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Study on Construction Mechanical Behavior of Intersection of Tunnel Group in Metro Station

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Abstract: *In view of the many crossing areas of urban subway station tunnel groups, the construction is difficult, the form of force is complicated, and safety risks are prone to appear. Relying on Nanning Qingxiushan metro station, a three-dimensional numerical model was established to assist on-site monitoring data, and the deformation of surrounding rock, stress changes, and plastic zone distribution during the construction of the tunnel group were analyzed, and the dynamic mechanics of the cross passage of the left tunnel behavioral research. The tunnel is significantly affected by the upper biased foundation pit, the displacement and stress of the surrounding rock at the intersection of the tunnel on the biased side are significantly greater than that on the non-biased side; The excavation of the cross passage has a significant impact on the intersection and large plastic strains appear in the intersection area; After the left-line tunnel is broken, the lining vault of the opening will generate tensile stress, there will be obvious stress concentration in the arch shoulder and arch foot area. Selecting appropriate deformation control measures can ensure the safety of the structure.*

Keywords: *Subway Station; Cross Tunnel Group; Numerical Simulation; Construction Mechanics*

1. Introduction

Due to limitations in underground space, construction scenarios involving intersecting tunnels are increasingly common in certain areas, constrained by existing structures, geological conditions, and the need for comprehensive development and utilization of underground space [1]. The spatial structure of tunnel intersection areas experiences complex stress conditions, with stress concentration at the intersections, making them weak points in the tunnel structure and critical focuses during construction [2-3].

With the rapid development of metro tunnels, numerous scholars domestically and internationally have conducted extensive research on the construction of intersecting tunnels. Model tests, based on similarity theory, are a crucial method for studying the mechanical behavior of tunnel construction [4]. Dong Jie et al. [5] used similar model experiments and numerical simulations to explore the stress variation laws of surrounding rock in intersecting tunnels and its influencing factors. Timely monitoring and analysis of field data can accurately and effectively reflect the deformation and stress changes in the surrounding rock during tunnel construction, guiding on-site construction [6]. Liang Qingguo et al. [7] statistically analyzed the surrounding rock pressure of different tunnels, studying the overall distribution characteristics of tunnel surrounding rock pressure and its relationship with factors such as tunnel lithology, construction method, tunnel depth, and span, and discussed the stress patterns of tunnel support structures and the spatiotemporal distribution characteristics of surrounding rock pressure. Numerical simulation offers advantages such as fast computation speed, intuitive results, and cost savings, playing a very important role in tunnel design and construction [8]. Wang Mengshu et al. [9], using the Tianhengshan Tunnel as an engineering background, investigated the influence of cross-passage construction stages on the lining structure of the main tunnel.

2. Engineering Background

Qingxiushan Station is the 17th station from north to south on Nanning Rail Transit Line 3. It is located west of the intersection of Qingshan Road and Fengling South Road, arranged across Fengling South Road. The station starts at chainage YCK20+044.918 and ends at YCK20+229.618, with a total design length of 184.7m. The open-cut station hall structure has a depth of 24.5m, a length of 82.4m, and a width of 41.8m. The platform tunnel and cross-passage are buried at approximately 47m depth, classifying them as deep-buried tunnels. The station comprises a station hall foundation pit, platform

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tunnels, cross-passages, and an inclined escalator tunnel.

The excavation height of the main station tunnels is 12.3 meters, with a single tunnel excavation span of 11.3 meters and a spacing between the twin tunnels of 20.5 meters. The cross-passage excavation height is 11.0m with an excavation span of 9.3m. The inclined escalator tunnel has an excavation height of 6.2m and an excavation span of 7.9m. The open-cut station hall connects to the platform level cross-passage via an escalator incline tunnel sloping downward from the base slab. The escalator incline tunnel consists of a 2.9m long straight section and a 39.8m long sloped section, sandwiched between the parallel platform tunnels, with minimum distances to the left and right main tunnels of 6.14m and 4.16m, respectively.

Geotechnical investigation data indicate that the main area of the Qingxiushan Station tunnel is located in Quaternary-Paleogene semi-diagenetic rock strata. For simplicity in analysis, the soil/rock mass is simplified into four layers: the first layer is 4m thick plain fill(①$_2$), the second layer is 18m thick mudstone(⑦$_{1-3}$), the third layer is 51m thick siltstone(⑦$_{2-3}$), and the fourth layer is 27m thick silty (fine) sandstone(⑦$_{3-3}$). The initial support consists of double-layer steel mesh, lattice girders, and C25 concrete, with a thickness of 0.35m. The secondary lining comprises a waterproof layer and cast-in-place 800mm thick C40, P12 concrete.

3. Calculation Model and Parameters

3.1 Analysis Model

The analysis model selects the section Z(Y)DK20+125.870 to Z(Y)DK20+189.968 of the station, simulating the excavation of the station platform tunnels, cross-passage tunnel, and inclined escalator tunnel. The model dimensions are 120m in the X-direction, 160m in the Y-direction, and 100m in the Z-direction. The top surface (ground surface) is free, while the other surfaces are constrained by normal displacements. The model uses a mixed mesh, containing 125,791 elements and 75,452 nodes in total, as shown in Figure 1. The tunnel excavation adopts the three-bench method, with the upper, middle, and lower bench faces spaced 4 meters apart, and each excavation cycle advances 1 meter. The positional relationship of the tunnels in the model is shown in Figure 2.

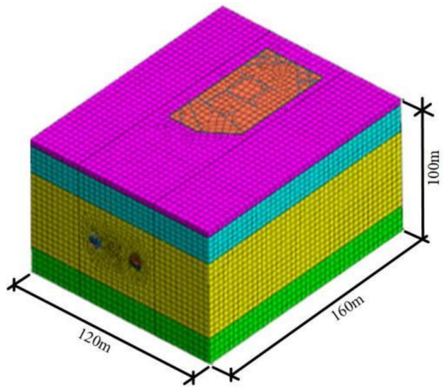

Figure 1 Finite element calculation model.

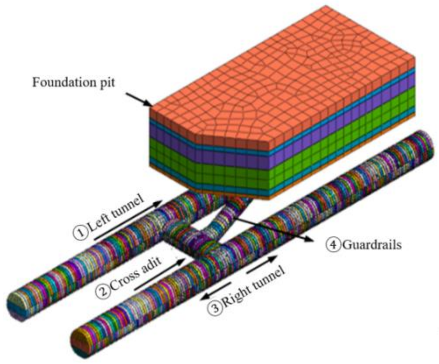

Figure 2 Tunnel location relationship diagram.

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3.2 Calculation Assumptions

The stratum where the underground excavated metro tunnel group is located is assumed to be an isotropic, homogeneous continuous medium. The Mohr-Coulomb constitutive model is adopted for the stratum, considering only the initial stress field caused by self-weight. The soil and station hall structure are simulated using solid elements. The tunnel support structures and rock bolts are simulated using shell elements and embedded truss elements, respectively. Before the tunnel group construction, the groundwater level is assumed to have been lowered below the tunnel invert elevation; thus, the influence of groundwater is not considered in the numerical analysis. In the finite element model, the parameters for the soil/rock mass and surrounding rock are obtained from geotechnical investigation reports and relevant experience. Specific material physical parameters are listed in Table 1.

Table 1 Physical and mechanical parameters of materials.

Name	Weight /kN·m^{-3}	Elastic Modulus /MPa	Poisson's Ratio	Cohesion *c* /kPa	Friction Angle *φ* / °
①$_2$	19.7	28.25	0.40	11	8
⑦$_{1-3}$	20.7	42.30	0.24	50	20
⑦$_{2-3}$	21.6	42.30	0.24	60	30
⑦$_{3-3}$	22.9	160.00	0.23	75	36
Initial Support	24.0	31500	0.20	—	—
Secondary Lining	25.0	33500	0.20	—	—

3.3 Tunnel Excavation and Support Sequence

The three-bench method is used for tunnel cross-section excavation in the model, with a cycle footage of 1 meter. The lagging distance between the upper, middle, and lower benches is 4m. Initial support and secondary lining are applied immediately after excavation. The station tunnel excavation sequence is as follows:

① Commence excavation of the left platform tunnel first.

② Start building the cross-passage when the tunnel face is twice the tunnel diameter past the cross-passage location.

③ After finishing the cross-passage, excavate the right tunnel in both directions.

④ Once both platform tunnels are complete, dig the inclined escalator tunnel downward from the base slab.

4. Analysis of Simulation Results

Three monitoring sections, located at the centerline of the cross-passage and 10 meters ahead and behind it, are selected for analyzing the station tunnels.

4.1 Surrounding Rock Displacement Analysis

Figure 3 shows the vertical displacement nephogram of Section B after tunnel excavation. Table 2 lists the crown settlement values of the main tunnels in Sections A, B, and C. Table 3 shows the maximum principal stress at different locations of the main tunnel in Section A. From Figure 6, Table 2, and Table 3, it can be observed that: (1) After the excavation stabilizes, the vertical displacement and maximum principal stress of the surrounding rock support structure of the main tunnel in Section A, and the vertical displacement of the support structure in Section B, are all greater for the right tunnel than for the left tunnel. This is analyzed to be due to the influence of the eccentric loading from the foundation pit above, resulting in a greater burial depth at the intersection of the right tunnel and larger stress release. When the eccentric load reaches a certain critical value, the entire support structure might "drift" towards the non-eccentrically loaded side. Simultaneously, according to Table 3 data, the maximum principal stress values at the crown of the main tunnel are relatively small, indicating lower pressure on the surrounding rock at the crown. Measures such as advanced small pipes and pipe roof reinforcement can be applied to the crown area to ensure tunnel safety. (2) The crown settlement and invert heave of the platform tunnel are not symmetric about the tunnel centerline. The reason is that the

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platform tunnel is located below an eccentrically loaded foundation pit; the strength of the base slab and retaining piles of the upper foundation pit is greater than that of the surrounding rock, causing the deformation trend of the surrounding rock to incline towards the inside of the tunnel. (3) The vertical displacement at the crown in Section A is greater than in Sections B and C. This indicates that the excavation of the cross-passage has a greater impact on the tunnel intersection area. Construction should closely monitor the deformation of the surrounding rock in this area. If necessary, the strength of shotcrete should be increased, and the lining thickness and corresponding reinforcement should be appropriately enhanced.

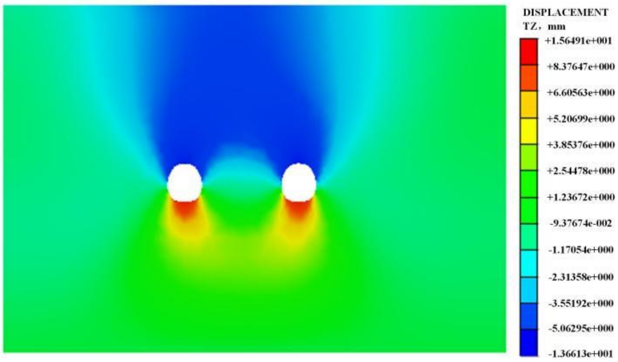

Figure 3 Nephogram of vertical displacement at Section B after tunnel excavation.

Table 2 Maximum deformation of surrounding rock (mm).

Section	Crown Settlement	
	Left Tunnel	Right Tunnel
A	-9.63	-11.55
B	-9.44	-10.04
C	-6.93	-6.16

Table 3 Maximum principal stress in surrounding rock at Section A (MPa).

Location	Max. Principal Stress	
	Left Tunnel	Right Tunnel
A2, A6	-0.087	-0.048
A1, A7	-0.23	-0.15
A15, A8	-0.42	-0.22
A14, A9	-0.62	-0.41
A13, A10	-0.21	-0.11

To further study the influence of construction steps on the displacement of surrounding rock at different locations in the tunnel intersection area, typical measuring points from three monitoring sections were selected for analysis: (1) The variation trends of crown settlement for each tunnel are similar: a stage of small settlement → a stage of rapid increase → a stage of slow increase → a stage of stable convergence. (2) The crown settlement curve of the left tunnel shows two jump increases. The first jump occurs when the excavation face reaches the monitoring section, caused by frequent disturbance and stress redistribution of the surrounding rock. The second jump occurs at step 59, which involves breaking the initial lining at the reserved cross-passage opening in the upper bench of the left tunnel, causing a stress system transformation in the support system. As the face moves away from the monitoring section, the settlement value gradually stabilizes near a certain value without significant further change. (3) After the completion of cross-passage excavation, the vertical displacement at the crown of the main tunnel in Sections A, B, and C increases by varying degrees: 2.03mm (A2) and 10.18mm (A6), 1.32mm (B2) and 5.54mm (B7), 0.78mm (C2) and 3.12mm (C5), respectively. During actual construction, smaller excavation advance steps should be used, structural monitoring should be strengthened, and pre-support measures such as large pipe roofs and small pipe grouting should be adopted to ensure the safety of the overall structure. (4) After the completion of the inclined escalator tunnel construction, the crown of the platform tunnel shows almost no change. This is analyzed to be because the secondary lining constructed earlier bears most of the force. The crown settlement of the cross-passage increased by 0.79mm (A4). Dynamic monitoring of surrounding rock data in this intersection area should be strengthened during construction, and corresponding reinforcement measures should be taken if necessary.

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4.2 Surrounding Rock Stress Analysis

The distribution of surrounding rock stress after tunnel excavation significantly affects tunnel stability. Limited by space, only the maximum principal stress nephogram for Section B is provided here. (see Figure 4).

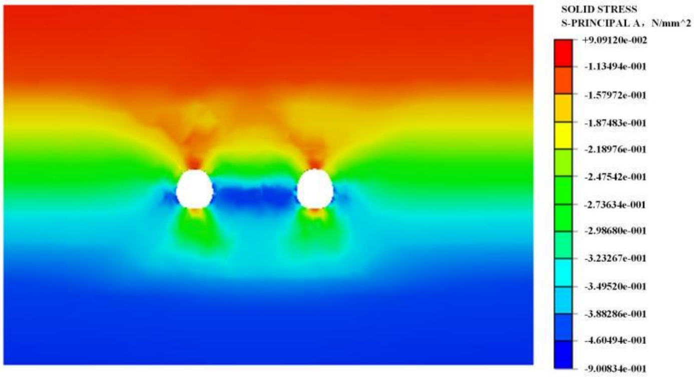

Figure 4 Nephogram of maximum principal stress at Section B.

From the figure, it can be observed that: (1) After tunnel excavation, the maximum value of the maximum principal stress in the surrounding rock around the tunnel occurs at the tunnel invert. (2) Tensile stress, which is relatively concentrated, appears at the tunnel crown and invert. Compressive stress appears at the tunnel haunch and springline, and the squeezing of the middle rock pillar is obvious. Measures should be taken promptly during construction for these areas to prevent surrounding rock instability.

Figures 5 and 6 show the variation curves of the maximum principal stress at crown monitoring points in different sections of the main tunnel with construction steps. It can be seen that: (1) The maximum principal stress at the crown monitoring points in different sections of the main tunnel changes abruptly when the excavation face passes the monitoring section. When the face moves away from the monitoring section, the maximum principal stress tends to stabilize. (2) The maximum principal stress at measuring point A2 shows two jump changes. The first jump occurs when construction reaches monitoring section A. The second jump occurs at step 59, which involves breaking the lining at the reserved cross-passage opening in the upper bench of the left tunnel, causing a stress system transformation in the support system. During construction, the lining thickness of the cross-passage and corresponding reinforcement should be appropriately increased. (3) After the completion of all tunnel excavations, the maximum principal stress at measuring point A2 is greater than at point C2. This is analyzed to be due to the influence of the upper eccentrically loaded foundation pit, resulting in a greater burial depth for point A2 compared to C2. Deeper burial depth leads to greater stress release in the surrounding rock. Construction should adopt methods emphasizing rapid excavation, rapid support, and rapid closure.

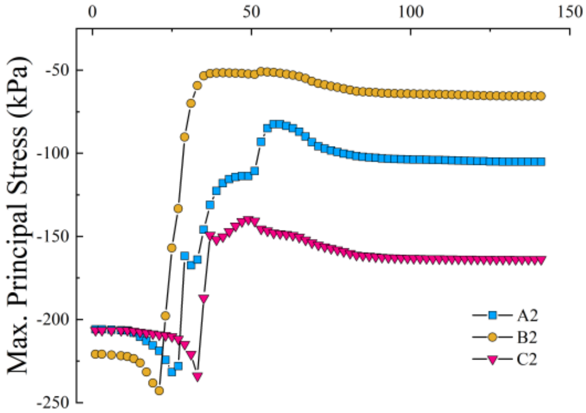

Figure 5 Variation curve of maximum principal stress at the crown monitoring point of the left tunnel with construction steps.

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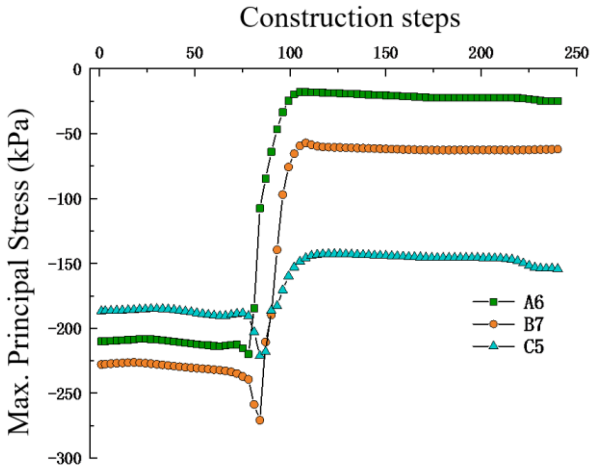

Figure 6 Variation curve of maximum principal stress at the crown observation point of the right tunnel with construction steps.

4.3 Plastic Deformation Analysis

After the complete excavation of the station tunnels, the equivalent plastic strain of the surrounding rock is shown in Figure 7. It can be seen that stress redistribution after excavation causes shear failure in the surrounding rock, leading to the development of plastic zones deep into the rock mass. Plastic zones with a height of about 9.5m and depth of about 1.2m~2.1m, extending throughout the station tunnels, appear on the left and right sidewalls of the platform tunnels. Through-going plastic zones appear on both sidewalls of the cross-passage. Larger plastic strains exist in the intersection area of the platform tunnels and the cross-passage. During the design phase, it is recommended to adopt measures such as advanced small pipes and pipe roof reinforcement in the crown area to reduce stress release and vertical settlement. The internal bending moments in the initial support at the intersection area increase; it is suggested to appropriately increase the main reinforcement of the lattice girders in the main tunnels and cross-passage. Additionally, advanced spiling (lock-foot) anchor pipes should be added at the connections of the initial support arches for reinforcement, ensuring firm connection with the tunnel arches. Longitudinal connecting bars of the steel frames should be strengthened. When breaking through the tunnel, similar lock-foot anchor pipes should be promptly added at the invert, and grouting consolidation should be enhanced to pre-consolidate the arch frame for the subsequent cross-passage excavation. For excavation in the surrounding rock of the tunnel intersection section, non-blasting methods, using machinery combined with manual trimming, are recommended to ensure the integrity of the surrounding rock and improve its own bearing capacity and stability. After excavation, protruding unstable rocks should be promptly removed to create a smooth face and excavation profile, avoiding stress concentration leading to local instability. The invert at the intersection of the main tunnels and the cross-passage should be closed into a ring as early as possible to prevent excessive plastic deformation and failure. Simultaneously, dynamic monitoring of areas with concentrated plastic zones, such as the sidewalls of the platform tunnels and cross-passage, and the tunnel intersection locations, should be strengthened, with corresponding construction measures taken if necessary.

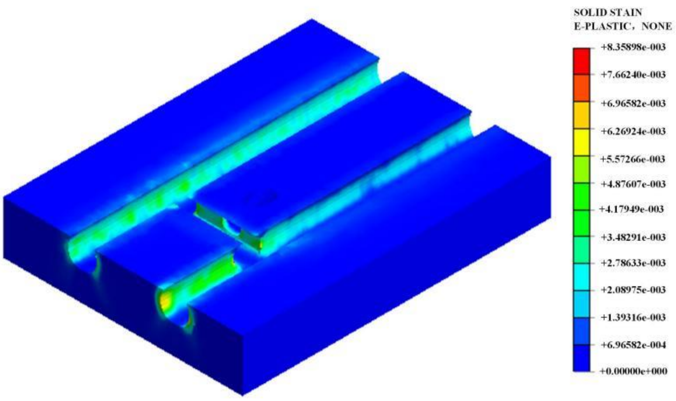

Figure 7 Nephogram of equivalent plastic strain.

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5. Validation of Effectiveness

To verify the reliability of the simulation results, a monitoring section was installed at location KGD93-2 during tunnel construction, using a total station to monitor tunnel crown settlement.The trend of the monitored crown settlement curve is generally consistent with the simulated curve. The final monitored crown settlement value was -12.5mm, slightly larger than the numerical simulation result of -11.3mm. This discrepancy is analyzed to be because, in actual excavation, the installation of monitoring points lags behind the tunnel excavation and support process, and the influence of groundwater was not considered. Overall, the difference between the monitored and simulated crown settlement values at section KDG93-2 is 1.2mm, which is small, and the trends are similar. Therefore, the numerical simulation results can be considered reliable.

6. Conclusions

(1) Influenced by the upper eccentrically loaded foundation pit, after tunnel excavation stabilizes, the displacement and stress of the surrounding rock in the intersection area of the tunnel on the eccentrically loaded side (right tunnel) are significantly greater than those on the non-eccentrically loaded side (left tunnel). The support structure may tend to deflect towards the non-eccentrically loaded side, potentially affecting the normal performance of the support system.

(2) The excavation of the cross-passage significantly affects the intersection area of the platform tunnels. It is recommended to adopt pre-support measures such as large pipe roofs and small pipe grouting, add advanced spiling (lock-foot) anchor pipes, and appropriately increase the main reinforcement of the lattice girders at this location to reinforce the area and ensure the safety of the overall structure.

(3) Through-going plastic zones appear on the left and right sidewalls of the platform tunnels and cross-passage throughout the station, with larger plastic strains occurring in the tunnel intersection area. It is recommended to use non-blasting methods, combining machinery with manual trimming, to achieve a smooth tunnel profile as much as possible, reducing overbreak/underbreak and stress concentration. The invert at the tunnel intersection should be closed into a ring as early as possible. Monitoring of areas with concentrated plastic zones, such as the intersection of the main tunnels and the cross-passage, should be strengthened, allowing for dynamic construction based on field monitoring data.

Author Biography

Li Zhi-Yong works at Jing Jin Ji Intercity Rapid Railway Investment Co., Ltd., Xiong'an 070001, China. His research mainly involves shield tunnel construction technology and the mechanical behavior of tunnel in metro station.

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