

Article

Mechanical Characteristics of Surrounding Rock for Neighborhood Tunnels Using the Schwarz Alternating Method Model: A Case Study

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Abstract: During the drilling and blasting excavation of neighborhood tunnels, blast-induced vibrations negatively affect the stability of the interlaid rock, particularly for the following tunnel. This paper presents a case study of neighborhood tunnels with small clearance in Shenzhen, China, where the minimum thickness of interlaid rock is only 0.5 m. Therefore, the tunnelling method of the following tunnel should be precisely designed to ensure the safety of surrounding rock. Initially, we investigated the damage mechanism of the interlaid rock under the blasting load from the following tunnel using LS-DYNA R11.1 software. To control the damage of the interlaid rock caused by the following tunnel blasting, the four-part excavation method with a reserved vibration-cushioning layer for the following tunnel is proposed. Subsequently, the analytical stress of the surrounding rock for neighborhood tunnels was obtained by the Schwarz alternating method (SAM). By analyzing the variation patterns with different thicknesses of the cushioning layer, the optimal thickness of the cushioning layer was determined to be 3.0 m. Consequently, a safety excavation partition scheme was implemented for the following tunnel. As a result of this case study, suggestions were identified for the safe excavation of neighborhood tunnels with small clearance.

Keywords: neighborhood tunnels; safety excavation; Schwarz alternating method (SAM); tunnelling method; LS-DYNA



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1. Introduction

Interlaid rock is a crucial component of neighborhood tunnels, playing supporting and reinforcing roles between the two tunnels. During tunnel excavation using drilling and blasting methods, the cumulative damage from blasting vibrations on the interlaid rock can lead to a decrease in its mechanical properties [1], posing a threat to the stability of the tunnel structure. In neighborhood tunnels, where the distance between the dual tunnels is relatively small, the constructions of an advance tunnel (AT) and a following tunnel (FT) mutually influence each other. In the research process, it is essential not to consider a single tunnel in isolation. Studies [2–5] indicate that the excavation of the FT has a greater impact on the AT. Lin et al. [3] monitored the vibration velocity induced by FT blasting and found that the peak particle velocity (PPV) at the monitoring point close to the interlaid rock was 5-to-10-times more than that of away from the interlaid rock in the AT. The results indicate that the negative influence for interlaid rock is mainly from FT blasting. Li et al. [4] and Song et al. [5] have also reached this conclusion.

Therefore, the key technical aspects [6] of safe construction in neighborhood tunnels mainly include appropriate clearance distance, support parameters, construction methods, and reinforcement techniques for the interlaid rock, among others.

For the AT in neighborhood tunnels, its excavation method can be referenced from conventional single-tunnel construction. The selection of an excavation method for the FT, however, leans more towards geological conditions and the surrounding environment.

In favorable geological conditions, the full-face method [7] or the benching method [8] are commonly adapted. In challenging geological conditions, options include the center diaphragm method with multiple sections [9,10], the cross-diaphragm method [11,12], or the method of circular excavation with reserved core soil [13]. For the study cases where the thickness of the interlaid rock is small, it is necessary to optimize conventional excavation methods. In the excavation of neighborhood tunnels with a 2.7 m thick interlaid rock, Liu et al. [14] have proposed the method of reserving a thin rock layer for segmented excavation to safely excavate the FT. This is an effective solution. However, the thickness of the reserved rock layer is determined based on construction experience.

Wu et al. [15] have simplified the cross-section of laterally excavated tunnels into a dual-connected domain problem using the method of equivalent circles. By combining the theory of complex variables and the Schwarz alternating method (SAM), they derived the specific form of the stress function for laterally excavated tunnels and established the relationship between single excavation width, surrounding rock stress, and displacement characteristics. The SAM is a successive approximation analytical method for studying the problems of double circular holes [16,17] and multiple elliptic holes [18,19] in solids. Zeng et al. [20] and Lin et al. [21] have used the SAM to solve the analytical stress and displacement of twin tunnels. The analytical solutions are verified by the good agreement between the analytical and finite-element method results under the same assumptions. Inspired by those researchers, we similarly simplify the excavation of neighborhood tunnels into a dual-connected domain problem using the SAM. We deduce the impact of the small clearance distance on the surrounding rock stress of the two tunnels and use numerical methods to determine the optimal distance between neighborhood tunnels. This information is then used to calculate the thickness of the reserved rock layer.

This paper takes the Shenzhen Liantang tunnel, a typical neighborhood tunnels project, as the engineering background, where the minimum distance between the two tunnels is 0.5 m. The aim is to safely excavate the FT and protect the interlaid rock. Firstly, numerical simulations are employed to study the damage characteristics of the interlaid rock under the blasting load of the FT. This information serves as a basis for designing the excavation method of the FT. Subsequently, to obtain the excavation partition scheme of the FT, theoretical analyses are conducted to determine the parameters of the tunnelling method by the SAM. The mathematic model is established using the equivalent circle method. The results serve as the theoretical basis for the parameters of the excavation partition scheme for the FT.

2. Engineering Background

2.1. Project Overview

Shenzhen Eastern Transit Expressway is a major construction project in Shenzhen, Guangdong Province, China, starting at the planned Liantang Port and ending at the Qian Au interchange of the Shenzhen–Shantou and Huiyan expressways. In this project, the Liantang tunnel, located in the Luohu District of Shenzhen, is the key component of the Shenzhen Eastern Transit Expressway. As shown in Figure 1, the Liantang tunnel is an important link between the Liantang Port and Shenzhen's trunk road network. The mainline section of the tunnel is a freeway-separated double-hole eight-lane tunnel with a total length of 2.65 km and a design speed of 60 km/h.

In this paper, we focus on the bifurcation section of the Liantang tunnel. The main features of the bifurcation section include a super-large section tunnel and the neighborhood tunnels with small clearance. The neighborhood tunnels are composed of two non-circular tunnels, the AT and FT. The two tunnels present a non-parallel and asymmetric “dovetail” structure, with an included angle of 10° . Thereby, the thickness of the interlaid rock increases with tunnelling. However, the minimum thickness of interlaid rock between the two tunnels is 0.5 m. According to the tunnel classification by the International Tunnel Association [22], the neighborhood tunnels can be classified in the super-small clearance tunnel group.

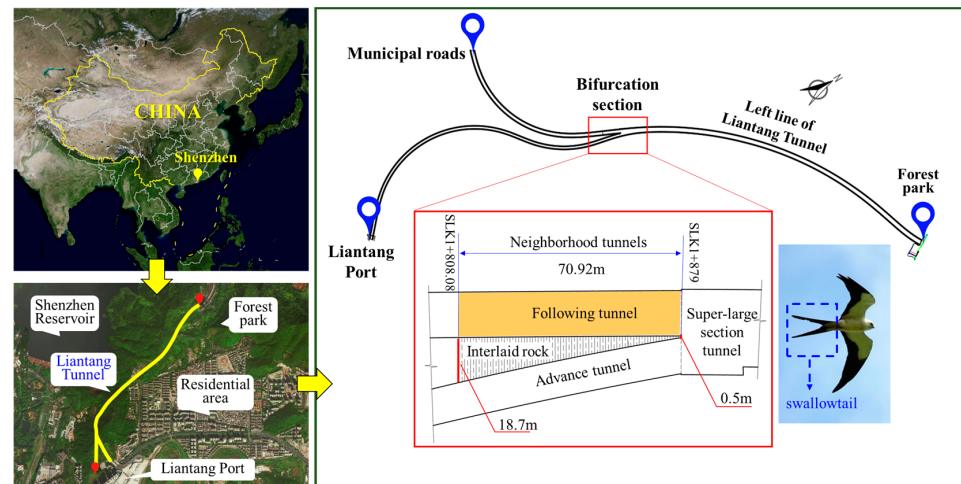


Figure 1. Structural characteristics of the Liantang tunnel.

The neighborhood tunnels were excavated into a mountainous area. The depth of overburden above the tunnels is appropriately 90 m. The tectonic stress on the surrounding rock can be ignored. The surrounding rock mainly included slightly weathered sandstone, which is colored as gray-brown and blue-gray. The elastic modulus and Poisson's ratio of the slightly weathered sandstone are 12 GPa and 0.27, respectively. The natural density of rock mass is appropriately 2500 kg/m³. Based on the Chinese basic quality (BQ) rock mass classification system [23], the surrounding ground is mainly categorized as class III with developed joints and fissures. The tunnels were planned to be excavated using the benching tunnelling method with traditional drilling and blasting techniques.

2.2. Construction of the Following Tunnel

Figure 2 shows the cross-section design and supporting structure diagram of the neighborhood tunnels. The AT is a standard two-lane, and the FT is a standard three-lane. Furthermore, both tunnels are three-centered arch structures. The excavation area of the AT and FT are 103.0 m² and 139.2 m², respectively. When the thickness of the interlaid rock between the two tunnels is 0.5 m to 1.5 m, the interlaid rock plays a supporting role without anchors. The tunnels are constructed by drilling and blasting method. After the completion of the AT excavation, the left side of interlaid rock is exposed to the free surface, and its stability is guaranteed by the supporting structure of AT. During the blasting of the FT, the interlaid rock is in the condition of three free surfaces. Once the vibration induced by the FT blasting is out of control, the blasting load will break through the interlaid rock, which will lead to safety accidents such as spalling and collapse. This is a potential safety hazard to the stability of surrounding rock of the tunnel.

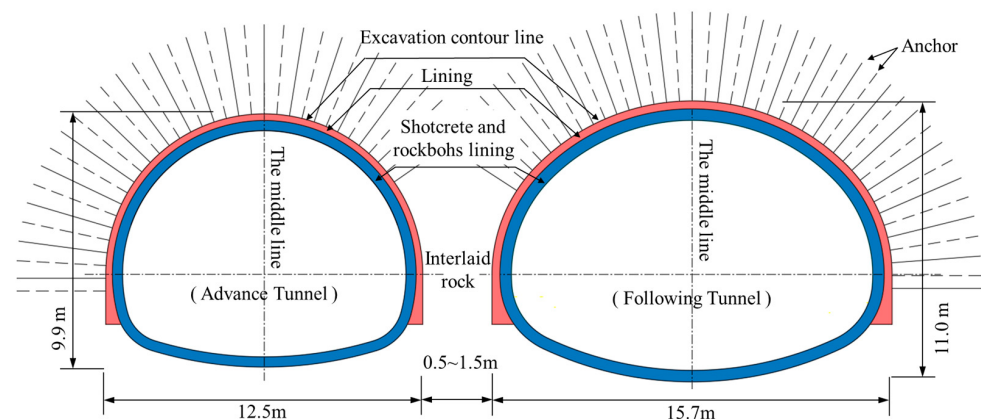


Figure 2. Design of the neighborhood tunnels.

3. Damage Mechanism of FT Blasting Load on Interlaid Rock

The main structural feature of the neighborhood tunnels with small clearance in the Liantang tunnel project is that the distance between two tunnels is small. Thereby, the mechanical behavior of construction is complex. During the blasting excavation of two tunnels, the stress field and deformation field of surrounding rock interact with each other. At present, there are few cases of neighborhood tunnels with a distance of less than 1 m. There is a lack of research topics on similar projects. To study the effect of the FT divisional excavation scheme on the interlaid rock, firstly, LS-DYNA R11.1 software was used to analyze the damage mechanism of interlaid rock under the FT blasting load, and the results will be used as a theoretical basis for the design of the FT divisional excavation scheme.

3.1. Model and the Basic Mechanical Parameters of Materials

For the tunnel blasting design, all blastholes can be classified into two categories, one is the blasthole with single free surface, such as cut blasthole; the other is the blasthole with double free surfaces, such as easer blasthole, peripheral blasthole, etc. During blasting, the cavity of rock mass produced by cut blasting increases the number of free surfaces for blastholes, especially peripheral holes. Smooth blasting describes making rational use of the free surface conditions produced by blasting. By reducing the charge weight and spacing of blastholes, the effect of half-hole marks after blasting is produced by the superposition of stress waves, so that the tunnel excavation profile surface is flat. To compare the damage process of rock caused by single-hole blasting loads under different free-surface conditions, two numerical simulation calculation models of single-hole blasting were established based on the 40 mm blasthole commonly used in tunnel blasting, as shown in Figure 3. The dynamic calculation software LS-DYNA R11.1 was used to simulate the damage process of rock around the single-hole blasting with different numbers of free surfaces.

The rock material can be modeled by the Riedel-Hiermaier-Thoma (RHT) model [24], which describes the variation in the initial yield strength, failure strength, and residual strength of the materials by introducing three limit surfaces: the elastic limit surface, the failure surface, and the residual failure surface. History variable #4 [25] in the post-processing of simulation results can be used as a damage variable of materials.

For the No.2 rock emulsion explosive, the MAT_HIGH_EXPLOSIVE_BURN model [25] was adapted to model the detonation of a high explosive. In addition, the Jones-Wilkens-Lee (JWL) equation of state is applied to describe the hydrodynamic of explosive detonation products. Tables 1 and 2 list the material parameters required in the calculation model.

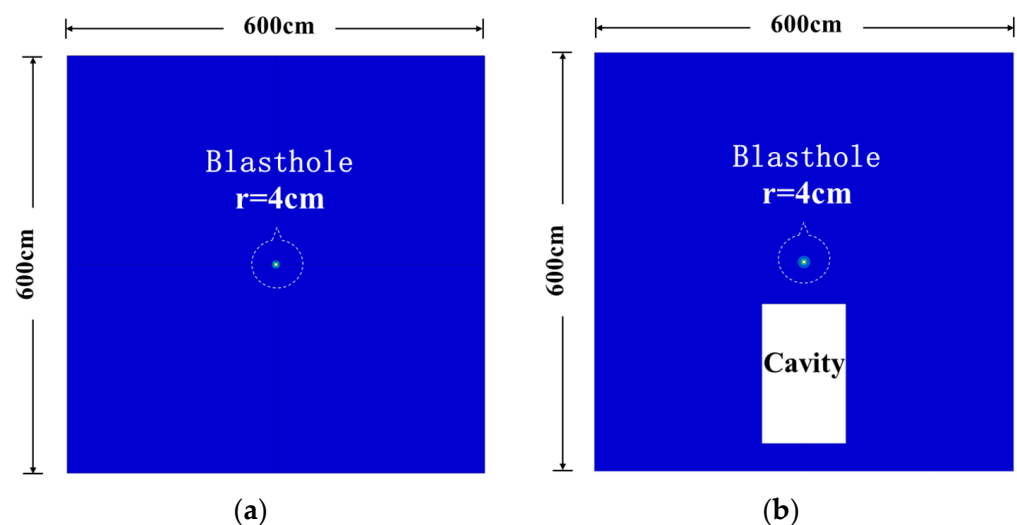


Figure 3. Single-hole blasting model with different numbers of free surface. (a) Single-hole blasting model with single free surface. (b) Single-hole blasting model with double free surfaces.

Table 1. Physical and mechanical parameters of slightly weathered sandstone.

Density (kg/m ³)	Compressive Strength (MPa)	Tensile Strength (MPa)	Elastic Modulus (GPa)	Poisson's Ratio
2450	47.51	3.67	8.35	0.15

Table 2. Material parameters of the No.2 rock emulsion explosive.

Density (kg/m ³)	Detonation Velocity, D (m/s)	Chapman-Jouget Pressure, P_{CJ} (GPa)	Parameters of JWL Equation of State					
			A (GPa)	B (GPa)	R_1	R_2	ω	E_0 (GPa)
1300	4000	9.70	214.4	0.182	4.2	0.9	0.15	4.192

3.2. Damage Process of the Interval Rock

The simulation results are shown in Figures 4 and 5. In the semi-infinite rock mass and rock mass with the cavity, the crush zone and the fracture zone formed by the blasting load have the same size, and the radii of two zones are 0.06 m and 1.23 m, respectively. However, in the fracture zone, the number of fractures in the model with the cavity was significantly lower than that in the semi-infinite rock mass. Meanwhile, the fracture density on the side of the temporary cavity was obviously higher than that in semi-infinite rock mass due to stress wave reflection. Finally, the blasting funnel was formed on the side of the cavity due to the free surface.

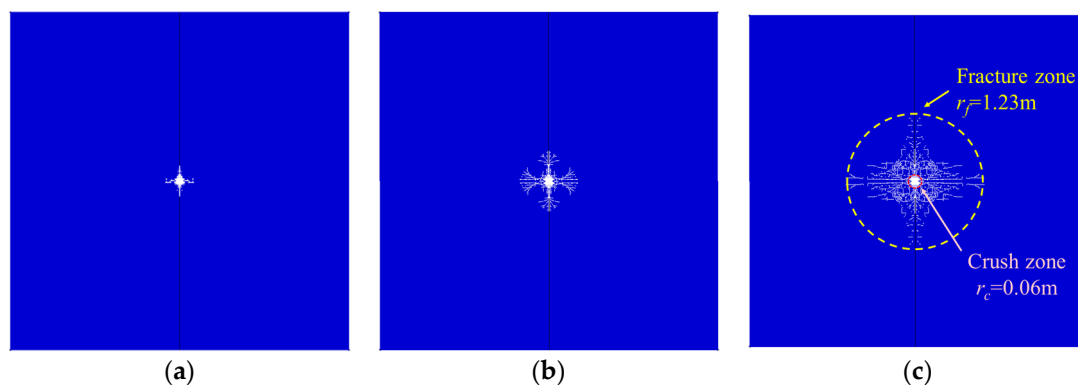


Figure 4. Damage process of single-hole blasting model with single free surface. (a) $t = 100 \mu\text{s}$; (b) $t = 160 \mu\text{s}$; (c) $t = 400 \mu\text{s}$.

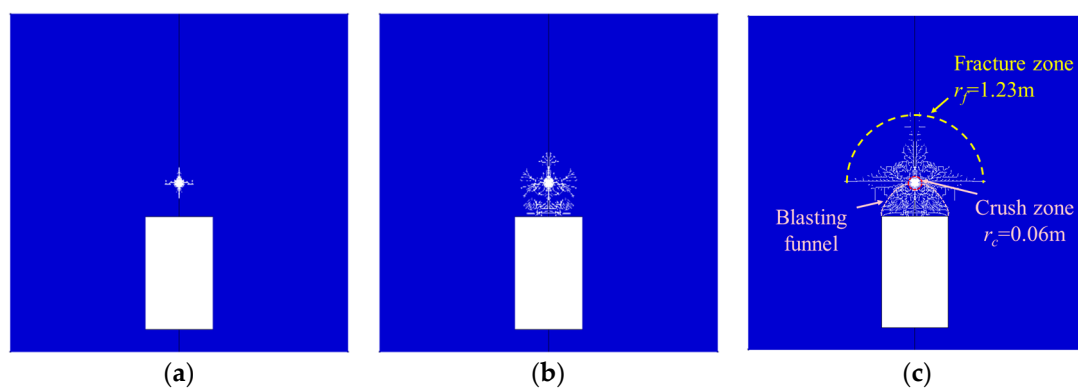


Figure 5. Damage process of single-hole blasting model with double free surface. (a) $t = 100 \mu\text{s}$; (b) $t = 160 \mu\text{s}$; (c) $t = 400 \mu\text{s}$.

The correlation analysis of the blasting process of a peripheral blasthole near interlaid rock in the FT can be seen in Figure 6. As shown in Figure 6, the free surfaces on both

sides of the peripheral blasthole are composed of the excavation space in the AT and the temporary surface produced by easier blastholes in the FT. When the stress wave induced by blasting propagates to the free surface in the FT, it will reflect and break the rock mass on the side of the minimum burden (the minimum distance between the blasthole and free surface). So, the explosion energy on this side is converted into the kinetic energy of the rock for throwing of the broken rock. When the stress wave is propagated to the free surface in the AT, it will be launched onto the supporting structure surface of AT, which will aggravate the damage to the interlaid rock. Moreover, if the blasting load exceeds the damage strength of the rock, the interlaid rock mass will be seriously damaged, resulting in its instability. The dynamic response of the interlaid rock subjected to the blasting load is a process of cumulative damage, not of only the blasting load of peripheral holes. Therefore, to protect the neighborhood tunnels with thin-thickness interlaid rock from blasting load, it is necessary not only to consider the control blasting technology, but also to analyze the excavation methods under the conditions of different numbers of free surfaces.

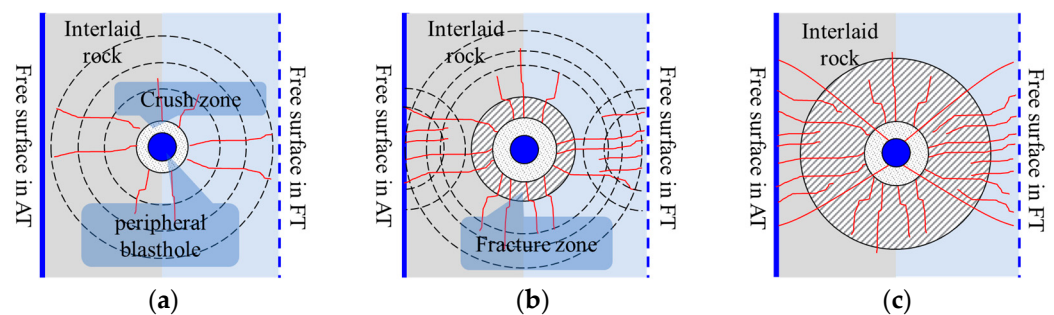


Figure 6. Schematic diagram of the action boundary of peripheral blasthole blasting in FT. (a) Compression stage; (b) reflection stage; (c) funnel formation stage.

3.3. Divisional Excavation Scheme of the FT

Based on the above discussion, on the basis of the conventional blasting excavation zoning in the FT, a method incorporating the excavation in four parts with a reserved vibration-cushioning layer is proposed, as shown in Figure 7. Different from the conventional four-part excavation method with a single side wall, the excavation sequence of the four-part excavation method with a reserved vibration-cushioning layer is optimized from ‘up to down’ to ‘left to right’. By reserving a thin cushioning layer (marked as FT-II), the first blasting excavation area (marked as FT-I) is separated from the interlaid rock, reducing the direct impact of the blasting load on the interlaid rock. Under the action of the blasting load in FT-I, the mechanical properties of the rock mass are weakened, and the amount of explosive charge in the blastholes can be appropriately reduced during blasting excavation. On the other hand, the excavation volume of the FT-II is small, with fewer blastholes, facilitating precise control. Therefore, to achieve safe excavation of the FT through blasting, the thickness of the FT-II is a key parameter for the safe excavation of the FT.

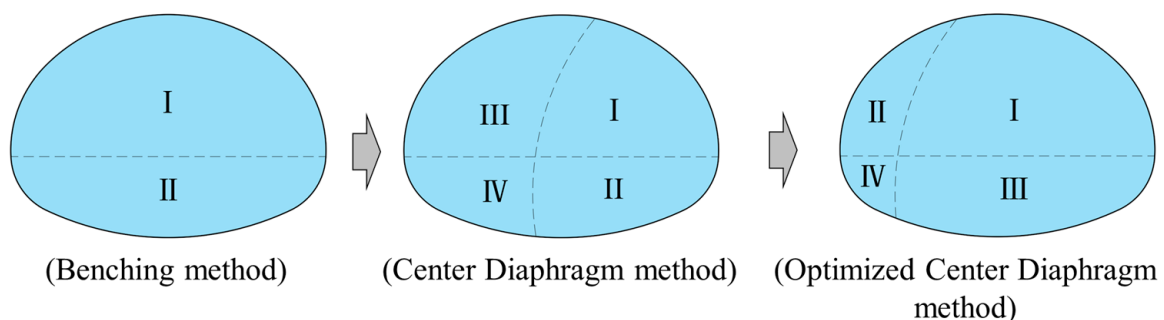


Figure 7. Optimization process of tunnelling method for the FT.

4. Key Parameters of Tunnelling Method for the FT

To obtain the optimal parameters of the four-part excavation method with a reserved vibration-cushioning layer, this chapter discusses the impact of different layer thicknesses on the stress of the surrounding rock in the FT through numerical methods. The optimal solution for the layer thickness is obtained by comparing the computational results.

4.1. Overview of the SAM

As shown in Figure 8, consider the excavation of tunnels AT and FT-I as the problem of two arbitrarily shaped cross-sectional tunnels. Assume that in a semi-infinite plane, tunnel 1 (AT) and tunnel 2 (FT-I) can be excavated at arbitrary positions with any shape. To determine a reasonable spacing between the two tunnels, an analysis of the forces acting on the boundaries of both tunnels is conducted. Due to the existence of two tunnel boundaries, this engineering problem can be analogized as a dual-connected domain problem for discussion and solution. Based on this approach, the forces on the tunnels are solved using the commonly employed analytical method of the SAM for dual-connected domain problems. Consequently, the optimal and reasonable spacing between cross-sections AT and FT-I is obtained, leading to the determination of the value for the thickness of the reserved vibration-cushioning layer, denoted as d_2 .

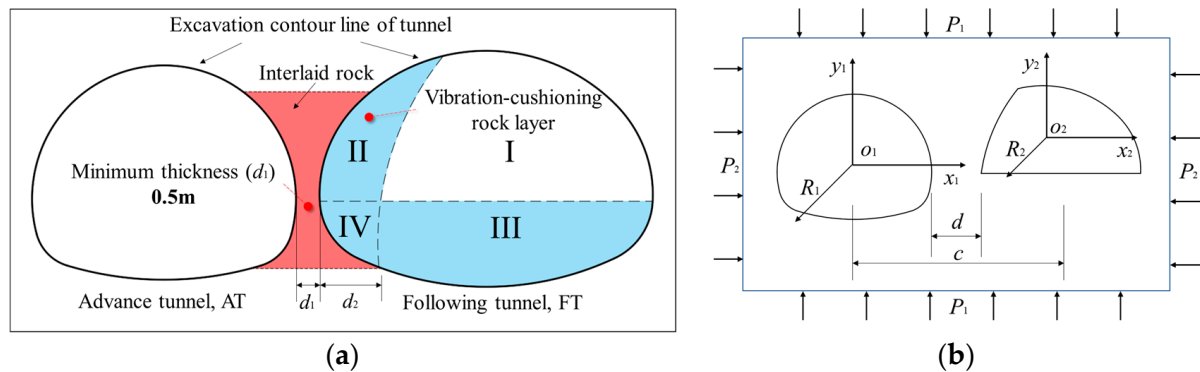


Figure 8. (a) Cross-section of the neighborhood tunnels. (b) Schematic diagram of calculation model.

4.2. Mathematic Model of SAM

According to the cross-section characteristic of the neighborhood tunnels, the AT and part I of the following tunnel (FT-I) are discussed as a double-connected domain problem in underground engineering. The stress and displacement at any point on the surrounding rock of the AT and FT-I section can be solved using the Schwarz alternating method (SAM). Based on the results of the analytical solution, the optimal value of d_2 can be determined. However, the AT section outline and FT-I outline are non-circular, causing great difficulties in calculating the stress field by the SAM [26]. Wang et al. [27] have concluded that the straight-wall arches, curved-wall arches, and three-center arch tunnels can be simplified into circles using the method of conformal transformation to equivalent circles. The equivalent circle radius (R) is given by

$$R = (h + l)/4 \quad (1)$$

where h is the section height, and l is the section width.

The mathematic model is shown in Figure 9. Two rectangular coordinated systems $o_1x_1y_1$ and $o_2x_2y_2$ are established. Circle 1 and circle 2 are the equivalent circle of the AT section outline and FT-I outline. The distance between two circles center is c , while the horizontal distance between the outline of the two circles is d . As shown in Figure 9, d equals to the sum of the minimum thickness of the interlaid rock (d_1) and the width of the reserved vibration-cushioning layer (d_2). To solve the problem, the following assumptions are made:

(1) The geomaterial is elastic, homogeneous, and isotropic in the plane of the tunnel surrounding (similar assumptions can be found in Refs Lu et al. [28] and Zhang et al. [29]). This paper focuses on the influence of the changed d on the stresses of the tunnels surrounding rock. Therefore, the support structures are not included in the mathematic model.

(2) The external loads P_1 and P_2 are applied to the ground at infinity (similar assumptions can be found in Refs Zhang et al. [23]). Based on the Chinese Highway Tunnel Design Rules, the neighborhood tunnels can be classified as the shallow tunnel. Therefore, the tectonic stress is not obvious. Here, P_1 is the vertical crustal stress under gravity. P_1 is given by

$$P_1 = pH \quad (2)$$

where p is the average bulk density, and H is the average buried depth of the tunnel. P_2 is the horizontal ground stress, which is equal to P_1 multiplied by the lateral pressure coefficient γ (usually equal to 0.3).

(3) For the conventional blasting pattern of tunnels with surrounding rock class III, the row spacing of functional blastholes is 0.6 m [30], which would be taken as the minimum and the tolerance of the series of d_2 . Moreover, the maximum of d_2 has been defined by half of the FT-I section width: 5.9 m.

(4) The tension is positive, and the pressure is negative.

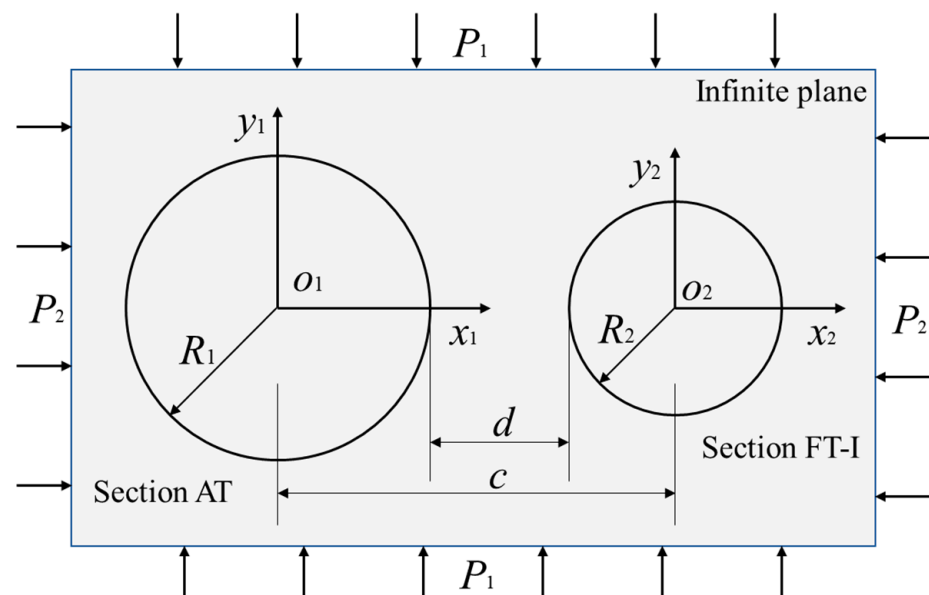


Figure 9. Mathematic model of two sections.

4.3. Fundamental Theories and Equations for Solution

The SAM aims to decompose the multi-connected domain problem into a series of single-connected domain problems and yields an approximate solution through several interactions. The solution procedure of the SAM for solving the double-connected domain problem is shown in Figure 10. First, the AT section has been excavated in the infinite plane. The stress on the FT-I section outline can be solved under the external load, which is a single-connected domain problem. Subsequently, the FT-I section would be excavated. To balance the forces on the surrounding rock of the FT-I section, an external balance force is added to make the external load approach zero. Similarly, the stress on the AT section outline can be obtained when the FT-I section has been excavated.

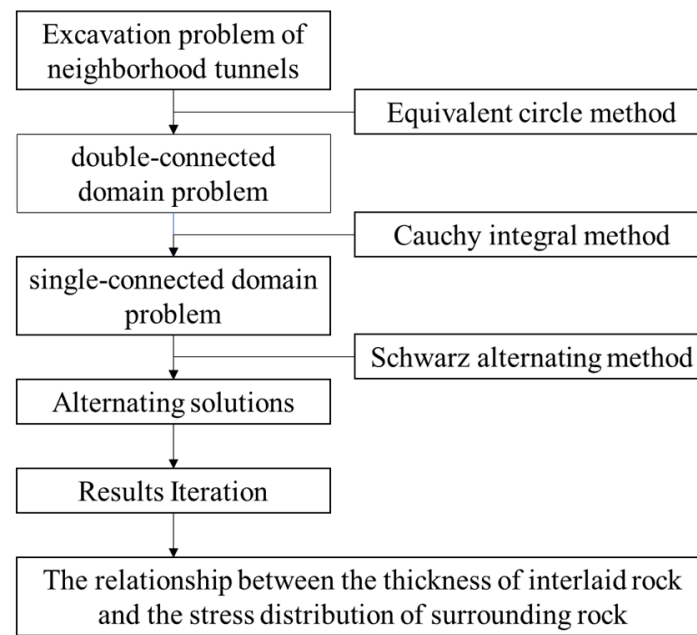


Figure 10. Solution procedure of the SAM for solving double-connected domain problem.

The double-connected domain problem can be solved by superimposing the solutions until the peripheral surface forces of the two circular sections reach zero. In fact, due to the limited number of iterations, it is impossible to make the peripheral force of the two circular cross-sections reach zero [15]. The iterative process keeps approaching acceptable values, but high-precision analytical solutions can still be obtained. Lu et al. [31] have indicated that the results of 3 or 4 iterations can accurately obtain the stress distribution at various positions and achieve engineering accuracy, which can be used as a theoretical basis for tunnelling and support.

The iteration processes and the functions that need to be solved are listed in Table 1. To distinguish all the symbols for each interaction, the complex stress functions $\varphi_1(z)$ and $\psi_1(z)$ were defined for the coordinate systems $o_1x_1y_1$; $\varphi_2(z)$ and $\psi_2(z)$ were for the coordinate system $o_2x_2y_2$. For the functions in Table 3, the number in the superscript parentheses of the function denotes the number of iterations. The odd numbers in the superscript denote the surface forces around the AT section, and the even numbers denote the surface forces of the FT-I section. The subscript numbers denote the coordinate system number.

Table 3. Solution procedure and functions of the SAM.

Number of Iterations	Step	Coordinate System	Content			Function for Solved	Load
			Excavate AT	Excavate FT-I	Content to Be Solved		
1	(a)	$o_1x_1y_1$	✓		Solve the complex stress around AT	$\varphi_1^{(1)}(z_1), \psi_1^{(1)}(z_1)$	Far-field load
	(b)	$o_2x_2y_2$	✓		Solve the complex stress around FT-I	$\varphi_2^{(1)}(z_2), \psi_2^{(1)}(z_2)$	Far-field load
2	(c)	$o_2x_2y_2$		✓	Solve the complex stress around FT-I	$\varphi_2^{(2)}(z_2), \psi_2^{(2)}(z_2)$	Balance external forces
	(d)	$o_1x_1y_1$		✓	Solve the complex stress around AT	$\varphi_1^{(2)}(z_1), \psi_1^{(2)}(z_1)$	Balance external forces

Table 3. Cont.

Number of Iterations	Step	Coordinate System	Content		Function for Solved	Load
			Excavate AT	Excavate FT-I		
3	(e)	$o_1x_1y_1$	✓		Solve the complex stress around AT $\varphi_1^{(3)}(z_1), \psi_1^{(3)}(z_1)$	Balance external forces
	(f)	$o_2x_2y_2$	✓		Solve the complex stress around FT-I $\varphi_2^{(3)}(z_2), \psi_2^{(3)}(z_2)$	Balance external forces
4	(g)	$o_2x_2y_2$		✓	Solve the complex stress around FT-I $\varphi_2^{(4)}(z_2), \psi_2^{(4)}(z_2)$	Balance external forces
	(h)	$o_1x_1y_1$		✓	Solve the complex stress around AT $\varphi_1^{(4)}(z_1), \psi_1^{(4)}(z_1)$	Balance external forces

Figure 11 shows the vector relationship of the coordinate system of circle 1 and circle 2. The transformation relationship between the derivative of the complex stress function of the dual coordinate system is given by Equations (3) and (4).

$$\varphi_2'(z_2) = \varphi_1'(z_2 + c) \quad (3)$$

$$\psi_2'(z_2) = \psi_1'(z_2 + c) + \bar{c}\varphi_1''(z_2 + c) \quad (4)$$

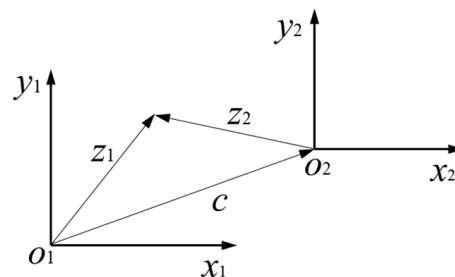


Figure 11. Illustration of the vector relationship between the coordinate system of circles 1 and 2.

Integrating both sides of Equations (3) and (4) and deleting the integration constants, the transformation relationship of the complex stress functions is obtained as follows:

$$\varphi_2(z_2) = \varphi_1(z_2 + c) \quad (5)$$

$$\psi_2(z_2) = \psi_1(z_2 + c) + \bar{c}\varphi_1(z_2 + c) \quad (6)$$

$$\varphi_1(z_1) = \varphi_2(z_1 - c) \quad (7)$$

$$\psi_1(z_1) = \psi_2(z_1 - c) + \bar{c}\varphi_2(z_1 - c) \quad (8)$$

The solving steps of the interactions are expressed in Appendix A. After the third iteration, the complex stress around section AT and FT-I is given as follows:

$$\begin{cases} \varphi_1(z_1) = \varphi_1^{(1)}(z_1) + \varphi_1^{(2)}(z_1) + \varphi_1^{(3)}(z_1) \\ \psi_1(z_1) = \psi_1^{(1)}(z_1) + \psi_1^{(2)}(z_1) + \psi_1^{(3)}(z_1) \\ \varphi_2(z_2) = \varphi_2^{(1)}(z_2) + \varphi_2^{(2)}(z_2) + \varphi_2^{(3)}(z_2) \\ \psi_2(z_2) = \psi_2^{(1)}(z_2) + \psi_2^{(2)}(z_2) + \psi_2^{(3)}(z_2) \end{cases} \quad (9)$$

In Equation (9), the explicit expressions of all the complex stress functions can be found in Equations (A1)–(A12). The stress component of any point in the surrounding rock is given by Equations (10) and (11) [32].

$$\sigma_\rho + \sigma_\theta = 4\text{Re}[\varphi_1'(z_1)] \quad (10)$$

$$\sigma_\theta - \sigma_\rho + 2i\tau_{\rho\theta} = 2e^{2i\theta}[\bar{z}\varphi_1''(z_1) + \psi_1'(z_1)] \quad (11)$$

where σ_θ , σ_ρ , and $\tau_{\rho\theta}$ are the radial stress, hoop stress, and shear stress along the tunnel outline, respectively.

4.4. Impact of Different Layer Thicknesses on the Stress of the Surrounding Rock

According to the solution process in the previous section, an analytical solution for the stress of the cross-sectional surrounding rock can be obtained by writing a computational program using MATLAB R2023b software. To determine a reasonable width between the cross-sections of AT and FT-I, calculations are performed for various values of d_2 , such as 0.6, 1.2, 1.8, ..., 5.4 m. Stress monitoring points are set on the contours of the two cross-sections, as shown in Figure 12. The monitoring points are located within the contour range near the interlaid rock. For the AT section, the monitoring points are analyzed within the range of θ_1 from -90° to $+90^\circ$, while for the FT-I section, monitoring points are selected within the range of θ_2 from 90° to 270° . Twenty monitoring points were taken from each section, with an angular difference of 9° between the two points.

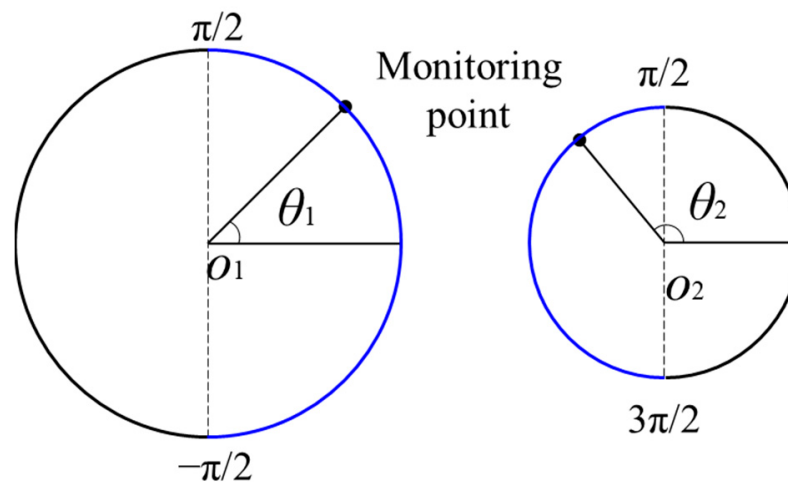


Figure 12. Diagram of the stress monitoring point on the section contour.

Figure 13 illustrates the radial stress variation curves of the monitoring points on the cross-sectional contours under different values of d_2 . Due to the relatively small difference in radii between the two concentric circles, the calculated results show a minor discrepancy in the radial stress experienced by the two cross-sectional contours, and their variation patterns are similar. The peak stresses on the surrounding rock occur in the crown and invert regions. In the crown shoulder region, the stress on the surrounding rock increases with the thickness of the interlaid rock. However, in the crown waist region, the stress on the surrounding rock decreases with an increase in the thickness of the interlaid rock. For the cross-section of AT, this variation occurs within the crown waist range of -30° to 30° , while for the cross-section of FT-I, it occurs within the crown waist range of 140° to 220° .

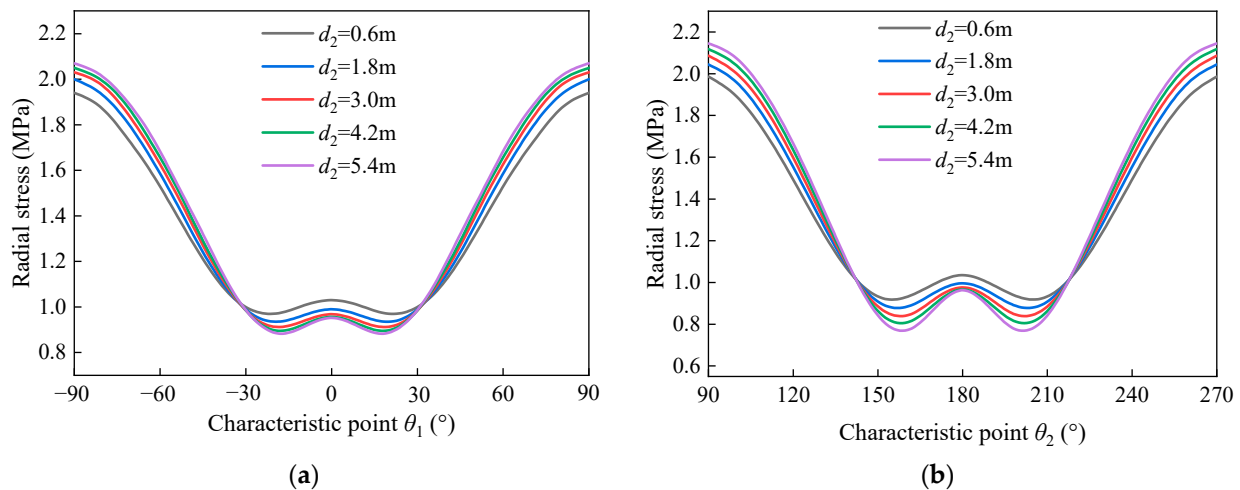


Figure 13. The radial stress variation curves of monitoring points on the cross-sectional contours. (a) AT section; (b) FT-I section.

Analyzing the data from the feature points, the curve depicting the variation in radial stress with the thickness of the interlaid rock is plotted within the crown waist range, as shown in Figure 14. From the graph, it can be observed that when d_2 is less than or equal to 3 m, the influence on radial stress is significant, and as the width decreases, the rate of stress increase becomes higher. Conversely, when d_2 is greater than 3 m, the stress variation around the tunnel becomes gradual and tends to reach equilibrium as the width decreases.

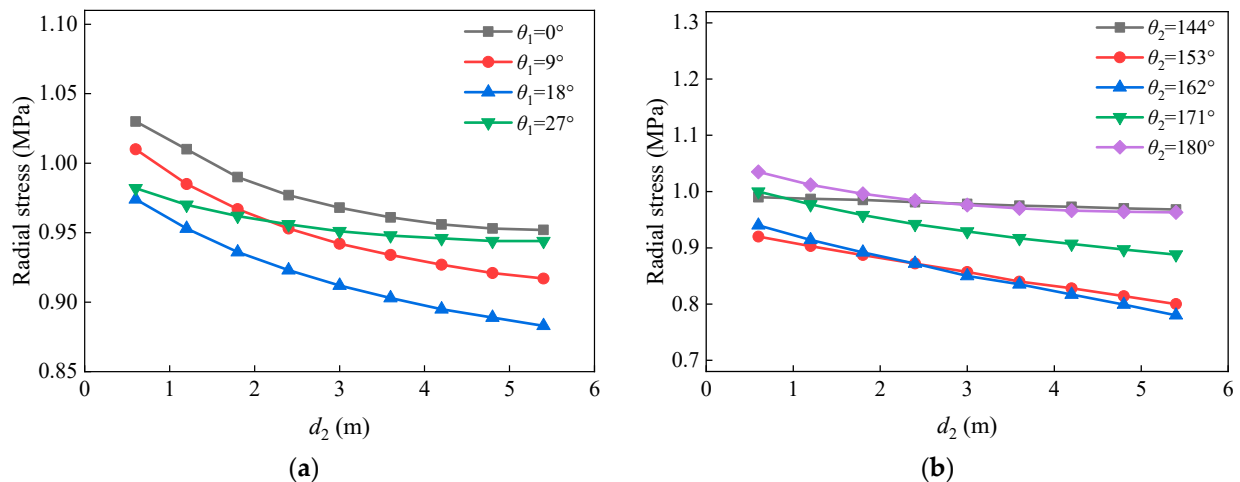


Figure 14. Variation curves of radial stress with different values of d_2 in the arch waist range. (a) AT section; (b) FT-I section.

Figure 15 illustrates the results of hoop stress at different values of d_2 . The results indicate that the peak hoop stress occurs at monitoring points with $\theta_1 = 0^\circ$ and $\theta_2 = 180^\circ$, which correspond to the closest points on the profiles of the two tunnels. The hoop stress around the AT decreases with an increase in d_2 , especially in the arch waist region where the influence of d_2 is more pronounced. Around the FT-I, in the arch shoulder region with $\theta_2 = 90^\circ \sim 150^\circ$ and $\theta_2 = 210^\circ \sim 270^\circ$, the hoop stress decreases with an increase in d_2 . However, in the range of $\theta_2 = 150^\circ \sim 210^\circ$, the hoop stress increases with an increase in d_2 . Analyzing the monitoring points within this range, the curves depicting the influence of different d_2 values on hoop stress are shown in Figure 16. From the graph, it can be observed that with an increase in d_2 , the impact on hoop stress is minimal, and the curve changes gradually. When d_2 is less than or equal to 3 m, the curve tends to reach equilibrium. Moreover, when d_2 is greater than 3 m, hoop stress shows a slow increasing trend.

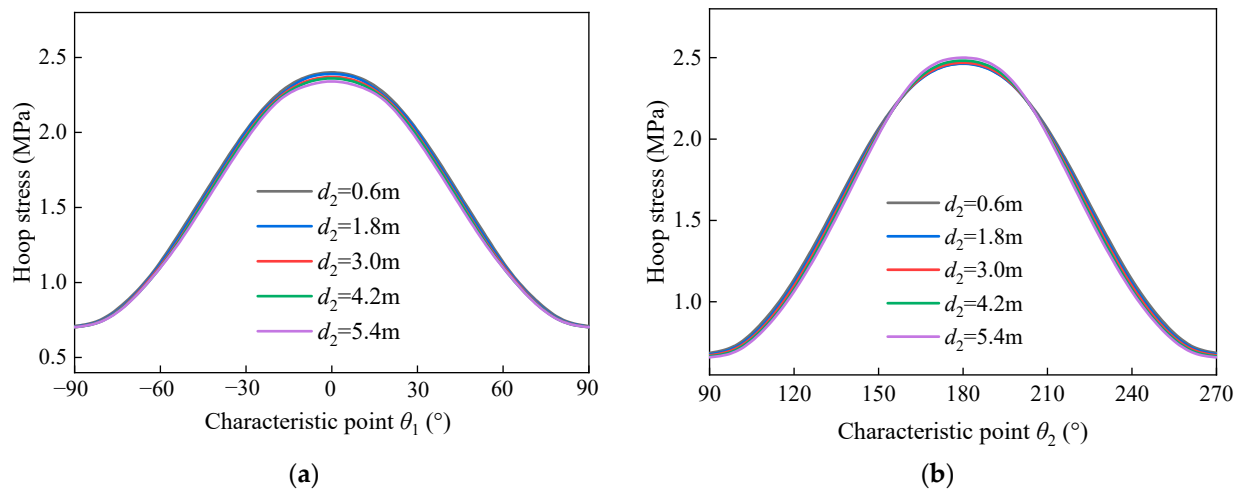


Figure 15. The hoop stress variation curves of monitoring points on the cross-sectional contours. (a) AT section; (b) FT-I section.

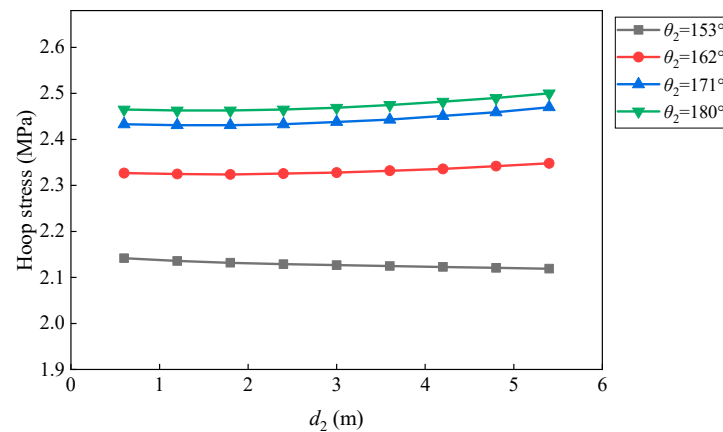


Figure 16. Variation curves of hoop stress with different values of d_2 in the FT-I arch waist range.

The results of shear stress around the cross-section contours are shown in Figure 17. In the bottom region of the arch waist, the shear stress is less affected by the thickness of d_2 . In the arch shoulder region, the shear stress around the cross-section decreases with an increase in d_2 , and both cross-sections exhibit this pattern.

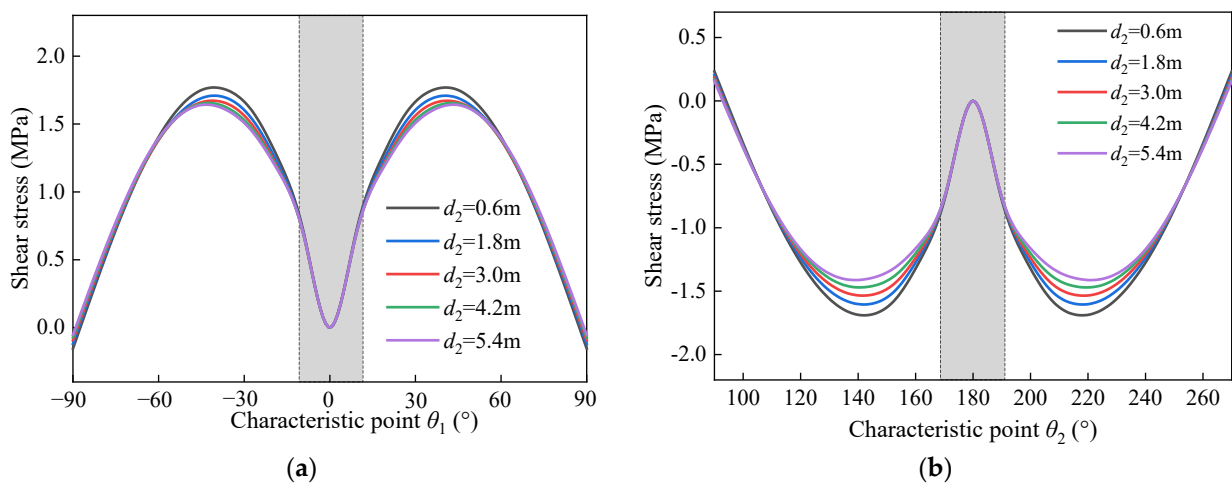


Figure 17. The shear stress variation curves of monitoring points on the cross-sectional contours. (a) AT section; (b) FT-I section.

In summary, based on the observed patterns and distribution characteristics of radial stress, hoop stress, and shear stress around the cross-sectional contours, along with the practical considerations of tunnel excavation, and setting the width of the FT-II (d_2) to 3 m, with an interlaid rock thickness of 3.5 m, results in a relatively uniform stress distribution. When d_2 is less than 3 m, there is a significant impact on stress variations in the arch waist region of both cross-sections. On the other hand, when d_2 exceeds 3 m, controlling rock damage during FT-II blasting becomes challenging, adversely affecting blasthole arrangement and on-site construction. Considering these factors, it is recommended to set the width of the FT-II (d_2) to 3 m as a safe spacing.

5. Excavation Partition Scheme

Based on the optimal solution of the width of the FT-II, the excavation partition scheme for the FT has been obtained (Figure 18). The FT is divided into two components with the thickness of the interlaid rock of 3.5 m as the boundary. The component of the width where the interlaid rock is greater than 3.5 m was classified as the general controlled-blasting area, using the benching tunnelling method. The other component with less than 3.5 m interlaid rock was classified as the key controlled-blasting area, using the four-part excavation method with a reserved vibration-cushioning layer. The parameters of the excavation partition scheme are shown in Figure 18. The width of the FT-II is 3 m. The shape of the FT-II section outline is irregular and resembles a “crescent”. After the excavation of the FT-I, the FT-II section has two free surfaces. However, under the blasting load from the FT-I, the rock mass of the FT-II section is relatively broken. So, the firing pattern and the construction of charge are the key points for peripheral blastholes of the FT-II blasting.

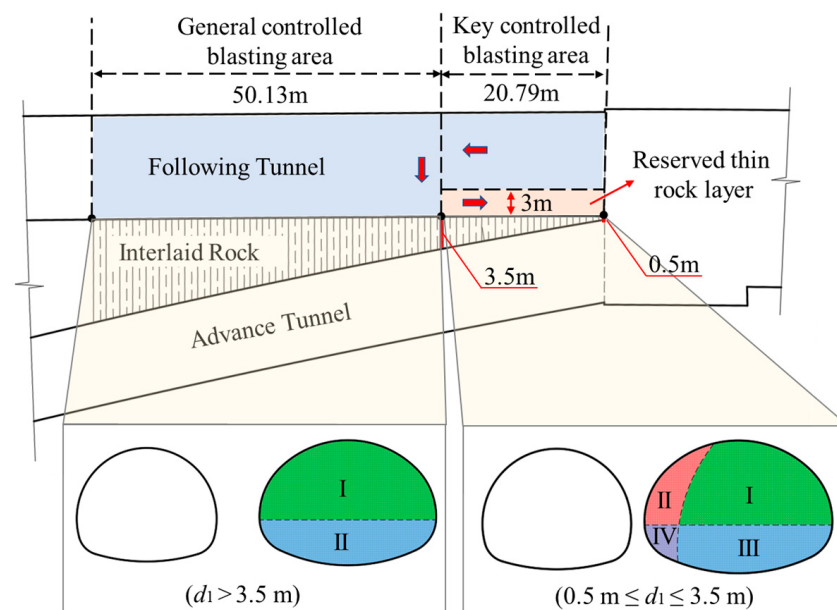
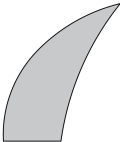


Figure 18. Excavation partition plan of the following tunnel.

The FT-I, situated away from the interlaid rock, is the initially excavated cross-section, presenting an irregular semi-circular shape. Subsequently, the FT-II, directly connected to the interlaid rock, is excavated, and this cross-section exhibits an irregular crescent shape. Their features and on-site excavation photos are presented in Table 4. The blasting excavation of these two cross-sections directly affects the stability of the interlaid rock. The blasting and excavation process should control the blasting vibrations to reduce damage to the surrounding rock. However, further discussion is needed to establish control indicators for blasting vibrations in these two cross-sections.

Table 4. Cross-sectional features and on-site excavation photos of the FT-I and FT-II.

Section Structure	Site Photo	Marking	Height (m)	Weight (m)	Feature
		FT-I	6.86	12.73	Semi-circular shape
		FT-II	6.63	3.00	Crescent shape

6. Discussion

The Chinese Blasting Safety Regulations (GB6722-2014) [33] stipulate that the allowable criteria for blast-induced ground vibration in traffic tunnels is 10 to 15 cm/s. The measured peak particle vibration velocities of the protected objects (i.e., vibration monitoring points) serve as the basis for safety assessment of blasting-induced vibrations. However, the safety allowable criteria for the blast-induced ground vibration in the neighborhood tunnel with small clearance are not explicitly defined. Table 5 lists some typical control criteria for blasting-induced ground vibrations in the neighborhood tunnel with small clearance in China. For the section with a 0.5 m-thick interlaid rock in the Liantang tunnel, considering the limited cases, it is more appropriate to strictly control the blasting-induced vibrations. According to the existing blasting-induced vibration control criteria, the stability of this vulnerable section with a 0.5 m thick interlaid rock cannot be adequately protected. Therefore, based on the actual conditions of the Liantang tunnel, the following blasting-induced vibration control criteria are proposed for different zones of the neighborhood tunnel.

(a) Blast-induced vibration control criteria for FT-I

For the FT-I blasting, there is a 3 m cushioning layer to mitigate the damage of blasting vibrations on the interlaid rock. Based on the on-site monitoring conditions, the safety control criteria for the FT-I blasting are set so that the peak particle vibration velocity at a point 20 m from the blasting source within the tunnel is less than 2.0 cm/s.

(b) Blast-induced vibration control criteria for FT-II

The FT-II section, directly connected to the interlaid rock, is the core and crucial area for blasting in the neighborhood tunnels. During the blasting process, vibration monitoring points are positioned in the AT, close to the blasting source, and aligned horizontally with the source. The distance between the monitoring points and the blasting source increases with the thickness of the interlaid rock. According to the gradual variation in the thickness, when the thickness exceeds 3.5 m, the peak vibration velocity at the control monitoring point (with a minimum distance of 6 m from the blasting source) is required to be less than 8.0 cm/s. When the thickness of the interlaid rock is between 0.5 m and 3.5 m, the peak vibration velocity at a point only 3 m away from the blasting source should be less than 3.0 cm/s. These control criteria are significantly higher than that of other small clearance tunnel projects.

Table 5. Blasting velocity control criteria for some typical neighborhood tunnels ¹.

No.	Name of Tunnel	Thickness of Interlaid Rock (m)	Vibration Control Criteria (cm/s)
1	New Kurutage tunnel [34]	15	10.7
2	Wutong mountain tunnel [35]	13.5	4.0
3	Maoding tunnel [36]	12	3.0
4	Xiaoyang mountain tunnel [37]	9.2	10
5	Damao mountain tunnel [3]	5.9	15
6	Jiaojin mountain tunnel [38]	5.0	10
7	Wulong tunnel [39]	4.0	25
8	Zhaobao mountain tunnel [40]	3.0	12

¹ Vibration monitoring points set up in the AT to close the explosion side, the length from the source of the explosion is equal to the thickness of interlaid rock.

7. Conclusions

This paper presents a case study of neighborhood tunnels with small clearance in Shenzhen, China, where the minimum thickness of interlaid rock is only 0.5 m. In this study, numerical simulation and theoretical analysis were carried out to analyze the cross-sectional structural characteristics of the neighborhood tunnels. We aimed to propose a safety excavation partition scheme for the following tunnel. The conclusions are as follows:

- (1) The damage mechanism of the interlaid rock under the blasting load from adjacent tunnels was investigated using LS-DYNA R11.1 software. The results suggest that meeting high-vibration control requirements for the upper and lower step zoning in the following tunnel is challenging, making it difficult to achieve damage control for the ultra-small clearance section for interlaid rock of 0.5 m thickness.
- (2) The four-part excavation method with reserved vibration-cushioning layer was proposed for the following tunnel. By incorporating a cushioning layer near the interlaid rock, direct damage from adjacent tunnel blasts was minimized. Based on the method of equivalent circles and the Schwarz Alternating Method (SAM), a mathematical model was established. By analyzing variations in surrounding rock stress with different thicknesses of the cushioning layer, the optimal thickness for the cushioning layer was determined to be 3.0 m.
- (3) Based on the parameters of the cushioning layer, the safe excavation partition scheme for the following tunnel was delineated. When the thickness of the interlaid rock exceeds 3.5 m, the benching method is employed for excavation. When the thickness of the interlaid rock ranges from 0.5 m to 3.5 m, the four-part excavation method with reserved vibration-cushioning layer is utilized.

As a result of this case study, suggestions were identified for the safe excavation of neighborhood tunnels with small clearance.

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Appendix A Solving Steps of the Interactions in Section 4.3

Step (a):

For the first iteration process, $\varphi(z)$ and $\psi(z)$ can be obtained by the Cauchy integral method. The calculation steps are as follows:

When $\sigma_\rho = P_1$, $\sigma_\theta = \tau_{\rho\theta} = 0$, the solutions are indicated by Equation (A1).

$$\begin{cases} \varphi_1(z) = \frac{P_1}{2} \left(\frac{z}{2} + \frac{R^2}{z} \right) \\ \psi_1(z) = -\frac{P_1}{2} \left(z + \frac{R^2}{z} - \frac{R^4}{z^3} \right) \end{cases} \quad (\text{A1})$$

When $\sigma_\theta = P_2$, $\sigma_\rho = \tau_{\rho\theta} = 0$, the solutions are indicated by Equation (A2).

$$\begin{cases} \varphi_2(z) = \frac{P_2}{2} \left(\frac{z}{2} - \frac{R^2}{z} \right) \\ \psi_2(z) = \frac{P_2}{2} \left(z - \frac{R^2}{z} - \frac{R^4}{z^3} \right) \end{cases} \quad (\text{A2})$$

Therefore, the solution for $\sigma_\rho = P_1$, $\sigma_\theta = P_2$, $\tau_{\rho\theta} = 0$ can be solved by superimposing Equations (A1) and (A2), which is the functional solution of $\varphi_1^{(1)}(z_1)$ and $\psi_1^{(1)}(z_1)$, which are given by

$$\begin{cases} \varphi_1^{(1)}(z_1) = \frac{P_A z_1}{2} - \frac{P_B R_1^2}{z_1} \\ \psi_1^{(1)}(z_1) = P_B z_1 - \frac{P_A R_1^2}{z_1} - \frac{P_B R_1^4}{z_1^3} \end{cases} \quad (\text{A3})$$

where P_A and P_B are related to P_1 and P_2 , $P_A = (P_1 + P_2)/2$, $P_B = (P_2 - P_1)/2$, respectively.

Step (b):

According to the coordinate transformation formula, the valuable z_1 is converted to z_2 to obtain the function of the complex stresses $\varphi_2^{(1)}(z_2)$ and $\psi_2^{(1)}(z_2)$ around the FT-I section when the AT section is excavated in the coordinate system $o_2x_2y_2$.

$$\begin{cases} \varphi_2^{(1)}(z_2) = \frac{P_A(z_2+c)}{2} - \frac{P_B R_1^2}{z_2+c} \\ \psi_2^{(1)}(z_2) = P_B(z_2+c) - \frac{P_A R_1^2}{z_2+c} - \frac{P_B R_1^4}{(z_2+c)^3} \end{cases} \quad (\text{A4})$$

Step (c):

In addition, the excess surface force $f_2^{(1)}$ generated around the FT-I section can be obtained, and follows the form indicated in Equation (A5).

$$f_2^{(1)} = \varphi_2^{(1)}(t_2) + t_2 \overline{\varphi_2^{(1)}(t_2)} + \overline{\psi_2^{(1)}(t_2)} \quad (\text{A5})$$

where t_2 represents the point on circle 2.

Apply the Cauchy operator to the boundary conditions Equation (A6), as follows:

$$\frac{1}{2\pi i} \int_{O1} \frac{f_2^{(1)}}{t_2 - z_2} dt_2 = \frac{1}{2\pi i} \int_{O1} \frac{\varphi_2^{(1)}(t_2)}{t_2 - z_2} dt_2 + \frac{1}{2\pi i} \int_{O1} \frac{t_2 \overline{\varphi_2^{(1)}(t_2)}}{t_2 - z_2} dt_2 + \frac{1}{2\pi i} \int_{O1} \frac{\overline{\psi_2^{(1)}(t_2)}}{t_2 - z_2} dt_2 \quad (\text{A6})$$

From the Cauchy integral formula, the analytical solution of circle 2, $\varphi_2^{(1)}(z_2)$, can be obtained by Equation (A7).

$$\frac{1}{2\pi i} \int_{O1} \frac{\overline{\psi_2^{(1)}(t_2)}}{t_2 - z_2} dt_2 = -\varphi_2^{(1)}(z_2) \quad (\text{A7})$$

To balance the surface force around the FT-I section, the surface force with opposite direction, $-f_2^{(1)}$, needs to be added. Under the balance external forces, the complex stress functions $\varphi_2^{(2)}(z_2)$ and $\psi_2^{(2)}(z_2)$ are indicated by Equation (A8), as follows:

$$\begin{cases} \varphi_2^{(2)}(z_2) = \frac{P_B R_1^2}{(\bar{c})^2} \left[\frac{-2R_2^6 - 3R_2^4 \bar{c} z_2}{\bar{c}(\bar{c} z_2 + R_2^2)^2} \right] - \frac{P_B R_2^2}{z_2} + \frac{P_A R_1^2 z_2}{R_2^2 + \bar{c} z_2} - \frac{P_B R_1^2 c z_2^2}{(R_2^2 + \bar{c} z_2)^2} + \frac{P_B R_1^4 z_2^3}{(R_2^2 + \bar{c} z_2)^3} \\ \psi_2^{(2)}(z_2) = -\frac{P_A R_2^2}{z_2} + \frac{P_A R_1^2 z_2}{R_2^2 + \bar{c} z_2} - \frac{P_B R_1^2 R_2^2}{z_2 c^2} - \left(\frac{R_2^2}{z_2} - \bar{c} \right) \varphi_2^{(1)'}(z_2) \end{cases} \quad (\text{A8})$$

Step (d):

The complex stress functions $\varphi_1^{(2)}(z_1)$ and $\psi_1^{(2)}(z_1)$ in step (d) can be obtained by the coordinate transformation.

$$\begin{cases} \varphi_1^{(2)}(z_1) = \varphi_2^{(2)}(z_1 - c) \\ \psi_1^{(2)}(z_1) = \psi_2^{(2)}(z_1 - c) - \bar{c} \varphi_2^{(2)'}(z_1 - c) \end{cases} \quad (\text{A9})$$

Thus, the first iterative process is complete. The complex stress around section AT and FT-I is given by

$$\begin{cases} \varphi_1(z_1) = \varphi_1^{(1)}(z_1) + \varphi_1^{(2)}(z_1) \\ \psi_1(z_1) = \psi_1^{(1)}(z_1) + \psi_1^{(2)}(z_1) \\ \varphi_2(z_2) = \varphi_2^{(1)}(z_2) + \varphi_2^{(2)}(z_2) \\ \psi_2(z_2) = \psi_2^{(1)}(z_2) + \psi_2^{(2)}(z_2) \end{cases} \quad (\text{A10})$$

Step (e):

Repeat the above steps to obtain all the complex stress solutions for the 3rd iteration. The complex stress functions $\varphi_1^{(3)}(z_1)$ and $\psi_1^{(3)}(z_1)$ in step (e) can be expressed as follows:

$$\begin{cases} \varphi_1^{(3)}(z_1) = \frac{P_A R_2^2 z_1}{E} - \frac{P_B R_1^2}{F} + \frac{P_B R_2^2 R_2^2}{(\bar{c})^2 E} + F \left[\frac{P_B R_1^2}{(\bar{c})^2} \left(\frac{3R_2^4}{c^2 G^2} - \frac{2R_2^6}{c^3 G^3} \right) \right. \\ \quad \left. + \frac{P_B R_2^2}{E^2} + \frac{P_A R_1^2 R_2^2}{E^2 F^2} + \frac{3P_B R_1^4 R_2^2}{E^2 F^4} - \frac{2\bar{c} P_B R_1^2 R_2^2}{E^2 F^3} \right] - \frac{3P_B R_2^2 H^2}{c^4 R_1^2} \left(\frac{H^3}{K^2} + \frac{2H^2}{K} \right) \\ \quad - \frac{2P_B R_2^2 H^2}{c^5 R_1^4} \left(\frac{H^4}{K^3} + \frac{4H^3}{K^2} + \frac{6H}{K} \right) - \frac{P_B R_2^2}{(\bar{c})^2} \left(\frac{R_1^6}{(\bar{c})^3 E^2} + \frac{3R_1^4}{(\bar{c})^2 E} \right) \\ \quad - \frac{P_A R_2^2 H^2}{c^2 R_1^2} \left(\frac{H^3}{K^2} + \frac{3H^2}{K} \right) + \frac{3P_B R_2^2 H^4}{c^4 R_1^4} \left((\bar{c})^2 z_1 - \frac{z_1^3 (R_1^2 - \bar{c} z_1)^2}{K^4} \right) \\ \quad + \frac{2\bar{c} P_B R_2^2 H^3}{c^3 R_1^4} \left(\frac{\bar{c} z_1^4 - R_1^3 z_1^3}{K^3} - \bar{c} z_1 \right) \\ \psi_1^{(3)}(z_1) = \frac{P_B R_1^2}{(\bar{c})^2} \left(\frac{3R_2^4}{c^2 G^2} - \frac{3R_2^6}{c^3 G^3} \right) + \frac{P_B R_2^2}{E} - \frac{P_A R_1^2}{F} + \frac{P_B R_1^4}{F^3} \\ \quad + \frac{P_B \bar{c} R_1^2}{F^2} - \frac{R_1^2}{z_1} \varphi_1^{(2)'}(0) - \frac{R_1^2}{z_1} \varphi_1^{(3)'}(z_1) \end{cases} \quad (\text{A11})$$

where $E = \frac{R_1^2}{z_1} - \bar{c}$, $F = \frac{R_2^2 z_1}{R_1^2 - z_1 \bar{c}} + c$, $G = \frac{R_1^2}{z_1} - \bar{c} + \frac{R_2^2}{c}$, $H = \frac{R_1^2 c}{c\bar{c} - R_2^2}$, and $K = z_1 - \frac{R_1^2 c}{c\bar{c} - R_2^2}$, respectively.

Step (f):

The complex stress functions $\varphi_2^{(3)}(z_2)$ and $\psi_2^{(3)}(z_2)$ in step (f) can be obtained by the coordinate transformation.

$$\begin{cases} \varphi_2^{(3)}(z_2) = \varphi_1^{(3)}(z_2 + c) \\ \psi_2^{(3)}(z_2) = \psi_1^{(3)}(z_2 + c) - \bar{c}\varphi_1^{(3)'}(z_2 + c) \end{cases} \quad (\text{A12})$$

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