energy Relationship Theory

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Abstract: In view of the actual gap between the theoretical solution and the measured value of the range of the surrounding rock loose zone excavated by blasting, the formation process and influencing factors of the surrounding rock loose zone after blasting are analyzed by combining theoretical analysis, numerical simulation, and field tests. It is proposed that the formation of the surrounding rock loose zone can be divided into two parts: the stress loose zone formed by in situ stress release after chamber excavation and the blasting–expansion loose zone formed by blasting, corresponding to the stress loose zone crack propagation. In this paper, the stress method and work–energy relationship method based on the Fenner solution were used to study the range of the surrounding rock loose zone formed after chamber excavation. A theoretical formula for determining the range of the surrounding rock loose zone is presented. These methods can clearly explain the formation mechanism of the loose zone, and the parameter selection method is simple and fast, which can be used for the theoretical prediction of the loose zone of surrounding rock in tunnel engineering in rock areas. Finally, the two methods were used to calculate the range of surrounding rock loose zone under three different working conditions in the Qingdao Metro single-track section, and the results were compared with the field monitoring results to verify the applicability of the proposed methods.

Keywords: stress method; work–energy relationship method; scope of loose zone; theoretical calculation

1. Introduction

The process of chamber excavation causes disturbance to the surrounding rock mass, which disturbs the stress state of the original rock and causes the stress in the surrounding rock to be redistributed. In this process, the maximum principal stress of the in situ stress component in the surrounding rock will exceed the stress intensity of the original rock. At this time, some surrounding rock will change from an elastic state to a plastic state. The damaged part of the surrounding rock is distributed around the chamber to form an annular broken belt area, which is called the loose zone in existing theory [1].

According to previous research and engineering experience, in the process of tunnel (chamber) construction, the loose zone exists in the surrounding rock. Many scholars have also conducted considerable research on the formation mechanism and determination of the ranges of loose zones. For the determination of the scope of a loose zone, the existing research methods include field measurements, the acoustic method [2–4], the multi-point displacement method [5], and the geological radar method [6] to test the loose zone on-site. Through numerical simulations [7–10], the loose zone ranges under different working conditions have been simulated, and their influencing factors (surrounding rock damage coefficient, surrounding rock grade, blasting dynamic load, blasting cycle times)
and variation characteristics have been discussed. The formation mechanism and scope of the loose zone are studied and discussed through indoor model tests [11,12].

Most of the formulas for the loose zone are based on either the elastic–plastic theory or field-measured data. Dong et al. [13,14] proposed a support theory of surrounding rock loose zone and presented an empirical formula of the loose zone range, which was related to the original rock stress and surrounding rock strength in combination with practical engineering:

\[ L = a f (P, R) + b \]  

or

\[ L = 2.22B^{1.94}H^{4.30}R^{-1.05}10^{-11} \]  

where \( P \) is the initial in situ stress, \( R \) is the compressive strength of the rock, \( B \) is the tunnel span, \( H \) is the buried depth of the tunnel, and \( a, b \) are correction factors.

Zhou et al. [15] used grey system theory to classify the loose zone of surrounding rock and carried out grey prediction research on the loose zone under different rock strengths based on field measurements. Zhou et al. [16] established a mathematical model of the thickness of the surrounding rock loose zone, the unified compressive strength of the rock mass, and the original rock pressure. They fitted a mathematical model with actual measured engineering data and obtained a prediction expression of the loose zone range:

\[ L_p = a_1 \frac{h}{2} \pi^\frac{p}{R} + a_2 R \left( \frac{1}{2} \right) + a_3 \left( \frac{1}{2} \right)^2 \]  

where \( l \) is the tunnel span, \( h \) is the tunnel height, and \( a_1, a_2, \) and \( a_3 \) are correcting factors.

Zhao [17] introduced the theory of dimensional analysis, selected the parameters related to the loose zone, and established a prediction model of the loose zone range. Combined with measured data, a quantitative function was derived. Gao et al. [18], Xu et al. [19], Ma [20], and Zhu et al. [21] trained mathematical models by using an artificial neural network combined with field observation data, and they obtained the relationship between the loose zone range and its influencing factors, which could be used to predict the loose zone range. Li et al. [22] used the bilinear damage model and the principle of stress equivalence to deduce the analytical solution of the damage theory of the surrounding rock loose zone and proposed a standard definition:

\[ \rho_k = R_0 \left\{ \frac{\sin \varphi}{0.2c \cos \varphi} \left\{ c_0 + \left( 2 + \frac{\lambda}{E} \right) \frac{c \cos \varphi}{\sin \varphi} \right\} \left( \frac{1.8E + \lambda}{E + \lambda} \right) \frac{\sin \varphi}{\sin \varphi - 1} - \left( 2 - \frac{1.8 \sin \varphi}{1 - \sin \varphi} + \frac{\lambda}{E} \right) \frac{c \cos \varphi}{\sin \varphi} + 1 \right\}^{\frac{1 - \sin \varphi}{\sin \varphi}} \]  

where \( R_0 \) is the tunnel radius, \( c \) is the cohesion of the rock and soil mass, \( \varphi \) is the internal friction angle, \( \lambda \) is the damage modulus, \( E \) is the elastic modulus, and \( c_0 \) is in situ stress.

Chen et al. [23] calculated the range of the plastic softening zone after roadway excavation by using methods from elasticity and elastodynamics, and they verified the applicability of this approach by comparing it with the field-measured data. Huang et al. [24] deduced an expression of the loose zone range of surrounding rock based on the ideal elastic–plastic model of the Drucker–Prager criterion and compared it with field tests. Su et al. [25] and Wang et al. [26,27] deduced a theoretical expression of the loose zone range based on unified strength theory and compared it with the calculated value of the theoretical formula of the loose zone range based on the Mohr–Coulomb (M–C) criterion and Hoek–Brown criterion. It was found that the errors between the loose zone radius calculated based on unified strength theory and the measured values were small. The unified strength theory method is as shown in Equation (5):

\[ r = r_0 \left[ \frac{(B + c \cot \varphi_i)}{(T + c \cot \varphi_i)} \right]^{\frac{1 - \sin \varphi_i}{\sin \varphi_i}} \]  

\[ \sin \varphi_i = \frac{2(1 + b) \sin \varphi}{2 + b(1 + \sin \varphi)} \]
where $\alpha$ and $B$ are the tension-compression strength ratio and tension-shear strength ratio of rock and soil, respectively; $b$ is the influence coefficient of intermediate principal stress; $P_0$ is the original rock stress; $P_i$ is the support reaction; and $c_t$ and $q_t$ are the cohesion and internal friction angle corresponding to the tensile strength.

Jiang et al. [28] used the M–C yield criterion and the stress-strain continuity condition for non-correlation analysis, obtained the radius of the fracture zone (loose zone) through the stress-strain condition, and verified the theoretical formula through the analysis of an example. Based on the Hoek–Brown criterion, Li et al. [29] deduced a formula for calculating the loose zone radius by using equilibrium and continuity conditions and compared its predictions with the field test data. The formula for the loose zone radius was as follows:

$$R_0 = ae^{x - \frac{2b \sqrt{t}}{n}}$$

(9)

$$x = \frac{-m c_2 + \sqrt{m^2 c_2^2 + 4c_2(4c_2 + 4m P_0)}}{2mc_2}$$

(10)

where $m$ and $s$ are dimensionless coefficients related to lithology and integrity, respectively, and $c_t$ is the uniaxial compressive strength.

Based on the Hoek–Brown criterion, Pan et al. [30] analyzed the ideal elastic-plastic surrounding rock with a lateral pressure coefficient of 1 through a limiting equilibrium equation, and they deduced the formulas for calculating the radius, displacement, and stress of the plastic zone and fracture zone under the shear expansion criterion and unified strength criterion under low ground stresses. Research showed that the results of this method were closer to the numerical simulation results than the theoretical formula based on the M–C criterion and unified strength criterion under low ground stresses.

Based on the Drucker–Prager yield criterion, Chen et al. [33] deduced a closed analytical solution of the ranges of the plastic zone and fracture zone under the shear expansion of rock mass, and they compared and analyzed the influences of different yield criteria on the change of the surrounding rock conditions in combination with practical engineering. The research showed that the intermediate principal stress had a significant effect on the influence range of the fracture zone.

Through the analysis of previous studies, it was found that most theoretical analytical solutions have complex forms, include many parameters, and lack methods for determining parameter values, as shown in Table 1. In this study, the Fenner solution (modified Fenner formula or Castelner formula) with a concise form and calculation parameters related to
the actual construction project were selected to calculate the loose zone range during the construction of a single-track tunnel of the Qingdao Metro Line 1. The formula is as follows:

\[ R_p = r \left[ \frac{L}{\rho_{1-\sin \varphi}} \right]^{1-\sin \varphi} \]  

(15)

### Table 1. Summary of the scope of the existing surrounding rock loose zone.

<table>
<thead>
<tr>
<th>Method Type</th>
<th>Source</th>
<th>Theoretical Calculation Method</th>
<th>Shortcoming</th>
</tr>
</thead>
<tbody>
<tr>
<td>Empirical formula</td>
<td>Dong [14]</td>
<td>[ L = 2.222 \sqrt{0.84H3.30R^{-1.05}}10^{-11} ]</td>
<td>Poor applicability</td>
</tr>
<tr>
<td></td>
<td>Zou [16]</td>
<td>[ L_p = a_1 \sqrt{R_2} + a_2 R \left( \frac{L}{R_2} \right)^2 ]</td>
<td></td>
</tr>
<tr>
<td>Damage theory</td>
<td>Li [22]</td>
<td>[ \rho_k = R_0 \left{ \frac{\sin \varphi}{0.2 + \cos \varphi} \left[ \left( \frac{\rho_{1-\sin \varphi}}{0.2 + \cos \varphi} \right)^{1-\sin \varphi} - \left( 2-1.8 \sin \varphi + \frac{1}{2} \right) \right] \right} ]</td>
<td>Complex form and poor physical meaning</td>
</tr>
<tr>
<td></td>
<td>Su [26], Wang [27]</td>
<td>[ r = f_0 \left[ \left( \frac{\rho_{1-\sin \varphi}}{0.2 + \cos \varphi} \right)^{1-\sin \varphi} \right] ]</td>
<td>Complex parameter values</td>
</tr>
<tr>
<td>Strength theory</td>
<td>Zhang [32]</td>
<td>[ R_p = r_4 \left{ \frac{\tan \varphi - \sin \varphi \tan \varphi \rho_{1-\sin \varphi}}{\sin \varphi \sin \varphi} \right} ]</td>
<td>Ambiguous parameter value</td>
</tr>
<tr>
<td></td>
<td>Chen [33]</td>
<td>[ R_p = a \left( \frac{D_1(1+\varphi)}{D_2(1+\varphi)} \right)^{1-\sin \varphi} \left[ \frac{(\rho_{1-\sin \varphi})^{1-\sin \varphi}}{\rho_{1-\sin \varphi}} \right] ]</td>
<td>Complex form and Ambiguous parameter value</td>
</tr>
</tbody>
</table>

The calculated values are quite different from the actual values, with the actual values being 6–10 times greater. For example, in the working conditions of grade-IV surrounding rock, the thickness of the loose zone through the Fenner solution is 0.106 m, while the field measured value is 1.2–1.4 m; that is, the Fenner solution has poor applicability in the calculation of the loose zone of Qingdao Metro. When the numerical simulation method is used to calculate the range of the loose zone after chamber excavation, the thickness of the loose zone is 0.235 m.

Through comparison, it was found that the calculated results of the Fenner solution were similar to those obtained by numerical simulations. In view of this, some conjectures and assumptions were made, and the scope of the loose zone was studied. The existing loose zone theory lacks accuracy and applicability for practical engineering applications. In order to make the theoretical understanding of the loose zone more suitable for engineering practice, some exploratory research was carried out in this study. In this paper, from the perspective of the stress and work–energy relationship, a theoretical analysis was performed on the formation of the surrounding rock loose zone after chamber excavation. The loose zone was regarded as a stress loose zone formed under the action of in situ stress and a blasting–expansion loose zone formed by the blasting–expansion stress. In this study, using the method of theoretical analysis, from the perspective of stress and the work–energy relationship, the formation of the surrounding rock loose zone after tunnel excavation was regarded as the stress loose zone formed under the action of in situ stress and the blasting–expansion loose zone formed by the blasting–expansion stress loose zone. The theoretical calculation formula was derived, and the coupling effects of in situ stress and blasting load on the surrounding rock loose zone were considered. The coupling coefficient was introduced to modify the formula. Finally, the derivation of the theoretical calculation formula for the scope of the loose zone of the surrounding rock of the tunnel excavation was formed, and the applicability of the theoretical formula was verified by setting three working conditions against the background of the Qingdao Metro single track interval tunnel line 1 project, which provided a theoretical reference for further research in the future.

2. Theoretical Analysis and Definition of Surrounding Rock Loose Zone

After the rock mass in the center of a tunnel (chamber) is excavated, the initial stress state of the stratum is disturbed, and the original three-dimensional stress state changes to a two-dimensional stress state. This leads to stress redistribution and damage to the
surrounding rock in the process. After the surrounding rock is damaged, the surrounding rock stress decreases along the radial depth of the tunnel until it becomes the initial stress. Based on the damage degree, the surrounding rock is divided from inside to outside into the loose zone, plastic zone, elastic zone, and original rock stress zone, as shown in Figure 1.

During the excavation of the chamber, the formation of the loose zone is regarded as occurring in two parts. The first part is the stress release caused by the unloading and excavation of the chamber. Because the concentrated stress of some surrounding rock exceeds the strength of the surrounding rock, there will be a broken area along the radius of the chamber. The loose zone formed at this time is called the stress loose zone. In addition, the explosion produces a shock wave. When the shock wave enters the surrounding rock, it will attenuate into a stress wave for transmission. In the transmission process, it will produce a stress field in the surrounding rock. When the stress acts on the fracture zone of the loose zone and penetrates the crack tip, it will produce stress concentration at the crack tip. The singularity of the stress field at the crack tip leads to large failure stress caused by a small external load, which makes the crack in the loose zone of the surrounding rock expand and extend. At the same time, new cracks are generated in the original stress loose zone, and finally, a wider range of the loose zone is formed after expansion. At this time, the newly formed annular loose zone is called the blasting loose zone. Here, the final loose zone of the surrounding rock is regarded as consisting of two parts: a stress loose zone and a blasting–expansion loose zone.

Here, the loose zone is defined as the boundary range of the annular rock damage area around the chamber, which is formed by the damage of the rock mass under the combined action of the in situ stress and the blasting load after the chamber is excavated. To determine the boundary range of the damage area, combined with the technical code for the construction of the rock foundation excavation engineering of hydraulic structures, the range in which the ultrasonic wave velocity decreases by 10–20% in the surrounding rock tested in an actual project is defined as the boundary range of the damage area [34]. It can also be considered that the stress of the maximum principal stress component of the surrounding rock reaches its dynamic tensile strength or the surrounding rock strain exceeds the maximum tensile strain of the surrounding rock, which is considered to meet the criteria for judging damage. According to this definition, the formation diagram of the loose zone in chamber excavation is depicted, as shown in Figure 2.

![Figure 1. Surrounding rock zones of chamber excavation.](image-url)
Figure 2. Formation of a loose zone in chamber excavation: (a) Initial stress state of the unexcavated chamber; (b) initial loose zone under in situ stress; (c) final loose zone after blasting expansion. $P_0$ and $\lambda P_0$ are original rock stress, $P$ is in situ stress, $\sigma$ is blasting stress, $S_1$ is the in situ stress loose zone, and $S_2$ is the blasting loose zone.

3. Derivation of the Theoretical Formula of the Surrounding Rock Loose Zone

Based on the above definition and formation mechanism analysis of the surrounding rock loose zone, the theoretical solution method of the surrounding rock loose zone range can be discussed from two aspects: the stress method based on stress transfer and the work–energy relationship method based on work and energy conversion.

3.1. Theoretical Derivation of the Stress Method

In the process of chamber excavation, the crushing effect of blasting on rock is regarded as the coupling effect of the shock wave and the explosive gas, as shown in Figure 3. The crushing action on rock can be mainly divided into the crushing action on the rock mass of the tunnel face and the crushing action on the surrounding rock [35].
A shock wave is generated at the moment of the explosion, and the explosion expands along the tiny cracks in the tunnel face and breaks the rock mass. The high temperature generated by the explosion generates an explosive gas, which throws the broken rock mass on the tunnel face and peels back the original rock mass to form a chamber.

At this time, the stress state around the chamber changes, and the in situ stress in the surrounding rock is redistributed. In this process, part of the surrounding rock will be broken to form a stress-loosening circle. When the blasting shock wave is transmitted to the edge of the surrounding rock, it attenuates into a stress wave, expands the cracks in the surrounding rock, and forms a new blasting–expansion loose zone. The scope of the loosening force corresponding to the blasting circle and the scope of the loosening force corresponding to the blasting circle are deduced below.

3.1.1. Derivation of Stress Relaxation Range

In order to analyze the basic theory of the loose zone, the following basic assumptions are made. For a tunnel with a circular cross-section, the surrounding rock is infinite, uniform, continuous, and isotropic. The initial stress field of the surrounding rock (without gravity) is uniform. The tunnel size and surrounding rock stress do not change along the tunnel length direction, and the strain and displacement along the tunnel length direction are zero; that is, the tunnel length is infinite, and the surrounding rock of the tunnel cross-section is in a state of plane stress. The excavation and support of the tunnel are completed instantaneously, and the tunnel support is close to the surrounding rock.

Based on Fenner’s solution and the Mohr–Coulomb yield criterion, the expression for the plastic zone of the surrounding rock can be deduced through a geometric equation, equilibrium differential equation, and continuity condition, which is called the modified Fenner formula or the Castelner formula. The radius of the plastic zone is calculated as follows [36]:

$$ R_p = r_1 \left[ \frac{(P_0 + c \cot \varphi)(1 - \sin \varphi)}{H_1 + c \cot \varphi} \right]^{1 - \sin \varphi / \sin \varphi} $$

(16)

where \( P_0 \) is the initial in situ stress; \( P_1 \) is the support reaction; \( c \) and \( \varphi \) are the cohesion and internal friction angle, respectively; \( r_1 \) is the equivalent circle radius of the chamber section, which can be calculated by \( r_1 = (H + B)/4 \); and \( H \) and \( B \) are the height and width of the chamber section, respectively.

Two parameters related to the grade of surrounding rock are introduced to improve the formula of the plastic zone radius, which makes the formula more concise and suitable for engineering calculations. The parameters are \( \eta_{uc} = 2c \cdot \cos \varphi / (1 - \sin \varphi) \), which is the uniaxial compressive strength of the surrounding rock, and \( K_\varphi = (1 - \sin \varphi) / 2 \sin \varphi \), which is the internal friction angle coefficient of the surrounding rock. In addition, \( P_0 \) is...
the initial in situ stress, $P_i$ is the support reaction, and $c$ and $\varphi$ are cohesion and internal friction angle, respectively, of rock and soil mass. The equation for the radius of the plastic zone becomes:

$$R_p = r_i \left[ \frac{p_i + K_c q_c}{T + K_c q_c} \left( \frac{2K_c}{K_p + T} \right)^{K_p} \right]$$

(17)

Finally, according to previous research [37], the tangential stress at the outer boundary of the surrounding rock stress loose zone surrounded by the plastic zone is considered to be equal to the original rock stress, so the expression of the loose zone can be obtained as follows:

$$R_{b1} = r_i \left[ \frac{p_i + K_c q_c}{T + K_c q_c} \left( \frac{K_p}{K_p + T} \right)^{K_p} \right]$$

(18)

The thickness of the stress loose zone $L_{b1}$ is:

$$L_{b1} = R_{b1} - r_i = r_i \left[ \frac{p_i + K_c q_c}{T + K_c q_c} \left( \frac{K_p}{K_p + T} \right)^{K_p} \right] - r_i$$

(19)

3.1.2. Derivation of Range of Blasting–Expansion Loose Zone

In the process of crack propagation in the stress loose zone caused by the explosion, the open crack is the main form of crack propagation, and the analysis of the range of the blasting loose zone caused by the analysis of the crack propagation in the stress loose zone is mainly based on the analysis of the blasting stress. The stress wave generated by blasting can be divided into a vertical stress wave and a parallel stress wave [38–40], as shown in Figure 4.

![Figure 4](image_url)

**Figure 4.** Effect of blasting incident stress wave: (a) Stress of chamber section; (b) incident stress wave front. $F_1$ is the chamber section, $F_2$ is the incident stress wave front, $\sigma_{\perp}$ is the incident stress, $\sigma_{\parallel}$ is the vertical component of incident stress, and $\sigma_{\parallel}$ is the parallel component of incident stress.

The incident stress wave of blasting is mainly related to the nature of the explosive and charging mode. After the blasting in the blast hole, the shock wave is aroused in the rock near the blast hole. The initial impact pressure of the hole wall is approximately calculated based on acoustic approximation theory [41], as follows:

$$p_\lambda = \frac{\rho_0 D_1^2}{4} \times \frac{2\rho_m D_0}{D_0 \rho_m + D_1 \rho_0} = \frac{\rho_m D_0 \rho_0 D_1^2}{2(D_0 \rho_m + D_1 \rho_0)}$$

(20)

As the propagation distance of the stress wave increases, the peak value of the stress decreases continuously. At the same time, in the action area of the stress wave, the radial and tangential stress of the cylindrical stress wave in the rock have the following relationship:

$$\sigma_\theta = \frac{K_d}{1 + K_d} \sigma_r$$

(21)

If the rock body is regarded as a uniform elastic body, the stress at any point in the rock where the wave front goes can be expressed as follows:

$$\sigma_r = p_\lambda r^{-\alpha}$$

(22)
where \( \sigma_{\theta} \) and \( \sigma_{r} \) are the tangential stress and radial stress of the rock, \( r = r_{OA}/r_{0} \) is the contrast distance, \( P_{\lambda} \) is the initial impact pressure of the hole wall, \( \alpha = 2 - \mu_{d}/(1 - \mu_{d}) \) is the pressure attenuation index, and \( \mu_{d} \) is the dynamic Poisson’s ratio of the rock, which is generally taken as \( \mu_{d} = 0.8\mu \).

The incident stress of blasting can be calculated using the above formulas. The specific expression is as follows:

\[
\sigma_{\lambda} = \sigma_{r} = \rho_{m}D_{0}\rho_{0}D_{1}^{2}(1 + \rho_{0}/\rho_{m}) \cdot \left( \frac{r_{0}}{r_{OA}} \right)^{2} \cdot \frac{\mu_{d}}{1 - \mu_{d}}
\]

where \( \rho_{m} \) is the density of the rock medium, \( D_{1} \) is the wave velocity of the shock wave (detonation wave velocity) in the rock at the hole wall, \( \rho_{0} \) is the density of the explosive, \( D_{0} \) is the velocity of the elastic wave propagating in the explosion (initial velocity of the shock wave), \( r_{0} \) is the distance in the direction of the cylindrical charge (blast hole radius), \( r_{OA} \) is the distance in the incident direction of the stress wave, \( r_{OA} = \sqrt{L_{1}^{2} + L_{2}^{2}} \), \( L_{1} \) is the horizontal distance between the center of the cylindrical charge and the top of the roadway, \( L_{2} \) is the vertical distance between the center of the cylindrical charge and the outer contour of the loose zone at the top of the roadway, and \( \mu_{d} \) is the dynamic Poisson’s ratio of the rock, which is generally taken as \( \mu_{d} = 0.8\mu \).

Considering the influence of porous blasting on the crack propagation and extension as well as the influence of porous blasting on the incident stress, the incident stress is calculated by the following formula:

\[
\sigma_{\lambda} = (n)^{C}\sigma_{\lambda}
\]

where \( n \) is the number of blast holes and \( C \) is the influence coefficient.

What causes crack propagation in surrounding rock is the derived tensile stress of the vertical component of the incident stress. The relationship between the vertical component and the incident stress is as follows:

\[
\sigma_{\perp} = \sigma_{\lambda} \cos \alpha
\]

where \( \alpha \) is the angle of oblique incidence of the blasting stress wave, and the size is determined by the following formula:

\[
\cos \alpha = \frac{L_{1}}{\sqrt{L_{1}^{2} + L_{2}^{2}}}
\]

From the vertical component of the incident stress \( \sigma_{\perp} \), the tensile stress \( \sigma_{\theta\perp} \) is derived as follows:

\[
\sigma_{\theta\perp} = \frac{\mu_{d}}{1 - \mu_{d}}\sigma_{\perp}
\]

The derived tensile stress \( \sigma_{\theta\perp} \) makes the crack in the roadway sidewall expand.

According to the theory of rock fracture mechanics, for an open crack, when the crack is stable, under the action of the blasting stress wave, the stress field at any point near the top of the crack \( (r, \theta) \) is [42]:

\[
\begin{bmatrix}
\sigma_{x} \\
\sigma_{y} \\
\sigma_{z}
\end{bmatrix} = \frac{K_{1}}{\sqrt{2\pi r}} \begin{bmatrix}
\cos \theta & \left( 1 - \sin \frac{\theta}{2} \right) \\
\sin \theta & \left( 1 + \sin \frac{\theta}{2} \right)
\end{bmatrix}
\]

From the three-dimensional stress field expression, the maximum principal stress expression can be calculated as follows:

\[
\sigma_{1} = \frac{K_{1}}{\sqrt{2\pi r}} \cos \frac{\theta}{2} \left( 1 + \sin \frac{\theta}{2} \right)
\]
where \( K_1 \) is the dynamic stress intensity factor, which is \( K_1 = \sigma_{\theta \perp} \sqrt{2\pi R} \) for an open crack, and \( R \) is the half length of the crack. The strength factor of the surrounding rock will decrease continuously due to the damage to the surrounding rock caused by blasting. In the chamber excavation, the crack propagation can be judged by comparing the strength factor with the dynamic fracture toughness \( K_{1D} \) of the surrounding rock.

Cracking means that the maximum tensile stress of the concentrated stress field generated by the blasting stress at the crack tip exceeded the dynamic tensile strength of the damaged rock, and the critical state is:

\[
\sigma_1 = (1 - D)\sigma_D
\]  

(30)

Combined with the above analysis, we can obtain the following:

\[
\frac{K_1}{\sqrt{2\pi r}} \cos \frac{\theta}{2} \left( 1 + \sin \frac{\theta}{2} \right) = (1 - D)\sigma_D
\]

(31)

By solving for the crack length, it can be determined that the length of the blasting propagation crack is as follows:

\[
r = \frac{L_{b1}}{2} \left[ \frac{\sigma_{\theta \perp}}{(1-D)\sigma_D} \cos \frac{\theta}{2} \left( 1 + \sin \frac{\theta}{2} \right) \right]^2
\]

(32)

When \( \theta = 0 \), the maximum length of the crack propagation can be deduced. From this, the maximum propagation length \( r_{\text{max}} \) of the blasting crack under the far zone of the blasting stress wave can be calculated as follows:

\[
r_{\text{max}} = \frac{L_{b1}}{2} \left[ \frac{\sigma_{\theta \perp}}{(1-D)\sigma_D} \right]^2
\]

(33)

where \( L_{b1} = R_{b1} - r_t \) is the thickness of the first-stage stress release ring, and \( \sigma_D \) is the dynamic tensile strength of the rock.

Therefore, in the process of roadway blasting excavation, under the action of a blasting load, the maximum length of crack propagation of the stress loose zone under the action of the blasting load, \( r_{\text{max}} \), is equal to the expanded thickness of the blasting–expansion loose zone \( L_{b2} \):

\[
L_{b2} = r_{\text{max}} = \frac{L_{b1}}{2} \left[ \frac{\sigma_{\theta \perp}}{(1-D)\sigma_D} \right]^2
\]

(34)

According to the above analysis, the final loose zone thickness \( L_b \) is the combination of the stress loose zone thickness \( L_{b1} \) and the blasting–expansion loose zone thickness \( L_{b2} \). Here, influence coefficients \( m \) and \( n \) are introduced, which are related to the actual surrounding rock conditions of the project. Considering the influence of the coupling effect of the in situ stress on the blasting in the loose zone, the influence coefficient should satisfy the relationship \( m, n \geq 1 \), and the formula of the final surrounding rock loose zone thickness \( L_b \) is modified to the following:

\[
L_b = m L_{b1} + n L_{b2}
\]

(35)

3.2. Theoretical Derivation of Work–Energy Relationship Method

The work–energy relationship method refers to the method in which there is a relationship between the energy generated and the work done by an explosion to form a loose zone, and the thickness of the loose zone is determined through the relationship between the work and energy.

Because the internal energy of the system can be changed by transferring heat or performing work, there is a relationship between their units. The work performed by an explosion during the formation of a loose zone of the surrounding rock can be connected with the energy of the explosive itself through thermal work equivalence, and the scope of the loose zone can be studied and analyzed. Next, the functional relationship is analyzed
through two ideas, which are based on the simple functional theory and principles of fracture mechanics.

3.2.1. Based on Simple Work–Energy Relationship Theory

The blast heat works on the surrounding rock to overcome the resistance per unit area of the surrounding rock; that is, to overcome the tensile strength \( E_p \) of the surrounding rock, resulting in radial cracks and water wave circumferential cracks in the surrounding rock. Outside the roadway contour, within the range of the peripheral hole spacing \( a \) and blast hole charge length \( L \), the surrounding rock with a certain thickness \( H \) is broken, cracked, and loosened. Therefore, the mechanical work of the explosion heat on the surrounding rock can be calculated as follows [43]:

\[
W = ahLE_p \quad (36)
\]

The heat \( Q \) generated by the explosive loaded in the perimeter blast holes after the instantaneous explosion is:

\[
Q = keNqL \quad (37)
\]

Mechanical energy can generate heat by performing work, and heat can also perform work. Heat is a form of energy. The ratio of work \( W \) to its equivalent heat \( Q \) is the thermal work equivalent \( J \). That is, \( J = W/Q \). \( N \) is the charge per meter of the blast hole, \( q \) is the heat released when each kilogram of explosive explodes, and \( k \) is the correction coefficient, which increases with the increase in the surrounding rock grade.

The corrected thickness of the blasting–expansion loose zone \( L_{b2} \) is:

\[
L_{b2} = h = \frac{knqL}{hLE_p} \quad (38)
\]

Combined with the thickness of the stress loose zone derived based on the Fenner solution, the formula of the final loose zone thickness under the consideration of the influence coefficient is as follows:

\[
L_b = mL_{b1} + nL_{b2} \quad (39)
\]

3.2.2. Based on Principles of Fracture Mechanics

First, the total charge of a cycle is calculated with the volume formula, and then it is associated with the explosive energy per unit charge to obtain the total energy of each cycle blasting. Then, based on fracture mechanics principles, the work performed by the loose zone with a certain thickness of the surrounding rock is calculated, and the thickness of the loose zone is determined through the work–energy relationship. The formula for calculating the total charge is:

\[
N = nSL \quad (40)
\]

where \( N \) is the total charge of a blasting cycle (kg), \( n \) is the consumption of explosive required for blasting per cubic meter of rock mass (selected by looking up a table according to the rock firmness coefficient, \( \text{kg/m}^3 \)), \( S \) is the excavation section area (m²), and \( L \) is the planned cyclic footage (m).

Then, the explosive energy consumed per excavation footage is:

\[
Q = qN = qnSL \quad (41)
\]

where \( q \) is the energy released when each kilogram of explosive explodes (kJ/kg), which can be found in the Construction Manual of Shaft and Roadway Engineering.

The damage effect of blasting on the rock is mainly manifested in the formation of cracks in the surrounding rock; that is, the explosive works on the surrounding rock. Due to the stress wave generated by blasting at the boundary of the chamber, the rock is pulled tangentially to form a crack zone. For the crack formed by blasting, the plane wedge crack
model (under the action of stress wave, the initial crack grid is formed in the rock mass around the blast hole; then, under the quasi-static action of the explosive gas, the initial crack is penetrated and further extended, and finally the macro fracture of the rock mass is completed. At this time, the rock crack presents a wedge-shaped state in the plane region, so the crack model is called the plane wedge crack model) is more suitable. At the same time, according to the principle of rock fracture mechanics, the cylindrical charge blasting of the blast hole can be regarded as a plane strain problem, and the energy release rate per unit length can be obtained [44]:

\[ G_1 = \frac{(1-\mu^2)}{E} K_2^2 \]  

Due to crack length expansion \( \Delta a \), the stress intensity factor at the crack end can be considered constant. Therefore, it can be determined that the work performed by the tangential stress during the expansion of the crack is:

\[ W = n_1 \int_{r_p}^{R_2} \frac{(1-\mu^2)}{E} K_2^2 dr \]  

where \( n_1 \) is the number of radial main fractures, which is generally 8–12. When only the incident stress is considered to perform all the work, for an open crack, \( K_2 = \sigma_l \sqrt{2\pi R_j} \), which accounts for the effect of the blasting load on the crack propagation. \( R_j \) here is the integral of the distance from the blast hole center to the crack end. Combined with the above analysis, it can be seen that:

\[ W = \frac{n_1 \pi \lambda^2 r_p^2 (1-\mu^2)}{E (1-\alpha)} \left( \frac{R_2}{r_p} \right)^{2(1-\alpha)} - 1 \]  

where \( \lambda = \mu / (1-\mu) \) is the lateral pressure coefficient, \( \alpha = 2 - \mu / (1-\mu) \) is the pressure attenuation coefficient, \( P_1 = \rho_m D_0 \rho_0 D_1^2 / 2(D_0 \rho_m + D_1 \rho_0) \) is the initial impact pressure of the hole wall, \( r_p \) is the blast hole radius, \( E \) is the elastic modulus of the surrounding rock, and \( R_2 \) is the radius of the loose zone. The effect of blasting on the expansion of existing cracks is calculated from the blast hole.

It is assumed that the proportion of the blasting energy to the energy forming the loose zone is \( e \), and the effective energy utilization rate \( \epsilon \) is 23–48%, according to previous research. Because the energy released by explosives is considered to perform work on the surrounding rock, only the expansion of the main crack is considered in this process. The expansion of small- and medium-sized cracks and the energy consumed by penetration are ignored, the blasting energy correction coefficient \( f_k \) is introduced to correct for the loose zone deduced based on the work–energy relationship, and its parameters are related to the surrounding rock conditions and the effective energy utilization rate. Therefore, the heat generated by an explosion in the process of chamber excavation is converted into work performed on the rock:

\[ W = f_k eQJ \]  

According to the analysis, the formula for the radius of the surrounding rock blasting loose zone based on the work–energy relationship is as follows:

\[ R_{b2} = r_p \left[ \frac{E(1-\alpha)f_k eQJ}{n_1 \pi \lambda^2 r_p^2 (1-\mu^2)} + 1 \right]^{\frac{1}{2(1-\alpha)}} \]  

The thickness of the expanded loose zone under blasting action is \( L_{b2} = R_{b2} - r_t \). Combined with the thickness expression of the stress loose zone derived from the Fenner solution in the previous section, the formula for the final thickness of the surrounding rock loose zone considering the coupling coefficient is modified. The formula for the thickness of the surrounding rock loose zone after the correction is as follows:

\[ L_b = mL_{b1} + nL_{b2} \]
4. Applicability Analysis of Theoretical Calculation Method of Loose Zone

Analysis was performed on the single-track tunnel of the Qingdao Metro Line 1 project. Qingdao has a typical soil–rock binary composite stratum structure. In general, the surrounding rock is covered with quaternary soil layers of different thicknesses on a granite rock foundation with strong, medium, and slight weathering degrees. The station and interval tunnels pass through various weathered rock and soil layers, which are very uneven in the longitudinal direction, and the buried depth is about 17–26 m.

The acoustic method was adopted for the loose zone range field test. The process of the acoustic detection of the surrounding rock loose zone included the post-processing of the acoustic emissions, propagation, and reception display. The combination of a single hole, one generator, and two receivers was adopted for the test. The corresponding instruments were composed of a transmitting transducer, two receiving transducers, an RSM-RCT(B) acoustic logging tool, a liquid crystal display, and peripheral equipment. Inserting the test sensor assembly into the pre-drilled hole, first inject air into the air bag to block the hole, and then turn on the water pump. Then, start the transmitter–receiver T, and the receivers R1 and R2, respectively, receive the wave signal after different delays. The propagation speed of the sound wave in the rock mass at this position can be obtained through the computer program connected to the sensor assembly. Then, by changing the location of the test points, the rock mass propagation velocities at different depths can be measured, and these velocity points can be compared and analyzed to obtain the relevant data information of the whole tunnel loose circle. The test instrument and working principle are shown in Figure 5.

![Figure 5. Loose zone testing principle and instrument: (a) Loose zone test principle, (b) Loose zone test instrument.](image)

The field test diagram is shown in Figure 6.

![Figure 6. Loose zone range field test.](image)
Three typical working conditions were selected for the survey: the section of entrance B of Anshun station, the section of Anshun Road Station to Qingdao north station, and the section of Shuiqinggou station to Kaifeng Road station. The three tunnel sections had the same section form, which is shown in Figure 7.

Figure 7. Structural dimensions of the tunnel section (mm).

The measured results of the field acoustic method under three working conditions are shown in Figures 8–10. The results of the surrounding rock conditions and loose zone thickness obtained from the measured results and geological exploration data are shown in Table 2.

Figure 8. Monitoring waveform diagram of working condition 1: (a) Vault waveform; (b) spandrel waveform.
According to the field-measured waveform data, wave velocity analysis was carried out by the acoustic method. The wave velocity of the complete rock mass was generally high, while the wave velocity decreased relatively in the loose area of stress decline and fracture expansion. Therefore, there were significant wave velocity changes between the

![Figure 9](image-url-1)

**Figure 9.** Monitoring waveform diagram of working condition 2: (a) Vault waveform; (b) spandrel waveform.

![Figure 10](image-url-2)

**Figure 10.** Monitoring waveform diagram of working condition 3: (a) Vault waveform; (b) spandrel waveform.

<table>
<thead>
<tr>
<th>Working Condition</th>
<th>Surrounding Rock Type</th>
<th>Buried Depth (m)</th>
<th>Surrounding Rock Grade</th>
<th>Modulus of Elasticity (GPa)</th>
<th>Poisson's Ratio</th>
<th>Internal Friction Angle (°)</th>
<th>Cohesion (MPa)</th>
<th>Density (kg/m³)</th>
<th>Loose Zone Range (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Moderately weathered granite</td>
<td>20.0</td>
<td>IV</td>
<td>4.0</td>
<td>0.26</td>
<td>30</td>
<td>0.2</td>
<td>2400</td>
<td>1.2–1.4</td>
</tr>
<tr>
<td>2</td>
<td>Strongly weathered granite</td>
<td>22.5</td>
<td>IV-V</td>
<td>1.0</td>
<td>0.34</td>
<td>30</td>
<td>0.16</td>
<td>2200</td>
<td>1.8–2.6</td>
</tr>
<tr>
<td>3</td>
<td>Slightly weathered granite</td>
<td>23.0</td>
<td>III</td>
<td>4.0</td>
<td>0.26</td>
<td>38</td>
<td>0.2</td>
<td>2620</td>
<td>0.6–1.5</td>
</tr>
</tbody>
</table>

**Table 2.** Measured data of the surrounding rock and loose zone under different operating modes.
surrounding rock compaction area (increased stress area) and the loose area. However, it should be pointed out that the loose zone was not equal to the plastic zone but to the relaxed part of the rock mass in the plastic zone. By analyzing the measured wave velocity data, it could be ascertained that under the working conditions corresponding to moderately weathered granite, the thickness of the loose zone was 1.2–1.4 m. Under the working conditions corresponding to strongly weathered granite, the thickness of the loose zone was 1.8–2.6 m, and under the working conditions corresponding to slightly weathered granite, the thickness of the loose zone was 0.6–1.5 m. From the measured data, it could be seen that the worse the surrounding rock conditions, the greater the thickness of the loose zone.

The range of the loose zone under different working conditions can be obtained, as shown in Figure 11.

![Figure 11](image.png)

**Figure 11.** Schematic diagram of the loose zone range under three working conditions: (a) Schematic diagram of the loose zone range under working condition 1; (b) schematic diagram of the loose zone range under working condition 2; (c) schematic diagram of the loose zone range under working condition 3.

According to the field-measured data, under the same tunnel section, the higher the weathering degree of the surrounding rock, the worse the surrounding rock conditions, the deeper the burial depth, and the greater the thickness of the loose zone. Under the condition of strongly weathered and moderately weathered granite, the thickness of the surrounding rock loose zone was sorted as follows: arch shoulder > arch crown > arch waist; under the condition of weakly weathered granite, arch crown > spandrel > spandrel. The worse the surrounding rock condition, the greater the difference in the range of loose zones in different parts of the tunnel section. Through the above analysis, it could be seen
from the analysis of measured data that the scope of the surrounding rock loose zone was affected by surrounding rock conditions, burial depth, and section position.

Next, the stress method and work–energy relationship method described above were used to theoretically calculate the working conditions, and the results were compared with the actual values measured on site to test the applicability of these methods. During the calculation, the tunnel section was analyzed based on the equivalent circular tunnel section. The calculated results of tunnel radius are as follows:

\[ r_t = \frac{(H + B)}{4} = \frac{(6.6 + 6.2)}{4} = 3.2 \text{ m} \]  \hspace{1cm} (48)

With this circular tunnel radius, the stress method and work–energy relationship method were applied. The comparison between the theoretical calculation results and actual values is shown in Figure 12, and the comparison of the calculation results is shown in Table 3.

---

**Figure 12.** Calculation results and measured values of the loose zone range: (a) Working condition 1 loose zone range; (b) working condition 2 loose zone range; (c) working condition 3 loose zone range.
Table 3. Theoretical calculation results and measured values of the loose zone range under different operating modes.

<table>
<thead>
<tr>
<th>Working Condition</th>
<th>Stress Method (m)</th>
<th>Work–Energy Relationship Method 1 (m)</th>
<th>Work–Energy Relationship Method 2 (m)</th>
<th>Measured Value (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.36</td>
<td>1.47</td>
<td>1.43</td>
<td>1.2–1.4</td>
</tr>
<tr>
<td>2</td>
<td>2.47</td>
<td>2.73</td>
<td>2.66</td>
<td>1.8–2.6</td>
</tr>
<tr>
<td>3</td>
<td>1.17</td>
<td>1.51</td>
<td>1.21</td>
<td>0.6–1.5</td>
</tr>
</tbody>
</table>

By comparing the three theoretical calculation methods with the measured values, it can be seen that the error of the results calculated by the stress method and the work–energy method was 0.1–0.6 m on average, and the theoretical settlement results were more consistent with the maximum value of the measured value. Under the condition of moderately weathered granite and weakly weathered granite, the theoretical calculation value was close to the measured value at the arch crown, and under the condition of strongly weathered granite, the theoretical calculation value was close to the measured value at the arch waist. The thickness of the loose zone calculated by the stress method was in good agreement with the measured values. The thickness values of the loose zone calculated by the two energy relationship methods were conservative and mostly greater than the measured values. This was because, when blasting worked on the surrounding rock, different surrounding rock conditions had different utilization rates of the blasting energy, which were affected by the factors of the actual engineering conditions. In the process of practical theoretical calculations, the value of the effective energy utilization rate should be conservative.

5. Discussion on the Coupling of In Situ Stress and Blasting

In the process of calculating the range of the loose zone using the stress and work–energy relationship methods, the quantitative analysis of the coupling effect of the in situ stress and blasting on the formation of the final loose zone has always been a difficulty. Here, taking the stress method as an example, the coupling effect between the stress loose zone and the blasting–expansion loose zone is discussed through the comparison of the theoretically calculated values for both loose zones.

First, the calculation effect of the stress relaxation circle was analyzed through numerical simulations, and the three-dimensional numerical model was established by FLAC3D. The rock, soil, and tunnel body were modeled by solid elements, and the initial support was modeled by shell element simulations. The model size was 30 m (Y) × 12 m (Y) × 40 m (Z). The formation calculation parameters taken by the model are shown in Table 2, and the model diagram and calculation results are shown in Figure 13.

The calculation and numerical simulation results for the range of the stress relaxation zone using the Fenner solution under low ground stresses are shown in Table 4. The calculation results of the stress loose zone thickness $L_{\text{sl}}$ under different working conditions are shown in Figure 14.

The above analysis showed that when using the Fenner solution to find the stress loose zone, the calculated results were consistent with the numerical simulation results, which were quite different from the measured values. This showed that when using the Fenner solution to calculate the loose zone, the results only considered the loose zone caused by the stress release after the chamber excavation while ignoring the loose zone formed in the blasting process. The stress loose zone only accounted for a small part of the final loose zone. The main part was the loose zone produced by blasting crack propagation.
Figure 13. Cont.
Figure 13. Numerical model and simulation results: (a) Schematic diagram of the numerical model; (b) simulation results of condition 1; (c) simulation results of condition 2; (d) simulation results of condition 3.

Table 4. Theoretical calculation and numerical simulation results of stress loose zone.

<table>
<thead>
<tr>
<th>Working Condition</th>
<th>Theoretical Calculation (m)</th>
<th>Numerical Simulation (m)</th>
<th>Measured Value (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.106</td>
<td>0.235</td>
<td>1.2–1.4</td>
</tr>
<tr>
<td>2</td>
<td>0.326</td>
<td>0.487</td>
<td>1.8–2.6</td>
</tr>
<tr>
<td>3</td>
<td>0.254</td>
<td>0.408</td>
<td>0.6–1.5</td>
</tr>
</tbody>
</table>

In the above derivation and analysis, it was found that the final calculated value was smaller than the measured value after adding the thickness of the stress loose zone, and the blasting extended the loose zone directly. This was because, in the actual chamber excavation process, the formation of the loose zone under the action of in situ stress and the formation of the blasting stress wave, which extended the loose zone, did not occur independently but had a coupling effect. The calculation results and coupling coefficients of the stress loose zone and blasting–expansion loose zone are discussed below. The calculation results are shown in Table 5, where $L_{b1}$ is the thickness of the stress loose zone, $L_{b2}$ is the thickness of the blasting–expansion loose zone, $L_b$ is the thickness of the final loose zone, and $m$ and $n$ are the coupling coefficients.

As discussed above, the higher the grade of the surrounding rock, the smaller the values of $m$ and $n$, approaching 1.0. The lower the grade of the surrounding rock, the larger the values of $m$ and $n$. In comparison, the variation range of $m$ was larger, which showed that in the process of the loose zone formation, although the range of the loose zone under the influence of the in situ stress was small after chamber excavation, the influence of the in situ stress on the final loose zone formation was large. This was consistent with the loose zone theory proposed by Professor Dong (the size of the surrounding rock loose zone was a function of the in situ stress and surrounding rock strength). In addition, the values of $m$ and $n$ should be obtained from the site engineering conditions, if possible, and the difference between the values should not be too large.
In this paper, the range of the surrounding rock loose zone was theoretically analyzed by using the stress method and work-energy relationship method. The theoretical formula derived in this paper was compared with the empirical formula of Fenner, and the results showed that the theoretical calculation results were smaller than the measured values after adding the thickness of the stress loose zone, and this difference was more significant when the surrounding rock was harder. For different grades of surrounding rock, the theoretical calculation results were smaller than the measured values after adding the thickness of the stress loose zone, and the ratio tended to approach 1.0 with the increase of the grade of the surrounding rock. The reasons for this phenomenon were that the stress loose zone and blasting expanded loose zone acted independently but had a coupling effect. The calculation results and coupling coefficients are shown in Table 5, where the formation of the blasting stress wave, which extended the loose zone, did not occur in the excavation process, the formation of the loose zone under the action of in situ stress and blasting extended the loose zone directly. This was because, in the actual chamber excavation process, the stress release after the chamber excavation while ignoring the loose zone caused by the stress release after the chamber excavation while ignoring the loose zone, the calculation results only considered the loose zone formed in the blasting process. The stress loose zone only accounted for a small part of the final loose zone. The main part was the loose zone produced by blasting crack propagation, which were quite different from the measured values. This showed that when using the Fenner solution to calculate the loose zone, the results only considered the loose zone formed in the blasting process.

Table 5. Calculation results of the loose zone of each part and value of the coupling coefficient.

<table>
<thead>
<tr>
<th>Working Condition</th>
<th>L_{b1} (m)</th>
<th>m</th>
<th>L_{b2} (m)</th>
<th>n</th>
<th>m, n</th>
<th>L_{b}</th>
<th>Measured Value (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.106</td>
<td>1.0–3.6</td>
<td>1.026</td>
<td>1.0–1.3</td>
<td>m = 1.2, n = 1.2</td>
<td>1.36</td>
<td>1.2–1.4</td>
</tr>
<tr>
<td>2</td>
<td>0.326</td>
<td>1.0–3.0</td>
<td>1.599</td>
<td>1.0–1.4</td>
<td>m = 1.2, n = 1.3</td>
<td>2.47</td>
<td>1.8–2.6</td>
</tr>
<tr>
<td>3</td>
<td>0.254</td>
<td>1.0–1.2</td>
<td>0.913</td>
<td>1.0–1.2</td>
<td>m = 1.0, n = 1.0</td>
<td>1.17</td>
<td>0.6–1.5</td>
</tr>
</tbody>
</table>

6. Conclusions
In this paper, the range of the surrounding rock loose zone was theoretically analyzed by using the stress method and work-energy relationship method. The theoretical formula
for the loose zone thickness was given, and the formula was modified through the coupling coefficient. Based on an example of the Qingdao Metro single-track tunnel project, the applicability of the formulas of the two theoretical methods was analyzed. The final calculation results showed that the stress method was more applicable, while the calculation results of the work–energy relationship method were more conservative. Through this study, we drew the following conclusions:

1. The study of this paper shows that the scope of the loose zone of surrounding rock is affected by the original rock stress, surrounding rock conditions, burial depth, and blasting parameters. In the case of the same tunnel section, the worse the surrounding rock conditions, the greater the original rock stress, the larger the scope of the loose zone, and the greater the difference between the scope of the loose zone at the arch crown and the arch shoulder.

2. The formation of the surrounding rock loose zone can be divided into two parts. One part is the stress loose zone formed under the action of in situ stress due to the stress release after chamber excavation, and the second part is the blasting–expansion loose zone formed by the crack propagation under the action of a blasting load.

3. When using the Finner solution to calculate the range of the surrounding rock loose zone under low ground stress, only the stress loose zone underground stress is considered, and the blasting expansion loose zone under the blasting load is ignored, resulting in a large difference between the calculated value of the surrounding rock loose zone and the measured value. The stress loose zone only accounts for a small part of the surrounding rock loose zone, and the blasting expansion loose zone is the main part of the surrounding rock loose zone.

4. In the process of actual chamber excavation, the formation of a loose zone under the action of in situ stress and the expansion of the blasting stress wave were not simple and independent but rather had a coupling effect. Although the range of the loose zone under the influence of the in situ stress was small after chamber excavation, the influence of the in situ stress on the formation of the final loose zone was large.


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