

Article

Study on Optimization of Construction Parameters and Schemes for Complex Connecting Tunnels of Extra-Long Highway Tunnels Based on Field Monitoring and Numerical Simulation

Shaohui He ¹ , Jiaxuan Liu ¹, Dawei Huang ^{2,*} and Jianfei Ma ¹

¹ School of Civil Engineering, Beijing Jiaotong University, Beijing 100044, China; shhhe@bjtu.edu.cn (S.H.); 24110318@bjtu.edu.cn (J.L.); 22110333@bjtu.edu.cn (J.M.)

² State Key Laboratory of Safety and Resilience of Civil Engineering in Mountain Area, East China Jiaotong University, Nanchang 330013, China

* Correspondence: 2917@ecjtu.edu.cn

Abstract

To study the optimization of construction parameters and schemes for complex connecting tunnels in extra-long highway tunnels in granite strata, the research team, relying on the construction project of the complex connecting tunnel between the Xiaolongmen Extra-long Highway Tunnel and the ultra-deep shaft, established an on-site monitoring scheme and a refined numerical simulation model. It systematically analyzed the impact of various construction parameters on the construction process of connecting tunnels and the main tunnel, and on this basis, optimized the construction scheme, improving construction efficiency. The research results show that (1) after the excavation of the connecting tunnel, the confining pressure at the top of the working face decreases rapidly, while the confining pressure on both sides increases rapidly; the extreme point of the confining pressure decrease is located at the central point at the top of the excavated working face. (2) For Class III surrounding rock excavated using the full-face blasting method, the maximum influence range of working face excavation on the stratum along the tunneling direction is approximately 4D (where D represents the excavation step). (3) The larger the excavation step of the connecting tunnel, the more obvious the stress concentration phenomenon at the central point of the working face arch crown, and the excavation step should be optimally controlled within the range of 2–3 m. (4) When explosives in the blast hole adopt decoupled charging, the ratio of borehole diameter to charge diameter can be increased to utilize the air gap to buffer the energy generated by the explosion.

Keywords: connection channel excavation; construction parameter optimization; excavation step distance; blasting analysis; field monitoring; tunnel safety; MIDAS GTS NX; blasting vibration



Academic Editor: Francesca Dezi

Received: 23 June 2025

Revised: 13 July 2025

Accepted: 25 July 2025

Published: 26 July 2025

Citation: He, S.; Liu, J.; Huang, D.; Ma, J. Study on Optimization of Construction Parameters and Schemes for Complex Connecting Tunnels of Extra-Long Highway Tunnels Based on Field Monitoring and Numerical Simulation. *Infrastructures* **2025**, *10*, 197. <https://doi.org/10.3390/infrastructures10080197>

Copyright: © 2025 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the Creative Commons Attribution (CC BY) license (<https://creativecommons.org/licenses/by/4.0/>).

1. Introduction

With the development of tunnel construction engineering technology, extra-long tunnels have been widely applied in highway transportation systems [1–3]. As an important component of the overall design of extra-long highway tunnels, the tunnel ventilation system generally adopts the longitudinal ventilation mode with air shafts for supply and exhaust when the total tunnel length exceeds 5000 m [4]. As a key facility connecting the tunnel and the ventilation shaft, the connecting channel has been the subject of extensive research by relevant scholars. This is aimed at reducing the impact of connecting channel

construction on the tunnel and surrounding strata, ensuring construction safety, and improving construction efficiency.

When the geological conditions are favorable, the connection channel is mostly excavated using the drill-blast method. With the characteristics of simple construction and strong adaptability, the drill-blast method has been widely applied in such fields as underground mineral mining and rock tunnel excavation [5–8]. Yang et al. [9] optimized the blasting construction scheme and analyzed the impact of different blasting positions on the tunnel in response to the adverse effects of Connection Channel blasting construction on the tunnel structure. Bai et al. [10] used FLAC3D and DEM methods, respectively, to simulate the tunnel blasting excavation process and evaluated the high-risk construction sections of the tunnel. Xie et al. [11] carried out phase space reconstruction on the vibration response time series of existing tunnels under the influence of different blasting actions based on chaos theory and analyzed the chaotic characteristics of tunnel blasting vibration response signals. Zhou et al. [12] conducted extensive investigations into the statistical data of working face collapse accidents in rock tunnels using the drill-blast method from the literature, providing new theoretical foundations and technical means for safety risk analysis and control in rock tunnel construction. However, most existing studies on drilling and blasting excavation directly apply the blasting parameters of the main tunnel, ignoring the unique structural characteristics of connecting tunnels, such as “small cross-section, large curvature, and strong constraint”. This leads to deviations of designs (including charge amount and hole network parameters) from actual needs. Meanwhile, on-site blasting monitoring data are mostly used to verify model results and rarely used for dynamically revising blasting parameters, resulting in insufficient practicality of optimization schemes.

In addition, the drill-blast excavation methods for tunnels and connection channels can also be divided into full-face excavation, bench method excavation, etc. [13–16], and the connection channel causes significant disturbance to the surrounding strata during the excavation process. To minimize the impact of the construction process on adjacent buildings and the ground surface of the strata, relevant scholars have carried out extensive research on the stability of surrounding rock [17–19] and on this basis, conducted studies on the optimization of construction parameters for tunnels and connection channels: Zhou et al. [20] optimized the slurry ratio for freezing hole drilling under water-rich sandy cobble strata conditions using single-factor tests and orthogonal tests. He et al. [21,22] analyzed the impact of different construction parameters on the construction process based on indoor tunneling tests. Xie et al. [23], taking practical projects as examples, carried out optimization analysis of construction parameters using FLAC3D software (FLAC3D version 5.0.) through test correlation and field monitoring results. Wei et al. [24] established the correlation between tunneling construction parameters and soil displacement and further revealed the influence of the construction process on the development law of soil displacement. But existing studies on construction parameters of connecting tunnels mainly focus on the shield tunnel field, and there is basically no systematic analysis on construction parameters of connecting tunnels in extra-long highway tunnels. Therefore, relevant personnel mainly rely on empirical methods for parameter design in the construction of complex connecting tunnels in extra-long highway tunnels, which makes it difficult to guarantee engineering quality and results in relatively low construction efficiency.

In summary, existing scholars’ research on the control of construction parameters for tunnels and connecting tunnels is relatively limited in scope. Meanwhile, there are few studies on the construction parameters of highway connecting tunnels [25–27], and the influence characteristics of connecting tunnel construction parameters on surrounding strata and existing structures have not been systematically analyzed. Relevant research results and conclusions have limited guiding significance for construction projects involving

complex connecting tunnels in extra-long highway tunnels. Based on existing research, this paper investigates the impact of different construction parameter controls on existing main tunnels during the construction of connecting tunnels through methods such as on-site monitoring and numerical simulation. It conducts in-depth analysis on the excavation step and blasting parameters of connecting tunnels, optimizes construction excavation methods and schemes, and thus achieves the safe and efficient construction of connecting tunnels between extra-long highway tunnels and complex air shaft systems.

2. General Description of the Project

2.1. Project Background

This project is based on the Xiaolongmen Extra-long Tunnel of the Beijing–Zhangjiakou Expressway in China. The tunnel has a total length of 6.4 km and adopts a longitudinal segmented ventilation design. To meet the requirements of tunnel ventilation and smoke exhaust under fire emergency conditions, a ventilation shaft needs to be set up at about 95 m to the right of K1+362.013 in the right tunnel. Underground supply and exhaust air connecting tunnels should be constructed to connect with the main tunnel. The right tunnel is equipped with one supply air duct and one exhaust air duct, respectively, while the left tunnel is only provided with an exhaust air duct. The positional relationship of relevant underground facilities is shown in Figure 1. The total length of the underground supply and exhaust air connecting tunnels connecting with the main tunnel is 312.323 m. The underground connecting tunnel has 2 intersections and 2 transition widening sections. Since the connecting tunnel cannot be constructed in parallel, the construction period is long, and the construction technology and procedures are complex.

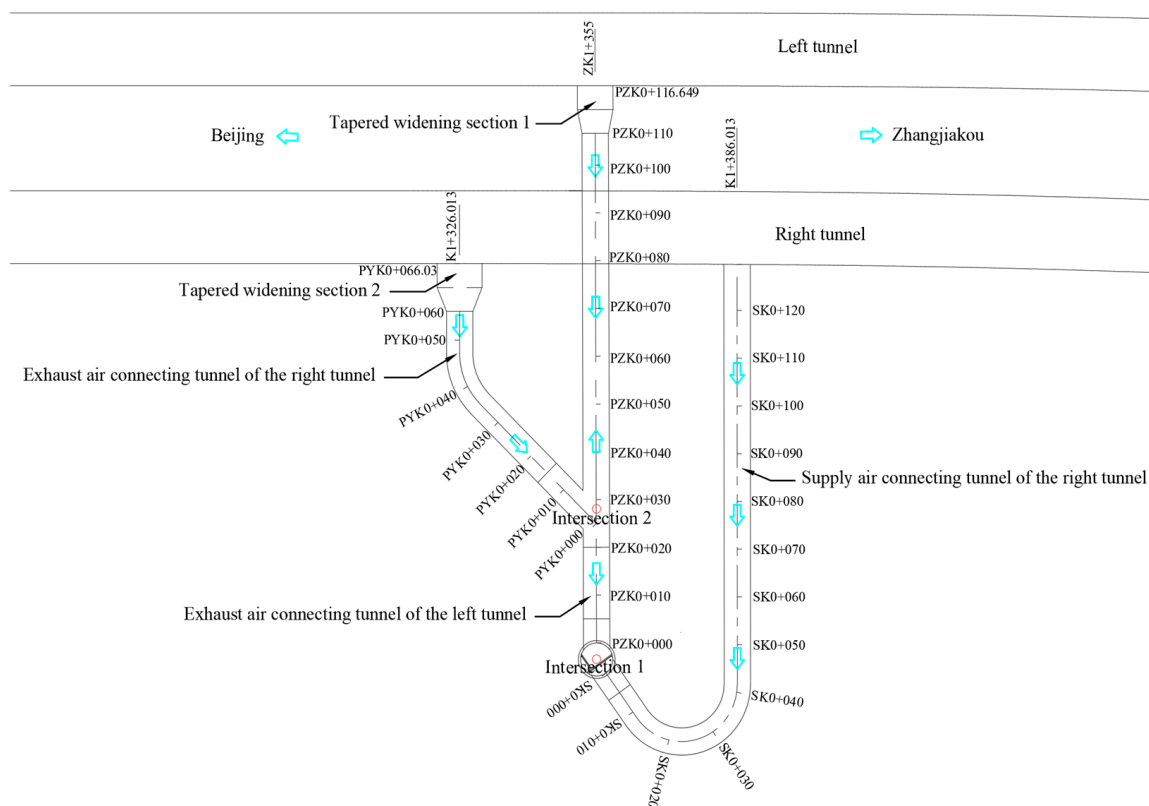


Figure 1. Position plan of the main tunnel and connecting tunnels.

To shorten the construction period, the construction plan on site specifies that the main tunnel and the ventilation shaft should be excavated simultaneously first. After the

construction of the main tunnel is completed, the lining support at the corresponding position inside the main tunnel will be dismantled for the construction of connecting tunnels. Specifically, the exhaust and supply air connecting tunnels of the right tunnel will be constructed simultaneously first. After the excavation of the right tunnel's exhaust air connecting tunnel is completed, the excavation of the left tunnel's exhaust air connecting tunnel will be carried out from both ends of Intersection 2. Therefore, the construction process of connecting tunnels has a significant impact on stratum disturbance and the main tunnel. Furthermore, during the construction of the left tunnel's exhaust air connecting tunnel, it needs to cross above the right tunnel with a minimum clear distance of only about 5 m. The disturbance control of the main tunnel lining during the construction of the overcrossing section is also a key technical challenge in this project.

2.2. Field Monitoring Deployment

To ensure construction safety, the stress variation in the surrounding rock around the connecting tunnels and concrete strain should be monitored during the construction. One test section is, respectively, selected in the exhaust air connecting tunnel and supply air connecting tunnel of the right tunnel, namely the right exhaust test section and right supply test section. The mileage post of the right exhaust test section is PYK0+060, as shown in Figure 2a. A total of seven measuring points are selected in this section, with an interval of 25 degrees between adjacent points. Each measuring point is equipped with one double membrane pressure cell and two concrete strain gauges, which are used to monitor the stress variation in the surrounding rock of the connecting tunnel and the strain of the primary lining concrete, respectively. The mileage post of the right supply test section is SK0+120. Similarly, seven measuring points are selected with an interval of 20° between adjacent points. The installation rules for double membrane pressure cells and concrete strain gauges are the same as those for the previously described test section (as shown in Figure 2b). As shown in Figure 3, the double membrane pressure cells are tightly attached to the surrounding rock and fixed with steel brackets to monitor the rock pressure. The concrete strain gauges are arranged in a cross shape and fixed on the steel reinforcement mesh to monitor the concrete strain in different directions. The measuring range of the pressure cells used is 1.5 MPa, and the tensile and compressive strain ranges of the concrete strain gauges are 1200 $\mu\epsilon$ and 800 $\mu\epsilon$, respectively.

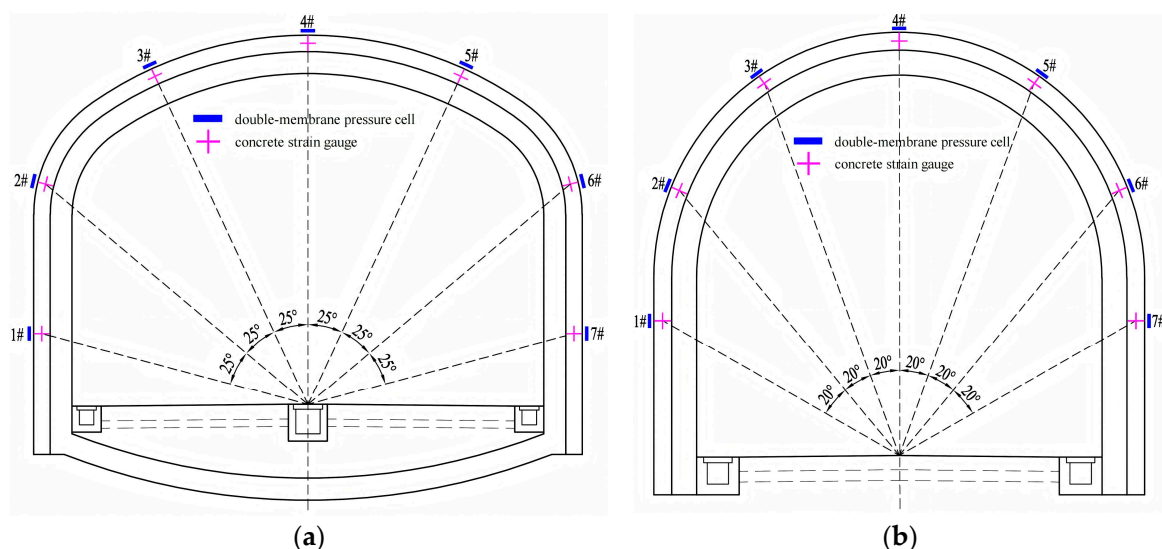


Figure 2. Arrangement of monitoring elements within the test section: (a) right exhaust test section; (b) right supply test section.

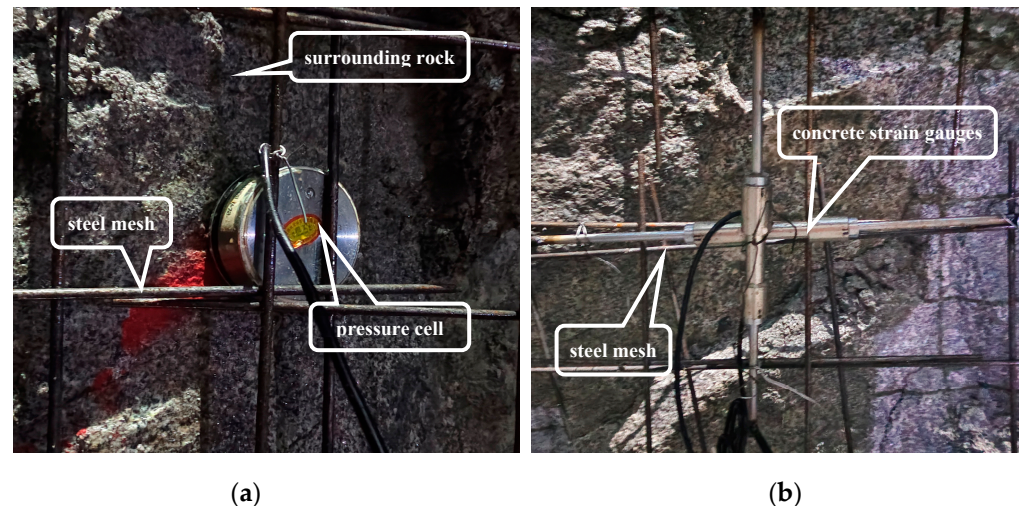


Figure 3. Installation of monitoring sensors: (a) double membrane pressure cell; (b) concrete strain gauge.

2.3. Mechanical Properties Test of Granite

According to the engineering geological survey and drilling results, the lithology of the stratum where the main tunnel and each exhaust air connecting tunnel are located is mainly Yanshanian granodiorite. This rock stratum has a medium-coarse-grained structure and massive tectonics, with well-developed joint fissures. Its main components are feldspar, quartz, and hornblende. The core is columnar or long columnar, with joint lengths of 15–30 cm and a maximum length of about 50 cm, locally fragmented. The characteristic value of stratum bearing capacity is $f_{a0} = 1800$ kPa, the surrounding rock grade is Grade III, and the surrounding rock has good stability. Furthermore, the connecting tunnel is located at a stratum depth of approximately 400 m, with the lateral pressure coefficient K_0 usually ranging from 0.5 to 2.0, and the surrounding rock pressure of the stratum around the connecting tunnel being approximately 8–15 MPa.

To analyze the failure modes of the surrounding rock during the excavation of the connecting tunnel, a granite triaxial compression test was conducted (as shown in Figure 4). The relevant test results can also provide a theoretical basis for subsequent numerical modeling. After drilling and sampling rock layers at the construction site, standard cylindrical granite specimens with a height of 100 mm and a diameter of 50 mm were prepared. After the test, relevant data were extracted to plot the stress–strain curves of the granite specimens under indoor triaxial compression tests. Based on the test results, the specimen mainly exhibited elastic deformation before compressive yielding, and its yield strength was close to the ultimate strength. When the confining pressure was within 5 MPa, the specimen showed a brief steep stress drop immediately after reaching the ultimate strength, indicating brittle failure at this stage. Thereafter, the stress–strain curve changed in a gentler trend until another steep stress drop occurred, leading to complete failure. When the confining pressure is above 7.5 MPa, the specimen does not exhibit a particularly significant steep stress drop after reaching the ultimate strength. The stress–strain curve changes gently, and the specimen gradually softens. In this project, the stratum where the main tunnel and connecting tunnels are located is relatively deep, with a large surrounding confining pressure (above 10 MPa). When the confining pressure is greater than 10 MPa, the yield strength of the surrounding rock should be greater than 240 MPa, indicating good stability of the surrounding rock.

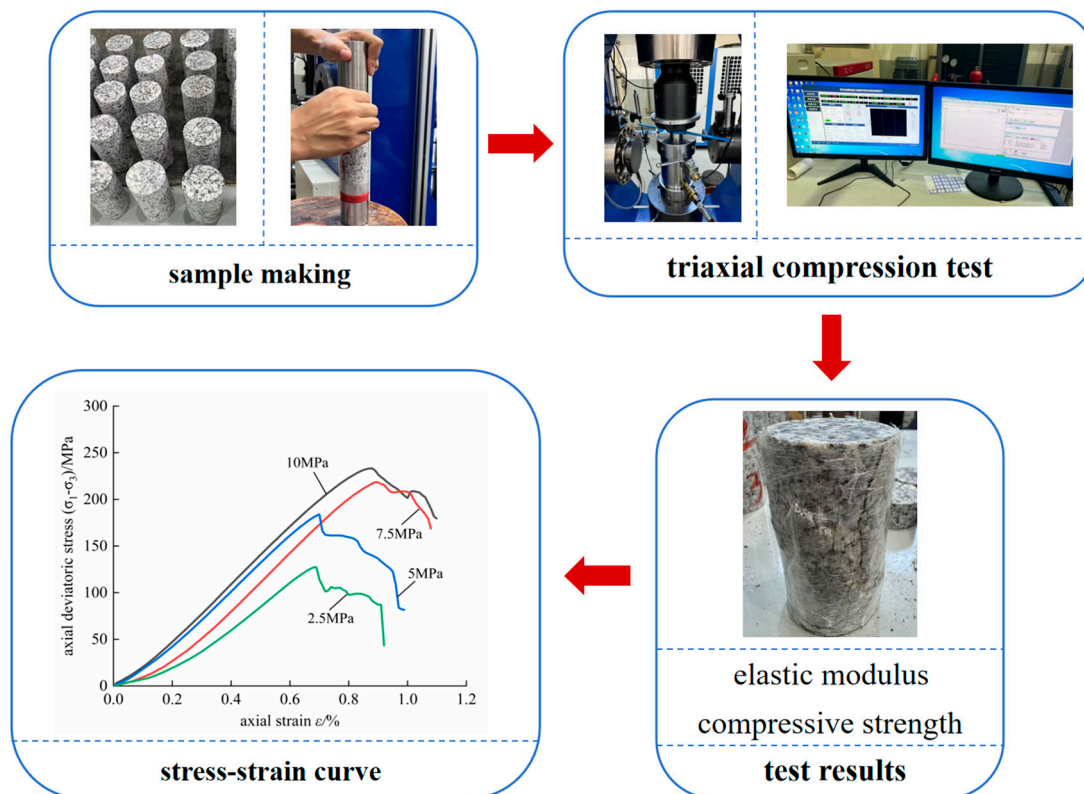


Figure 4. Triaxial compression test of granite.

3. Numerical Simulation Analysis

3.1. Establishment of a Computational Model

Based on the geological survey results of the construction site, the surrounding rock grade of the stratum in the construction area is Grade III, indicating the good self-stability of the surrounding rock. As shown in Figure 5, the MIDAS GTS NX software (is used to establish a 1:1 numerical simulation model (MIDAS GTS NX 2022 R1 (x64)). To maximize the reproduction of on-site excavation and avoid the influence of stratum boundary effects, the computational model includes the tunnel, shaft, and connecting tunnels, with length $\times$ width $\times$ height dimensions being 210 m $\times$ 140 m $\times$ 400 m. During the numerical simulation of on-site excavation, the connecting tunnels are mainly excavated using the full-face drill-and-blast method with an advancement of 2 m per round. To minimize the disturbance to the surrounding stratum and adjacent main tunnel during the construction of connecting tunnels, the widened transition sections and overcrossing sections are excavated using the bench cut method with an advancement of 0.6 m per round.

The longitudinal direction of the tunnel is set as the X-axis, the direction perpendicular to the tunnel's longitudinal axis as the Y-axis, and the gravity direction as the Z-axis. For mesh control of solid elements, the seeding number for the tunnel, shaft, and connecting tunnels is 1.5, meaning there is a node every 1.5 m; the seeding number for the stratum is 4, indicating a node every 4 m. The setting of model boundaries should follow the principles of geotechnical engineering boundary conditions. The bottom and sides of the model are set as fixed constraints and normal constraints, respectively, while the top is set as a free surface. Both the surrounding rock and lining are simulated using homogeneous hexahedral elements, and the surrounding rock follows the Mohr–Coulomb criterion. In the actual construction process, the construction unit had completed the construction of the main tunnel before carrying out the excavation of the shaft and connecting tunnel. Therefore, in the numerical simulation, it is necessary to first complete the excavation of

the main tunnel and set the displacement to zero. The numerical simulation parameters for the existing tunnels are determined according to the on-site construction drawings. The calculation parameters for connecting tunnels and shafts are selected and determined based on the on-site geological survey report and the Guidelines for Design of Highway Tunnels (JTG/T D70-2010) [28]. The material properties of the stratum and connecting tunnels are shown in Table 1.

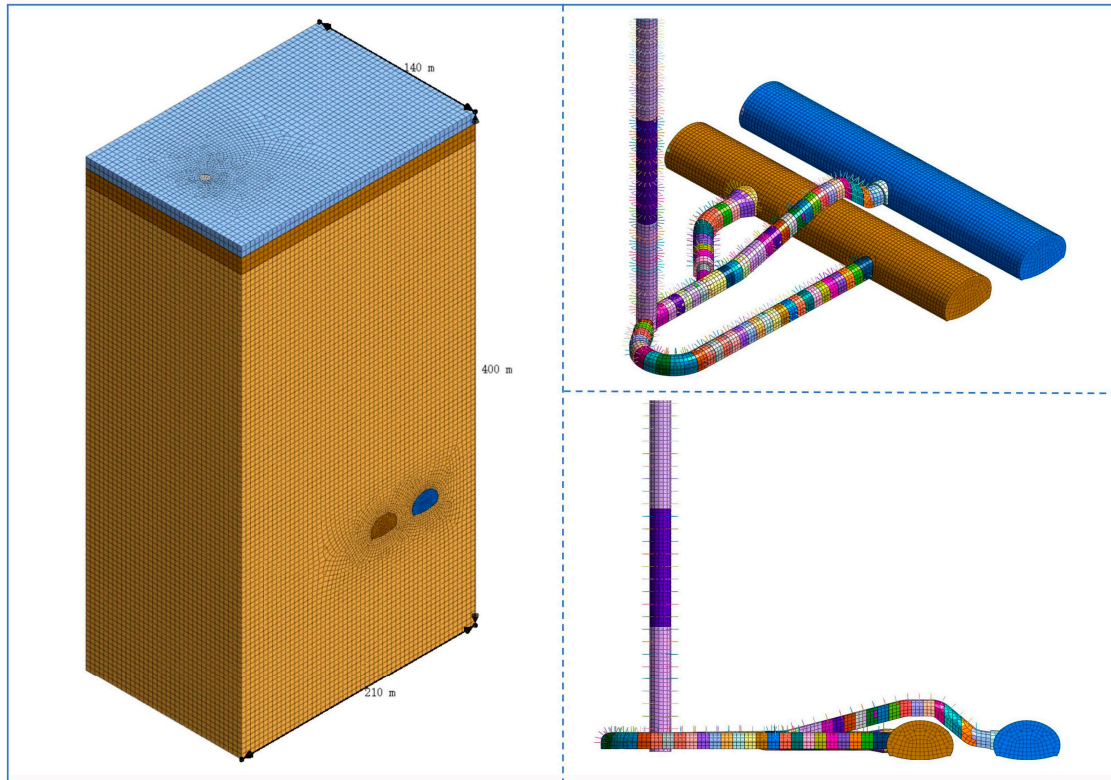


Figure 5. Numerical simulation model.

Table 1. Model material properties.

Material	C (kPa)	Φ (°)	Unit Weight (kN·m ³)	Elastic Modulus (MPa)	Poisson's Ratio
Completely Weathered Rock	160	35.0	18.2	3250	0.33
Highly Weathered Rock	690	39.3	22.8	8680	0.30
Moderately Weathered Rock	1620	51.5	26.2	29,130	0.26
Shotcrete	—	—	24	15,000	0.2
Anchor	—	—	78.5	210,000	0.3

3.2. Verification of Model Result Rationality

To verify the accuracy of numerical simulation results, comparative analysis was conducted between field monitoring results and numerical simulation results under the same working condition. Taking the surrounding rock stress measured in the first field monitoring as the initial value (Figure 6 shows the numerical simulation results of surrounding rock stress under the corresponding working condition), the variation values of surrounding rock stress at selected measuring points (Points 3#, 4#, and 5#) on the right exhaust test section and right supply test section were compared with the corresponding numerical simulation results. As shown in Figure 7, after the variation pattern of field

monitoring data became relatively stable, the measured and simulated results of 9 different construction stages were, respectively, extracted from the two test sections and plotted into curves.

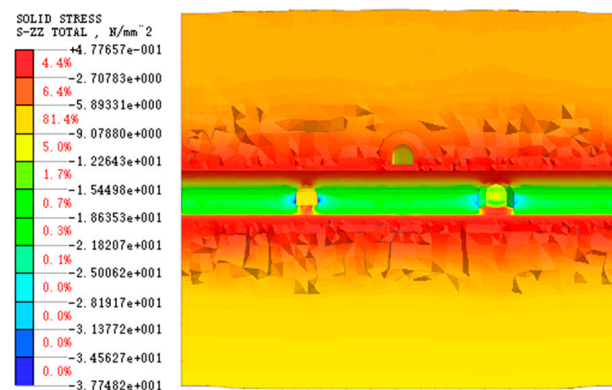


Figure 6. Distribution of vertical stress in the surrounding rock around the connecting tunnel under the corresponding working condition.

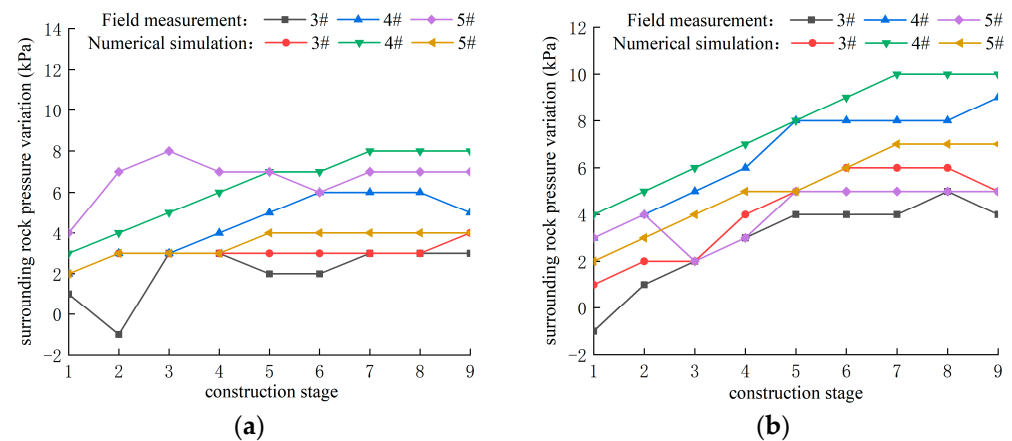


Figure 7. Results of surrounding rock stress in test sections under different construction stages: (a) right exhaust test section; (b) right supply test section.

According to Figure 7a, the numerical simulation results of measuring Points 3# and 4# in the right exhaust test section are generally slightly higher than the field measurement results. The data curves of the measured and simulated results have a high fitting degree and similar variation patterns. Except for individual measuring points, the error of the measuring points is basically controlled within about 15%. The field measurement results of Measuring Point 5# are generally higher than the simulation results. Among them, the curve of the field measurement results has larger fluctuations during the initial construction stage, and there is a certain deviation between the measured and simulated results, but the final variation trends are the same. As shown in Figure 7b, the numerical simulation results of surrounding rock pressure changes at each selected measuring point in the Right Supply Test Section are generally slightly higher than the field measurement results. Compared with the Right Exhaust Test Section, the data curves of measured and simulated results in the Right Supply Test Section have a higher fitting degree and partially overlap. Based on the comprehensive analysis of the above data, the accuracy of this numerical simulation can be fully verified.

4. Optimization of Construction Sequence Parameters

4.1. Optimization of Excavation Step Distance for Section Excavation

To explore the optimal excavation step distance during construction, three step distances of 2 m, 3 m, and 4 m were set based on the on-site construction design for numerical simulation analysis. The simulated excavation sequence of the connecting tunnels is consistent with the actual construction. The right exhaust test section and right supply test section were selected as representative sections. Figures 8 and 9 show the surrounding rock stress variation results at each measuring point of the corresponding sections under different construction step distances. Analysis of the data curves reveals that after the excavation of the connecting tunnels, the surrounding rock stress at the top of the test section decreases rapidly, with the extreme point of stress reduction located at the center of the section top. The surrounding rock stress on both sides increases rapidly, and the stress variation on both sides exhibits non-uniform distribution. When the excavation step distance is 2 m, the surrounding rock stress distribution at the top of the section is relatively uniform. The smaller the excavation step distance, the more uniform the stress distribution at the top of the test section. As the excavation step distance increases, stress concentration occurs at the central position of the section top. The larger the excavation step distance, the greater the extreme value of surrounding rock stress variation at the central top position. For the simulated construction with the three set excavation step distances, the surrounding rock stress shows larger fluctuations at the initial stage of section excavation. The surrounding rock stress basically remains unchanged after 4 excavation rounds. When the connecting tunnel section is excavated, the maximum influence range on the stratum along the tunneling direction is about 4D (D represents the excavation step distance).

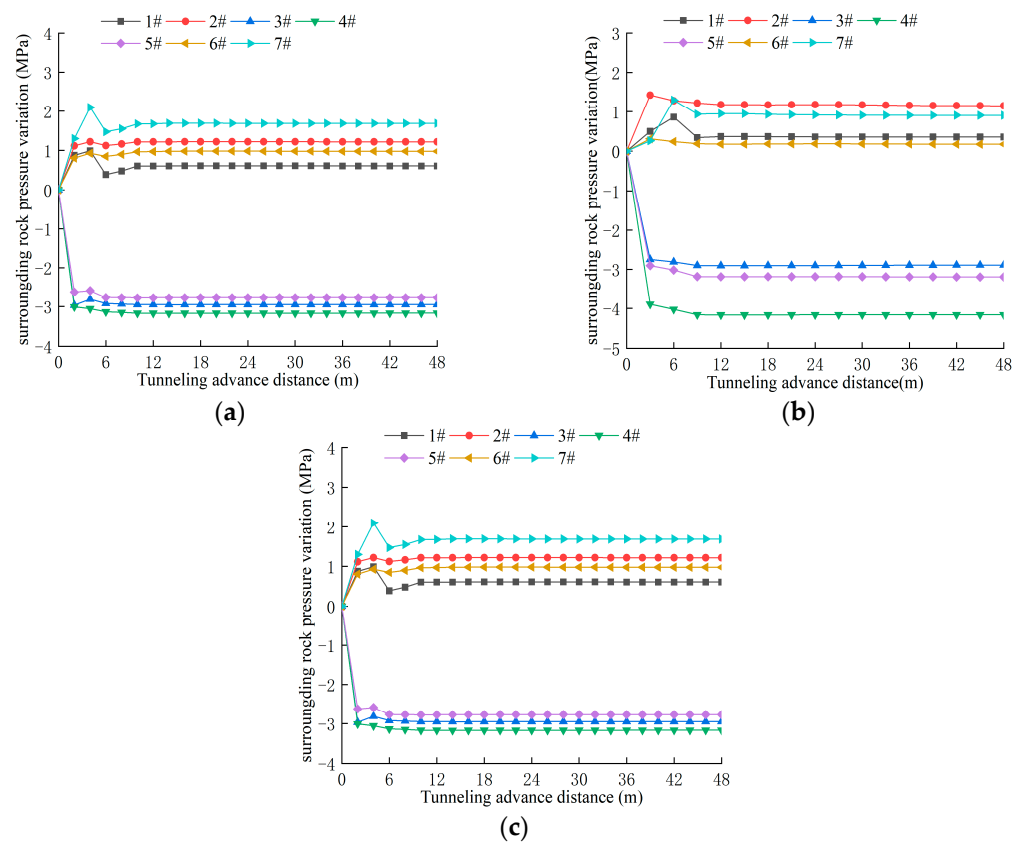


Figure 8. Stress variation in surrounding rock around the right exhaust test section under different excavation step distances: (a) 2 m; (b) 3 m; (c) 4 m.

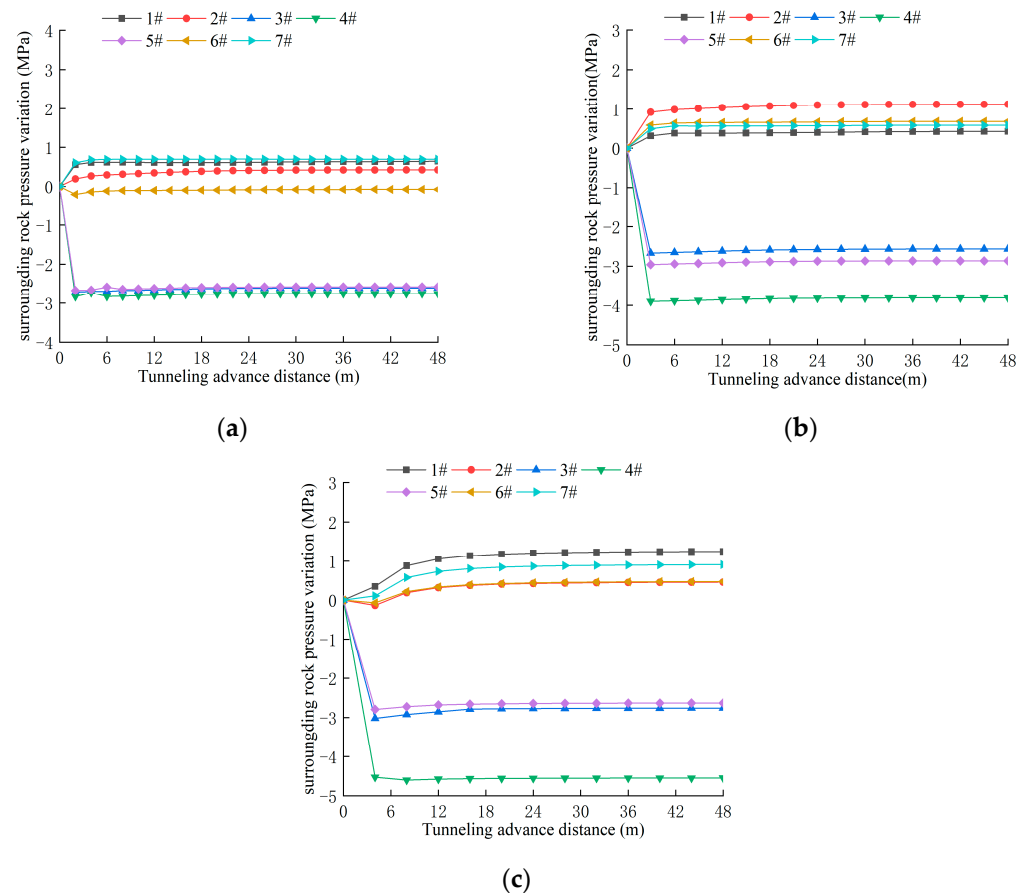


Figure 9. Stress variation in surrounding rock around the right supply test section under different excavation step distances: (a) 2 m; (b) 3 m; (c) 4 m.

As shown in Figures 8 and 9, the variation patterns of surrounding rock stress around the right exhaust test section and right supply test section are similar during the construction and excavation process. Therefore, the right exhaust test section is selected as the representative section. As shown in Figure 10, the deformation development process of the primary lining concrete during numerical simulation excavation of the section under different working conditions is presented. Combined with the results of surrounding rock stress variation, the deformation of the primary lining concrete after the excavation of connecting tunnels can be roughly divided into three stages.

In the first stage, the primary lining concrete has not fully solidified to develop effective strength, so the deformation is relatively large. When the top of the lining is under pressure, the vertical inner diameter decreases while the horizontal inner diameter increases. Thus, at the initial stage of section excavation, the stress at the measuring points on the lining top decreases, and the stress at the measuring points on both sides increases. During the second stage, the primary lining concrete starts to consolidate and shrink. The top of the lining continues to undergo settlement deformation under pressure, while the concrete on both sides of the lining shrinks inward, leading to a decrease in stress at the measuring points on both sides. During the third stage, after the primary lining concrete is fully solidified, no further deformation occurs. The surrounding rock stress around the excavation section gradually stabilizes, and meanwhile, the lining deformation reaches its maximum value.

Based on the above analysis, it can be seen that after the excavation of the connecting tunnel, the initial support is mainly subjected to downward soil stress from the top center. The larger the excavation step, the more obvious the stress concentration phenomenon at the central point of the top of the connecting tunnel's excavation working face; at the same

time, the greater the deformation of the initial support of the connecting tunnel, the greater the confining pressure borne at the arch crown of the initial support, with the increase in excavation distance. When the excavation step is controlled within the range of 2–3 m, the confining pressure distribution on the excavation working face of the connecting tunnel is relatively uniform, with no obvious stress concentration phenomenon, and the deformation of the initial support is small. Therefore, it is preliminarily concluded that for Class III surrounding rock excavated using the full-face method, the excavation step should be optimally controlled within the range of 2–3 m.

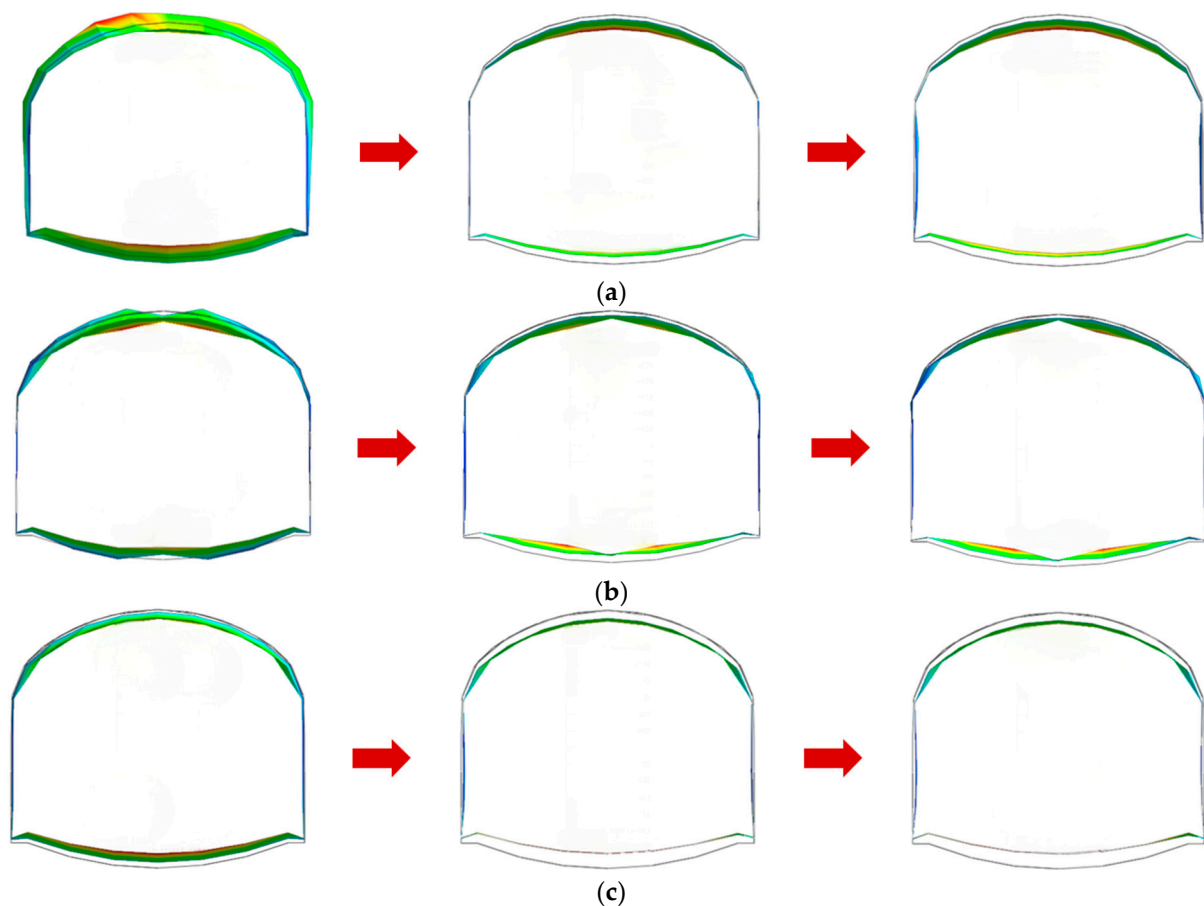


Figure 10. Deformation development process of the primary lining concrete under different excavation step distances: (a) 2 m; (b) 3 m; (c) 4 m. (The colors in the figure represent the deformation of the primary lining concrete of the connecting tunnel.)

4.2. Optimization of Blasting Excavation Construction Parameters

Taking the section at mileage post PYK0+060 as an example, the blasting design for this section is shown in Figure 11, and the relevant blasting parameters are listed in Table 2. Blasting dynamic loads are applied through numerical simulation software on this section to simulate the blasting effect, with three different working conditions of blasting dynamic loads set up. To ensure construction safety, the particle vibration velocity of the secondary lining of the main tunnel caused by blasting construction in the connecting tunnel should be controlled within 5 cm/s. The calculation formula for the safe distance of blasting vibration is as follows:

$$R = (K/V)^{\frac{1}{\alpha}} \cdot Q^{\frac{1}{3}} \quad (1)$$

where R is the permissible safety distance for blasting vibration; Q is the charge amount: for simultaneous blasting, it refers to the total charge; for delayed blasting, it refers to the maximum charge amount per segment; V is permissible particle vibration velocity at the

geological point of the protected object; K and α are coefficients and attenuation indices related to topographic and geological conditions between the blasting point and protected objects. K is mainly related to the topography and lithology between the blasting point and monitoring points: hard rocks usually take values of 50–150, while soft rocks can reach 250–350. α is affected by geological conditions: hard rocks generally take values of 1.3–1.5, and soft rocks generally take values of 1.8–2.0.

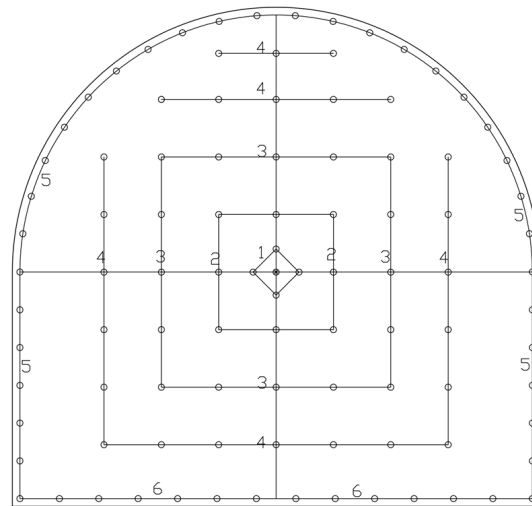


Figure 11. Borehole layout diagram for the right-line exhaust connecting tunnel. (The numbers in the figure correspond to the ring numbers in Table 2 respectively.)

Table 2. Blasting parameters for the single section of the right tunnel's exhaust air connecting tunnel.

Borehole Name	Ring Number	Borehole Depth (kN·m ³)	Number of Holes	Charge per Hole (kg)
Slot cutting hole	1	0.8	4	1.5
Slot expansion hole	2	1.6	8	1.2
Auxiliary hole	3	1.6	16	1.2
Auxiliary hole	4	1.6	25	1.2
Peripheral hole	5	1.6	32	0.4
Bottom hole	6	1.4	14	1.5

The section blasting method and maximum initiating charge of the explosive can be determined through the calculation formula for the safety distance of blasting vibration. According to the on-site rock layer sampling test, the rock firmness coefficient f is 8–10, belonging to hard rock. Therefore, when calculating the minimum allowable safety for blasting vibration, $K = 50$ and $\alpha = 1.5$. At this time, numerical simulation software is used to simulate the blasting dynamic loads with different maximum charge amounts, analyzing the effects of blasting dynamic loads on the main tunnel under different working conditions. The blasting parameters for different working conditions are shown in Table 3.

Table 3. Blasting parameters under different working conditions.

Working Conditions	Blasting Velocity (m/s)	Explosive Density (g/cm ³)	Charge Diameter (mm)	Borehole Diameter (mm)	Maximum Charge Amount (kg)
1	1	0.8	4	1.5	1.5
2	2	1.6	8	1.2	3
3	3	1.6	16	1.2	4.5

Figure 12 shows the numerical simulation results of particle vibration velocity for the secondary lining of the main tunnel under different working conditions. The maximum particle vibration velocities of the main tunnel's secondary lining in working condition 1 and working condition 2 are 2.9 cm/s and 4.3 cm/s, respectively, both complying with relevant code requirements. However, the maximum particle vibration velocity in working condition 3 exceeds the limit specified in the code. Moreover, the influence range of blasting dynamic loads in working condition 2 is significantly smaller than that in working condition 1. Therefore, the influence range of connecting tunnel blasting excavation on the main tunnel is related not only to the maximum charge amount but also to the charge diameter and borehole diameter.

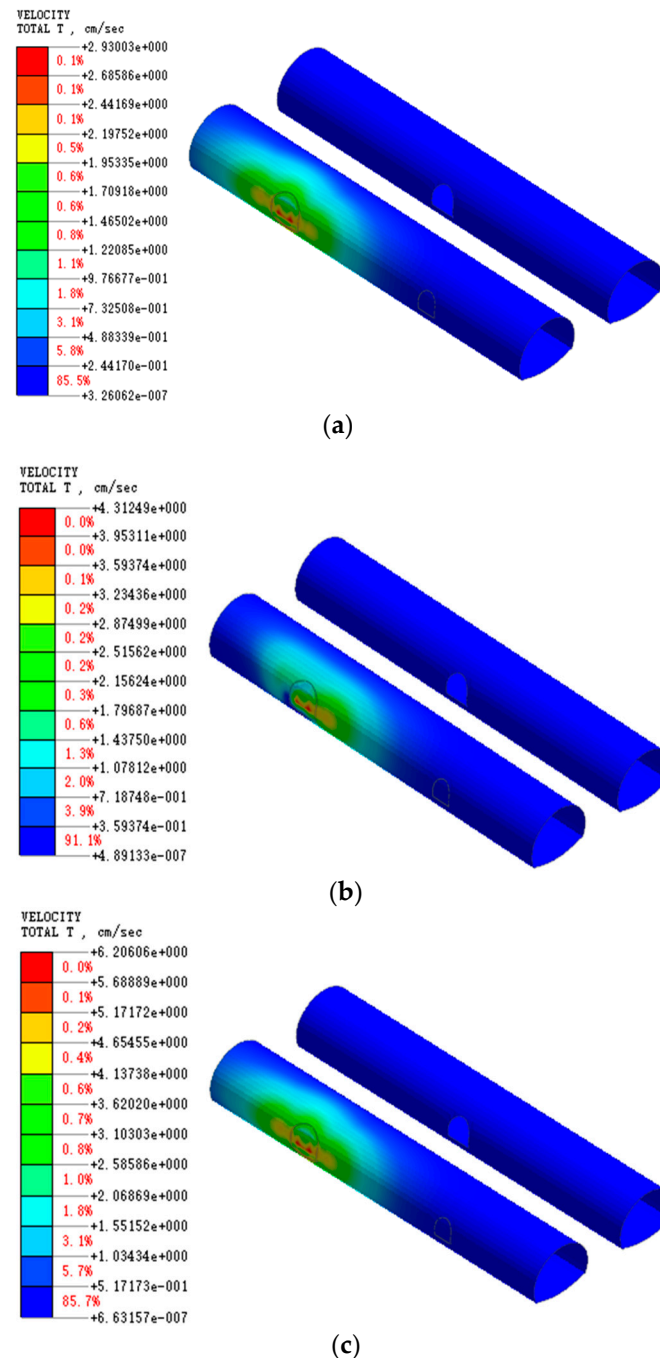


Figure 12. Particle vibration velocity of the secondary lining in the main tunnel under different working conditions: (a) working condition 1; (b) working condition 2; (c) working condition 3.

When the explosive in the borehole adopts decoupled charging, the ratio of borehole diameter to charge diameter can be increased, using the air gap to buffer the energy generated by the explosion. Combined with the optimization analysis results of the section excavation step distance in the previous text, the blasting construction scheme for the section of the right line exhaust connecting tunnel is now optimized. The optimized blasting parameters are shown in Table 4. The optimized blasting scheme adopts the single-hole single-segment initiation mode, with the maximum charge amount per hole increased to 3 kg and the excavation step distance increased to 3.0 m. Numerical simulation verifies that the optimized blasting parameters have increased the blast hole depth and charge per hole while appropriately reducing the number of blast holes in a single section. Compared with the pre-optimization blasting scheme, the new blasting scheme features a larger initiating charge per hole, a longer excavation step, and thus higher construction efficiency.

Table 4. Optimized blasting parameters for a single section.

Borehole Name	Ring Number	Borehole Depth (kN·m ³)	Number of Holes	Charge per Hole (kg)
Slot cutting hole	1	3.8	2	3.0
Slot expansion hole	2	3.6	6	2.6
Auxiliary hole	3	3.6	12	2.6
Auxiliary hole	4	3.6	20	2.6
Peripheral hole	5	3.6	28	0.8
Bottom hole	6	3.8	8	3.0

4.3. Micro-Disturbance Construction Control for Crossover Section

During the excavation of the left-line exhaust connecting passage, it needs to overpass the right tunnel of the main tunnel before connecting with the left tunnel. The minimum clear distance between the overpass section of the connecting passage and the top of the secondary lining of the right tunnel is only 5 m, and the total length of the construction section covering the overpass part of the connecting passage and the adjacent main tunnel is approximately 30 m. The on-site construction scheme is designed to adopt the full-face blasting method for excavation, with an advance of 1 m per cycle.

Figure 13a shows the numerical simulation of the secondary lining deformation of the main tunnel after the full-face construction of the overpass section of the connecting passage. At this point, the maximum deformation of the secondary lining at the position (on the top of the right tunnel) with the minimum clear distance from the connecting passage is approximately 0.9 mm, but the maximum cumulative deformation of the main tunnel's secondary lining reaches about 15 mm. Therefore, the adoption of the full-face method for the overpass section of the connecting passage causes significant disturbance to the main tunnel.

To reduce the impact on the main tunnel during the construction of the connecting passage, the overpass section of the connecting passage is planned to adopt upper and lower bench methods for excavation, with an advance of 0.6 m per cycle. Meanwhile, to ensure that construction schedule requirements are met, the left-line exhaust connecting passage will be constructed simultaneously from Junction 2 and the corresponding position of the left tunnel. Taking the section of the left-line exhaust connecting passage with a slope of -87% as an example, when using upper and lower bench methods for excavation, the upper bench height is 3.45 m, and the lower bench height is 3.05 m, with a single advance of 0.6 m and three consecutive advance cycles (as shown in Figure 14).

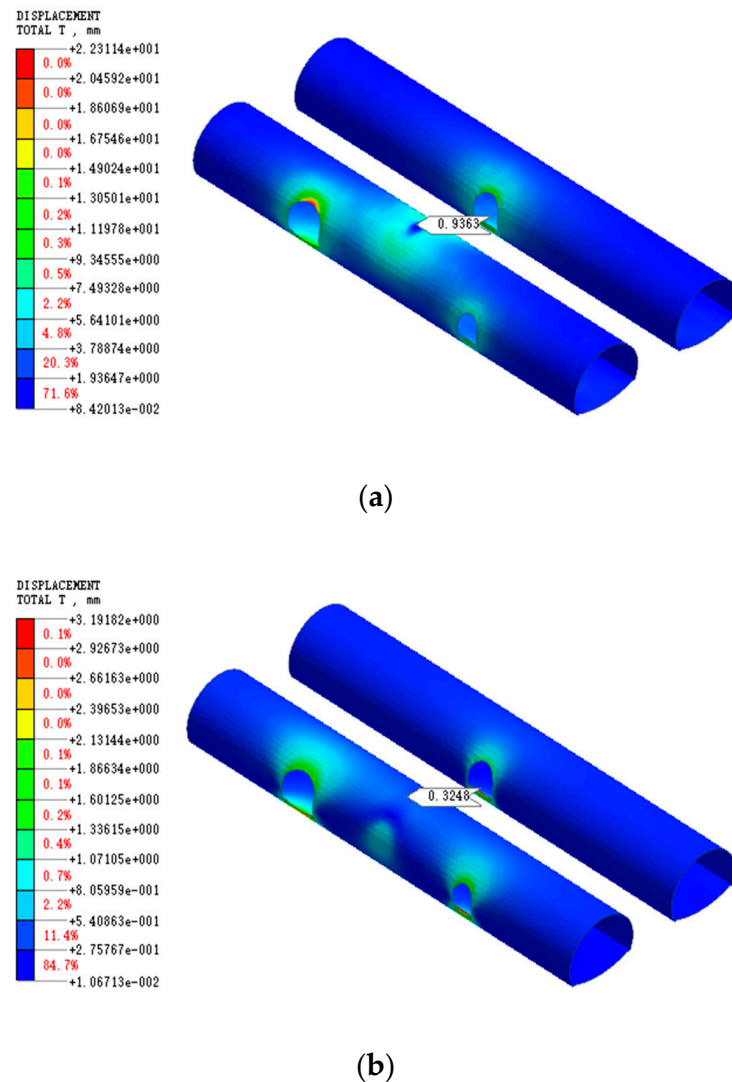


Figure 13. Deformation of the main tunnel's secondary lining under different construction methods: (a) full-face method excavation; (b) bench method excavation.

After analyzing the deformation of the secondary lining of the main tunnel during the construction of the left-line exhaust connecting passage using numerical simulation software, it is concluded that the maximum deformation of the secondary lining at the position (on the top of the right tunnel) with the minimum clear distance from the connecting passage is only approximately 0.3 mm (as shown in Figure 13b), which meets the requirements of relevant construction specifications. Therefore, compared with full-face excavation, the method of simultaneous excavation from both ends of the connecting passage using upper and lower bench methods not only reduces the impact on the secondary lining of the existing main tunnel during the construction of the overpass section of the connecting passage but also effectively improves construction efficiency.

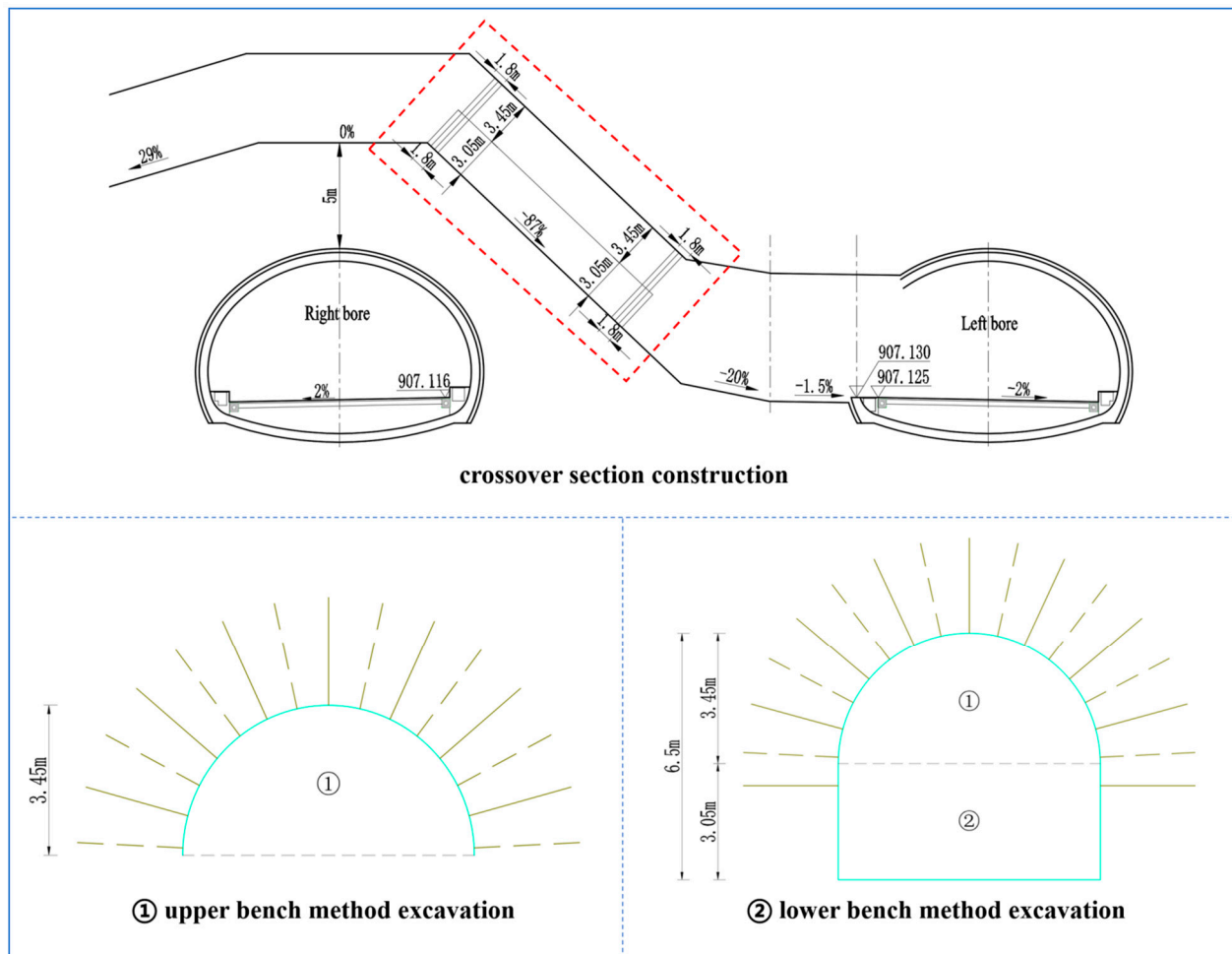


Figure 14. Micro-disturbance construction control for the left line exhaust connecting tunnel crossing over the right tunnel.

5. Conclusions

Based on the Xiaolongmen Tunnel Project in China, this study systematically investigated the impact of various construction parameters on the construction process of complex connecting tunnels in extra-long highways using methods such as on-site monitoring and numerical simulation. It analyzed the influence of excavation steps in connecting tunnel construction on surrounding rock and deformation of initial support, filling the gap in research on optimization of construction parameters for connecting tunnels in extra-long highways, and providing a theoretical basis for the construction of actual projects. Moreover, by combining on-site blasting construction schemes, numerical simulation was used to optimize blasting construction parameters and schemes, and the feasibility of the new scheme was verified through on-site construction, ensuring its practicality. The main conclusions of the study are as follows:

- (1) After the excavation of the connecting tunnel, the surrounding rock stress at the top of the tunnel face decreases rapidly, with the extreme value point of stress reduction located at the center of the section top. However, the surrounding rock stress on both sides increases rapidly, showing uneven distribution. The greater the excavation step distance, the greater the extreme value of surrounding rock stress variation at the center of the section top.
- (2) For Class III surrounding rock excavated using the full-face blasting method, the maximum influence range of working face excavation on the stratum along the

tunneling direction is approximately 4D (where D represents the excavation step). Meanwhile, the larger the excavation step, the more obvious the stress concentration phenomenon at the central point of the working face arch crown, and the excavation step should be optimally controlled within the range of 2–3 m.

- (3) The deformation of primary lining concrete after connecting tunnel excavation can be roughly divided into three stages: In the first stage, when the primary lining concrete has not yet solidified, the concrete deformation is large, and the surrounding rock stress changes significantly. In the second stage, the primary lining concrete begins to consolidate and shrink: the top of the lining undergoes settlement deformation under compression, while both sides deform with inward contraction. In the third stage, the state of the lining and surrounding rock basically tends to stabilize.
- (4) The influence range of blasting dynamic loads is affected by multiple factors. When the explosive in the borehole is charged in a decoupled manner, the ratio of borehole diameter to charge diameter can be increased, using the air gap to buffer the energy generated by the explosion. By adjusting the decoupling coefficient of borehole diameter to charge diameter and appropriately increasing the maximum initiation charge amount, the blasting scheme can be optimized to improve construction efficiency.

Author Contributions: Conceptualization: J.L. and S.H., software, D.H. and J.L.; validation, D.H., J.M., and J.L.; investigation, D.H., J.M., and S.H.; writing—original draft, S.H. and J.L.; writing—review and editing, D.H., J.M., and S.H. All authors have read and agreed to the published version of the manuscript.

Funding: This study was supported by the Jiangxi Provincial Leading Talent Program for Academic and Technological Leaders in Major Disciplines 20232BCJ22009 (the funder: Dawei, Huang), the Jiangxi Provincial Natural Science Foundation 20242BAB26077 (the funder: Dawei, Huang), and the Fundamental Research Funds for the Central Universities 2024YJS043 (the funder: Jianfei, Ma).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: Data is contained within the article.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Hong, K.; Feng, H. Development trends and views of highway tunnels in china over the past decade. *Chian J. Highw. Transp.* **2020**, *33*, 62–76.
2. Qian, Q. Challenges faced by underground projects construction safety and countermeasures. *Chin. J. Rock Mech. Eng.* **2012**, *31*, 1945–1956.
3. Editorial Department of China Journal Highway and Transport. Review on china’s traffic tunnel engineering research: 2022. *Chian J. Highw. Transp.* **2022**, *35*, 1–40.
4. China Merchants Chongqing Communications Research and Design Institute Co., Ltd. *Guidelines for Design of Highway Tunnels: JTG/T D70/2-02-2014*; China Communications Press: Beijing, China, 2014.
5. Jin, Q.; Yue, Z.; Ren, M.; Xue, K.; Pan, S. Design and application of intelligent blasthole detection and path planning system. *Coal Sci. Technol.* **2024**, *52*, 186–196.
6. Liu, D.; Gao, W.; Sun, B.; Liu, D.H. Numerical simulation of blasting vibration on existing tunnel extension. *Rock Soil Mech.* **2016**, *37*, 3011–3016.
7. Shan, R.; Zhao, Y.; Wang, H.; Liu, Z.; Qin, H. Blasting vibration response and safety control of mountain tunnel. *Int. J. Civ. Eng.* **2023**, *82*, c. [\[CrossRef\]](#)
8. Zeng, X.; Zhang, X.; Zhou, X.; Duan, Y.; Chen, J. Prediction of tunnel blasting vibration velocity considering the influence of free surface. *Appl. Sci.* **2023**, *13*, 1373. [\[CrossRef\]](#)
9. Yang, B.; Wang, H.; Chu, F.; Zhou, Y.; Xiao, J.; Zhang, D. Optimization of blasting construction for connecting passage of shield tunnel. *Sci. Technol. Eng.* **2021**, *21*, 8663–8669.

10. Bai, S.; Zhou, Z.; Gao, C.; Meng, Q.Y.; Liu, J.X.; Gao, T.; Wang, X.; Ke, C.L. Research on comprehensive analysis and control method of surrounding rock stability in submarine tunnel crossing fault fracture zone by drilling and blasting method. *Chin. J. Rock Mech. Eng.* **2025**, *44*, 691–704. [[CrossRef](#)]
11. Xie, Q.; Jia, Y.; Ding, K.; Zhang, J.; Ling, Z.D.; Liu, D.S. Chao characteristics analysis of tunnel blasting vibration signals. *J. Vib. Shock.* **2022**, *41*, 238–244+306.
12. Zhou, M.; Li, Z.; Huang, H.; Tong, Y.; Zhang, W. Method and validation for tunnel face collapse risk analysis considering construction sequence. *J. Disaster Prev. Mitig. Eng.* **2024**, *44*, 579–595.
13. Zhou, H.; Gao, Q.; Wang, Y.; Lu, W. Comparative analysis of vibration characteristics induced by different kinds of boreholes and their blasting boundaries during full-face tunnel blasting excavation. *Chin. J. Rock Mech. Eng.* **2022**, *41*, 785–797.
14. Lunardi, P. The design and construction of tunnels using the approach based on the analysis of controlled deformation in rocks and soils. *Tunn. Tunneling Int. ADECO-RS Approach* **2000**, *5*, 3–30.
15. Lunardi, P. *Design and Construction of Tunnels*; Springer: Berlin/Heidelberg, Germany, 2008.
16. Wu, S.; Tang, H.; Luo, H.; Dai, Y.; Deng, Q. Progressive failure law and control criterion for safe construction of shallow buried highway tunnel with different grades of surrounding rock. *Chian J. Highw. Transp.* **2019**, *32*, 205–216.
17. Zhu, W.; Li, X.; Zhang, Q.; Zheng, W.H.; Xin, X.L.; Sun, A.H.; Li, S.C. A study on sidewall displacement prediction and stability evaluations for large underground power station caverns. *Int. J. Rock Mech. Min. Sci.* **2010**, *47*, 1055–1062. [[CrossRef](#)]
18. Abdollahipour, A.; Rahmanned, R. Sensitivity analysis of influencing parameters in cavern stability. *Int. J. Min. Sci. Technol.* **2012**, *22*, 707–710. [[CrossRef](#)]
19. Jing, H.; Li, Y.; Xu, G. Analysis of surrounding rock stability of deeply buried roadways and study on its control techniques. *Rock Soil Mech.* **2005**, *26*, 877–880.
20. Zhou, H.; Zhou, X.; Ge, L.; Zhang, F.; Feng, L.H.; Liu, Y. Optimization of mud ratio for freezing hole drilling in water-rich sand pebble formation. *Sci. Technol. Eng.* **2023**, *23*, 6207–6216.
21. He, C.; Li, X.; Jiang, Y.; Jiang, Y.C.; Fang, Y.; Tan, Z. Laboratory test on shield tunneling in loess strata. *Chin. J. Rock Mech. Eng.* **2013**, *32*, 1736–1743.
22. Jiang, Y.; He, C.; Hu, X.; Feng, Y. Laboratory test study of soil disturbance cause by shield tunnelling in sandy strata. *Chin. J. Rock Mech. Eng.* **2013**, *32*, 2550–2559.
23. Xie, X.; Yang, Y.; Ji, M. Analysis of ground surface settlement induced by the construction of a large-diameter shield-driven tunnel in Shanghai, China. *Tunn. Undergr. Space Technol.* **2016**, *51*, 120–132. [[CrossRef](#)]
24. Wei, X.; Zhou, Y.; Wei, G. Research of EPB shield tunneling parameter relations and their influence on stratum displacement. *Rock Soil Mech.* **2013**, *34*, 73–79.
25. Chen, R.; Zhang, P.; Wu, H.; Wang, Z.; Zhong, Z. Prediction of shield tunneling-induced ground settlement using machine learning techniques. *Front. Struct. Civ. Eng.* **2019**, *13*, 1363–1378. [[CrossRef](#)]
26. Ding, Z.; Wang, F.; Wei, X.; Zhuang, Z. Three-dimensional deformation and pore pressure caused by shield construction in saturated soil. *Chin. J. Rock Mech. Eng.* **2018**, *37*, 2189–2199.
27. Ding, Z.; Hong, Z.; Feng, C.; Wei, X.; Xu, T.; Wang, T. Soil deformation and excess pore water pressure caused by excavation of twin-bored tunnels in soft soil. *Chin. J. Geotech. Eng.* **2020**, *42*, 1465–1473.
28. China Communications Second Highway Survey and Design Institute Co, Ltd. *Guidelines for Design of Highway Tunnels: JTG/T D70-2010*; China Communications Press: Beijing, China, 2010.

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.