Mechanical Characteristics of Structures and Ground Deformation Caused by Shield Tunneling Under-Passing Highways in Complex Geological Conditions Based on the MJS Method

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Abstract: This study defined the height ratio of soft-rock strata and established a numerical model for analyzing shield construction in upper-soft, lower-hard composite strata together with field monitoring data. In this way, the influence of shield tunneling while passing under the pile foundation of the culvert at a short distance (the shortest distance is 1.4 m) in the typical upper-soft, lower-hard composite strata in Guangzhou can be examined. Moreover, the reinforcement effects of the ground, the bridges, and the culverts, using the strata-reinforcing plan dominated by the metro jet system (MJS) in a narrow space, are evaluated. Based on the results, (i) the maximum ground subsidence is found at the position in which the height ratio of the soft rock is 1.0. (ii) However, differential subsidence might be found in the subsequent shield construction when the soft-rock height ratio of the adjacent excavated surface ranges from 0 to 0.2 and from 0.5 to 1. (iii) The concentrated release of stress has a greater impact on the structure than the geological conditions of the shield tunneling face. (iv) Reinforcing with the MJS method contains the concentrated release of stress. This study can provide a reference for controlling the deformation of the under-crossing structure in the shield construction of the upper-soft, lower-hard composite strata.

Keywords: tunnel engineering; deformation control; MJS method; shield tunnel; bridges and culverts

1. Introduction

With the increase in the scale development of underground transportation in recent years [1], the development of underground space has been subjected to significant challenges owing to geological environmental factors. Jae, K.L., Ziyong, H., Shaohua, L. and others [2–5] encountered complex geological conditions in practical projects. The shield method is the most frequently used construction method for subway tunneling. In most cases, the subway shield tunnel must be constructed in complicated environments and pass underneath various buildings, highways, and bridges. In general, measures to reinforce or isolate the soil mass around the existing structures [6–8] are taken to lower the influence of close-range shield tunneling on the existing structures. High-pressure jet grouting is a common reinforcement method [9–12]. However, foundation reinforcement itself is also a disturbance to the surrounding soil mass. Hence, it is of great importance to adopt an effective foundation-treatment method with less disturbance to the surrounding environment. The horizontal jet-grouting pile method, which has been adopted by most similar projects [13,14] for years, forms a reinforced arch shed around the excavation surface of the tunnel, safeguarding the stability of the excavation face and the strata [15]. However, the traditional jet-grouting pile method has problems such as the stagnation of the sludge discharge in the jet-grouting process [16]. Not until the extensive application of
the Metro Jet System (MJS) method did we settle these problems [17]. Applying the unique porous-pipe technology to the jet pipe of the MJS method is conducive to realizing the real-time monitoring of the sludge discharge and the ground pressure in the hole. In this way, the ground pressure can be controlled by adjusting the discharge volume, thereby effectively reducing the environmental impact. Moreover, the expanded diameter can be guaranteed by the decline in the ground pressure and the sludge discharge. Consequently, its pile-formation effect is significantly superior to that of the traditional high-pressure jet-grouting process [18,19].

Many studies on high-pressure jet-grouting piles have been conducted by scholars at home and abroad. Lignol [20] conducted a design study on the parameters of the horizontal jet-grouting pre-reinforcement with consideration of the possible defects of the jet-grouting pile and acquired the optimal shape and minimum thickness for reinforcing the horizontal jet-grouting pile, proving the stability of the reinforced structure. Tonon [21] effectively controlled ground-heave and subsidence in the jet-grouting pile construction by adopting the injection technology and referring to the case of the jet-grouting pile reinforcement in highway tunnel construction. Pichler [22] established a numerical model to study different jet-grouting pile layout plans using specific soil parameters and presented the best reinforcement plan for jet-grouting piles under the working conditions of upper-soft, lower-hard, upper-hard, and lower-soft, based on a comparative analysis. Moreover, there have been many proven pieces of research on stratum loss, providing a variety of calculation methods with respect to ground subsidence. Peck [23] first proposed the concept of stratum loss. Subsequently, Rowe [24–26] introduced the gap parameter “GAP” to theoretically analyze the stratum loss produced in the shield-tunneling process. In addition, Attewell [27] obtained the expression for the ground subsidence curve based on the error accumulation function. Bobet [28] provided analytical solutions for the surface deformation and the stress of the lining structure caused by shallow-tunnel excavation in the saturation stratum. Hisatake [29,30] proposed a new computation method for ground subsidence, taking the weight of the excavated soil as the key point based on engineering practices.

The MJS method has been extensively used as a pre-reinforcement plan in shield approach engineering. In addition, oblique jet-grouting piles at various angles can satisfy the reinforcement requirements of the increasingly complex subway lines and the surrounding environment. Furthermore, there are increasingly proven methods for predicting ground subsidence. With the complex and uncertain geological conditions in Guangzhou, homogeneous soil layers or strata cannot be found within the appropriate buried depth of the tunnel. As a result, shield tunnels need to be constructed in the upper-soft, lower-hard composite strata in most cases, leading to large disturbances to the ground and adjacent buildings [31–33]. In response to this, more evidence is required on the applicability of the MJS method in the Guangzhou upper-soft and lower-hard strata shield-tunneling project and the reinforcement mode of the MJS method. This study is aimed at investigating the reinforcement effect of the MJS-dominated reinforcement plan in the upper-soft, lower-hard composite strata with a focus on the construction of bridges and culverts at the upper-soft, lower-hard composite strata in the Tongdewei-Shangbu section of the northern extension of Guangzhou Metro Line 8.

2. Case Description

2.1. Overview

The Tongdewei and Shangbu stations in the northern extension of Guangzhou Metro Line 8 are located at ZDK20+896.550 and ZDK21+694.400, where “ZDK” refers to the distance between the milepost and the start of the tunnel on the left of Metro Line 8. The left and right lines of the section are 617.142 m and 616.385 m in length, respectively. A slurry shield was adopted for construction in this section. The cutter diameter of the shield tunneling machine was 6280 mm. The inner diameter of the tunnel was 5400 mm; the thickness of each circular segment was 300 mm; and the radial length was 1500 mm. Specifically, the tunnel passes under the Xicha pedestrian culvert of Guangzhou City Northern
Ring Road at the section between ZDK21 + 163.210 and ZDK21 + 198.830. The pedestrian culvert has hollow deck slab girders with a span of approximately 13 m. The tunnel section crosses at approximately 75° to the traffic direction of the ring road, as shown in Figure 1. The cross-sections of both the pedestrian culvert and the shield tunnel are shown in Figure 2. The left tunnel passes just 1.4 m below the tips of the pile foundations for the abutments of the pedestrian culvert.

2.2. Geological and Hydrologic Conditions of the Construction Site

According to the drilling data, the rock and soil layers below the vertical direction of the surface can be divided into four types: the artificial fill layer, the alluvial-river layer, the eluvial horizon, and the rock weathering zone. Owing to the large fluctuation of the limestone face in the area, a thick layer of sandy soil can be observed above the rock face. The main soil and rock layers at the site where the shield tunnel passes under the highway include <1> miscellaneous fill, <4-2> slit soil, <3-1> silty-fine sand, <3-3> gravelly sand, and <9C-2> slightly weathered limestone. Most of the tunnels are in the VI-grade surrounding rock, which is mostly surrounded by a sand layer. Parts of the tunnels are in the limestone with a high strength. The initial water level in the drill hole during investigation was between 3.10 and 4.70 m in burial depth, and the stabilized water...
level was between 3.33 and 4.80 m in burial depth. The geological profile of the left tunnel is shown in Figure 3. The geological condition of the right tunnel was similar to that of the left tunnel.

Figure 3. Geological profile of left tunnel.

2.3. Definition of Soft-Rock Height Ratio

The northern extension of Guangzhou Metro Line 8 is located in a typical upper-soft, lower-hard composite strata. Its overburden is roughly consistent with other shield tunnels constructed in the upper-soft, lower-hard composite strata in Guangzhou based on their properties. However, the distribution of soft rock in the tunnel excavation face changes with the direction of the tunnel axis, resulting in different proportions of soft rock in different excavation faces. To study the variation law of the structure and stratum deformation under different proportions of soft rock in the shield-excavation face, the soft-rock height ratio is proposed and defined.

$$\beta = \frac{h}{D}$$

where $\beta$ is the ratio of the soft-rock height to the total height of the shield-tunneling face, $h$ is the soft-rock height, and $D$ is the total height of the shield tunneling face, as shown in Figure 4.

Figure 4. Comprehensive stratigraphic diagram of Tongdewei-Shangbu shield tunnel.

2.4. Engineering Difficulties and Measure

(i) Complex stratum: The stratum where the shield tunnel is located is composed of soft rock (sand soil) and hard rock (slightly weathered limestone). Moreover, the combina-
tion ratios of hard-rock and soft-rock (sand soil) strata vary. Under a diversified stratum, it is difficult to control the tunneling parameters.

(ii) The minimum distance from the pile foundation of the culvert to the tunnel is 1.4 m. Therefore, a strong disturbance on the soil mass might lead to the deformation of the pile foundation of the culvert, threatening the stability of the present culvert.

(iii) The influence of the left-tunnel-excavation process on the right tunnel is a complex extrusion and unloading process. The extrusion results from the diffusion of the cutter pressure and the tail-grouting pressure in the stratum. Therefore, necessary reinforcement measures are expected to reduce the impact of the newly built tunnel on the preceding tunnel.

2.5. Reinforcement Plan for Shield Construction Under-Passing Bridges and Culverts

In this project, traditional means such as pile-foundation underpinning, cannot be adopted because the culvert is only one pathway across the busy highway for pedestrians within a 1 km radius, and the traffic volume is heavy on the highway. Long-term occupation of the highway and the closure of the culvert for construction might seriously affect urban transportation. In addition, the poor grouting effect was caused by grout running and grout leakage in the traditional grouting-reinforcement process. Therefore, it is difficult to avoid the risk of collapse during tunneling in the upper-soft, lower-hard composite strata without forming an effective overlying crust in the arch part of the tunnel.

MJS is a new reinforcement method developed based on traditional high-pressure jet-grouting technology. It can spray grouting from the ground at any angle to achieve the reinforcement of a large range of foundations in a narrow space. To control the influence of the construction on the culvert and the ground within a safe range, a plan combining MJS, composite foundation, and a raft was adopted in this project to reinforce the ground and the culvert to safeguard the normal function of the culvert and the highway during construction. Specifically, edge sealing was first conducted using the MJS oblique jet-grouting pile at a distance of 3 m from the tunnel lining. Subsequently, the sand layer within the edge-seal range was reinforced through sleeve-valve barrel grouting. To ensure the effect of the reinforcement, the disturbance and deformation of the stratum and bridge piles could be reduced during shield tunneling. Furthermore, to control the deformation of the bridge, rafts were set between the abutments on both sides of the original bridge and the culvert, connecting the abutments and pile foundations on both sides as a large raft, thus enhancing the overall rigidity of the bridge and the culvert. Meanwhile, a composite foundation reinforced by jet-grouting piles was constructed between the two tunnels to elevate the bearing capacity of the foundation. The overall reinforcement plan is illustrated in Figure 5.

3. Monitoring and Analysis of Subsidence in Shield Tunneling Under-passing the Bridge and Culvert

3.1. Layout of Monitoring Points for Subsidence

In considering the surrounding environment grade and ensuring the operability of monitoring the bridge, the culvert, and the ground, the settlement of the abutment bottom and the expressway pavement were monitored. Ten measuring points were arranged at the bottom of each abutment, and one measuring point was arranged every 5 m in the subgrade section, as shown in Figure 6. The layout of the on-site monitoring points is illustrated in Figure 7; the precision level was used to observe and record the settlement (Figure 8). Moreover, there were two types of ground-subsidence monitoring points: longitudinal monitoring points along the tunnel axis direction and transverse monitoring points along the direction of the tunnel section. The longitudinal monitoring points were set every 5 m along the tunnel alignment. The horizontal monitoring points were arranged every 5 m on both sides from the ground surface corresponding to the tunnel vault, and the density of the ground monitoring points near the top of the vault was appropriately increased. The layout is shown in Figures 9 and 10.
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Figure 6. Layout of measuring points in abutment.

Figure 7. Arrangement of abutment monitoring points on site.
3.2. Monitoring and Analysis of Ground Subsidence

Based on the geotechnical and field investigations, five excavation faces with different soft-rock height ratios of the left line tunnel were selected for monitoring to understand the relationship between the soft-rock height ratio of the excavation face and the ground subsidence; these excavation faces were all under the culvert. The ground monitoring points above the vault of these excavated faces were numbered as ZD134, ZD133, ZD131, ZD129, and ZD128, and the corresponding soft-rock height ratios of the excavated faces were 0, 0.20, 0.50, 0.80, and 1.00, respectively. Through the statistical analysis of the measured data of the ground surface settlement monitoring point above the excavation faces, the change law of the ground surface settlement of the transverse and longitudinal measuring points under the same shield tunneling measures was obtained.

3.2.1. Monitoring and Analysis of Surface Subsidence at Transverse Measuring Points

Figure 11 presents the transverse surface subsidence curves. The transverse surface subsidence curves of the five monitoring sections show that the surface above the tunnel vault has the largest settlement value. As the transverse monitoring point on the surface is far away from the tunnel vault, the surface settlement value gradually decreases. The surface settlement value near the tunnel vault changes faster than that far away from the tunnel vault. The measured surface settlement curve of the transverse monitoring point has an obvious settlement groove, which is consistent with the predicted curve of Peck [23]. In addition, owing to the influence of the soft-rock height ratio, the surface settlement value of the transverse measurement point increases with an increase in the soft-rock height ratio, and the settlement trough becomes deeper. When the soft-rock height ratio is 1, the settlement value reaches the maximum value of 7.97 mm. When the soft-rock height ratio is within 0.5–1.0 and 0–0.2, the settlement value of the monitoring point far away from the tunnel vault is close. Because the soil on the right side of the left tunnel axis has experienced two tunneling disturbances of the leading tunnel and the following tunnel, the surface settlement value of the soil in this area is slightly larger than that of the soil on the left. In the range of 7–12 m from the left and right sides of the tunnel axis, the settlement of the soil surface increases at different degrees, and the settlement amplitude increases with an increase in the height ratio of the soft rock.
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![Figure 11. Transverse surface subsidence curve.](image)

3.2.2. Monitoring and Analysis of Longitudinal Ground Subsidence

The curve shown in Figure 12 demonstrates the surface subsidence at longitudinal measuring points under different soft-rock height ratios during the shield-tunneling process. Moreover, the measuring point has an obvious heave in the range of 15 to 20 m away from the heading face owing to the effect of tunneling pressure, which belongs to the negative stratum loss. At the site 5 m before the excavation face, the ground surface...
deformation of the measuring point changes from heave to subsidence, presenting an accelerating trend. The ground surface deformation in this section is subsidence deformation resulting from the structural gap of the shield tunnel. Within 5 to 10 m behind the excavation face, the ground subsidence changes remarkably, with a maximum settlement value of 1.6 mm. This is a normal stratum loss. When the shield tunnel is excavated 20 m ahead of the measuring point, ground subsidence at the measuring point is no longer affected by shield excavation. Longitudinal subsidence is significantly affected by the soft-rock height ratio, and its ground subsidence increases with the increasing soft-rock height ratio. The longitudinal ground subsidence curves of the tunnel sections with varied soft-rock height ratios were compared, and the results showed that with an increase in the soft-rock height ratio, the surface settlement value of 5–10 m before the excavation became smaller. The subsidence rate of the ground behind the excavation face increased with the increasing soft-rock height ratio. In general, the degree and scope of influence of the shield tunnel on the longitudinal ground subsidence increased with the increasing soft-rock height ratio.

The curve in Figure 13 is the final subsidence value of each longitudinal monitoring point in the left tunnel. The monitoring points within 4–12 are the points of the shield tunnel under-passing the bridge and the culvert area of the highway. The geological map below the curve shows that the excavation face where the monitoring points 1 to 8 were located is covered with soft rock. In the area, the maximum subsidence of 14.8 mm could be found at monitoring point 3. The subsidence value of monitoring point 7, which had similar geological conditions to monitoring point 3, was only 7.08 mm. This proves that the surface subsidence was well controlled after the stratum was strengthened based on the MJS method. The monitoring points 8–12 were in the upper-soft, lower-hard composite strata. As the soft-rock height ratio decreased along the tunnel driving direction, the subsidence value of the corresponding monitoring points also decreased. Because foundation reinforcement was not performed in the area after monitoring point 12, and the geological condition changed from hard rock to soft rock, the ground subsidence value increased rapidly and returned to the subsidence value before entering the reinforced area.

Figure 12. Longitudinal surface subsidence curve.
3.3. Monitoring and Analysis of Subsidence in Bridge and Culvert

When monitoring the abutment subsidence, the data of each monitoring point were collected 12 times, once a day, in the process of the shield tunneling under-passing of the culvert. The subsidence curve was plotted with the monitoring data of the three measuring points at the left and right abutments 1, 8, and 10, as shown in Figure 14. The curve indicates that neither continuous sinking nor rising occurs at the abutment in the shield-construction process. The maximum subsidence value is 1.98 mm. With the shield tunneling, after the third and ninth day of monitoring, the abutments were lifted to varying degrees, with the lifting amount not exceeding 1 mm. The second lifting amount was smaller than the first, and the maximum floating value of the position was 0.57 mm. The maximum relative settlement between the monitoring points with the same serial number on the left and right abutments was 1.29 mm, exerting no effect on the safety of the bridge structure. In general, the greater the soft-rock height ratio of the shield-heading face at the location of the culvert, the larger the subsidence value of the culvert. However, during the process of shield-driving through the pile foundation, the influence of stress release caused by the tunnelling on the culvert was greater than that of the change in the height ratio of the soft rock on the excavation face. Therefore, the final settlement value of the monitoring point “L-10” was larger than that of the monitoring point “L-08,” even though the height ratio of the soft rock on the tunnel-excavation face corresponding to the monitoring point “L-08” on the left abutment was higher than that of the monitoring point “L-10.”

Figure 14. Abutment settlement curve.
4. Numerical Simulation

4.1. Establishment of the Finite Element Model

A 3D finite element model was established using the finite element software Midas GTS NX to simulate the construction process of the shield tunneling under-passing of the culvert. The grid covers an area that is six times the radius of the tunnel, according to geomechanical theory. Based on practical engineering, the stratum size of the model was set at 90 m in the X direction, 80 m in the Y direction, and 25 m in the Z direction. The tunnel overburden was 8 m in height; the soft-rock height ratio of the excavation face $\beta$ was 0.5, and the distance between the outer edge of the left and the right tunnel was 5.5 m.

The soil layer, the tunnel segments, the grouting layer, the pedestrian culvert (incl. the foundation and upper structure), the roadbed and pavement structure, the grouting reinforcement body of the MJS and the sleeve-valve pipe, the composite foundation of the jet grouting pile, and the raft with the planting bar needed to be considered in the model. Among them, the stratum based on the elasto-plastic deformation adopts the Mohr–Coulomb criterion. The reinforced body, the segments, and the other structures, considering only their elastic working, utilized the constitutive relationship of linear elasticity. According to the results [34] of the load test on the segments in the staggered joint assembly in the literature, the reduction in the circumferential and longitudinal stiffness had to be considered in the assembly of the shield segments. In this study, the reduction factor was taken as 0.7. The parameters of the stratum and main materials are listed in Table 1 (the definition of the “equivalent layer” shown in Table 1 is explained in Section 4.2). The groundwater level was set to 4.7 m below the surface. The overall model is shown in Figure 15.

Table 1. Stratigraphic and main material parameters.

| Name of Soil or Structure         | Modulus of Elasticity (MPa) | Poisson's Ratio | Severe (KN/m$^3$) | Cohesive Force (kPa) | Internal Friction Angle (°) |
|-----------------------------------|-----------------------------|-----------------|-------------------|----------------------|-----------------------------|
| <1> Miscellaneous fill            | 2.5                         | 0.35            | 16.5              | 8                    | 10                          |
| <4-2B> Silty sand                 | 5                           | 0.42            | 17                | 12.6                 | 12.8                        |
| <3-1> Silty-fine sand             | 12                          | 0.25            | 20.3              | 15                   | 35                          |
| <3-3> Gravelly sand               | 40                          | 0.30            | 19.6              | 10                   | 30.5                        |
| <9C-2> Slightly weathered limestone | 10,000                     | 0.30            | 25                | 1500                 | 55                          |
| Grouting reinforcement material   | 2000                        | 0.23            | 20                | -                    | -                           |
| Raft                              | 30,000                      | 0.3             | 25                | -                    | -                           |
| Segment                           | $34,500 \times 0.7$         | 0.2             | 25                | -                    | -                           |
| Equivalent layer                  | 13                          | 0.3             | 22.5              | -                    | -                           |

Figure 15. Structure diagram of 3D model.
4.2. Simulation of Related Working Conditions

In the simulation of the shield-tunnel construction, two shield tunnels were excavated. On the right was the tunnel built first, and on the left was the tunnel built later. Each shield tunnel was 78 m in length and divided into 26 excavation sections; that is, each excavation section was 3 m in length. A soil pressure of 0.18 MPa and a grouting pressure of 0.15 MPa were considered in the process of simulating the tunnel excavation. In the practical construction, synchronous grouting of a segment was a complicated process, which was simulated using the “presumption of equivalent replacement layer” in the finite element model. The “equivalent layer” was made of a mixed material of soil and cement mortar. The values of the elastic modulus and Poisson’s ratio referred to the cement soil, and the thickness was calculated as follows:

\[ \delta = \eta A \]  

where \( \delta \) is the thickness of the equivalent layer, \( A \) is the clearance of the shield tail construction, which is half of the difference between the outer diameter of the shield and the outer diameter of the segment, and \( \eta \) is the correction factor.

The range of 0.7–2.0 for the coefficient \( \eta \) given by the Japanese shield regulations based on many shield excavation examples was adopted. Sandy soil and part of the hard rocks formed most of the soil layers of the excavation face in the project. \( \eta \) was considered to be 1. In that case, the thickness of the “equivalent layer” was taken as \( \delta = 0.3 \) m. The hardening process of the grout must be considered in shield tail grouting, which is achieved by reducing the grouting pressure and enhancing the strength of the “equivalent layer.” At the early stage of grouting, the strength of the cement mortar is low with a high grouting pressure. It can ensure that the “equivalent layer” can fill the clearance of the shield tail and reduce the stratum loss. As the cement mortar hardens, the strength of the “equivalent layer” reaches the final point, and the radial grouting pressure decreases and dissipates to 0 MPa. Based on previous experiences [34], this assumption can effectively simulate the synchronous grouting of segments in the construction of a shield tunnel.

The working conditions simulated in this study involved bridge and culvert construction, shield tunnel excavation, and body reinforcement. Specifically, the pile foundation was simulated by beam elements, the raft was simulated by solid elements, the segment was simulated by slab elements, and the equivalent layer was simulated by solid elements. Regarding the foundation-reinforcement plan consisting of the MJS method and the double-pipe jet-grouting pile, reinforcement construction was implemented by activating the working conditions, which involved “changing the original soil mass parameters to the reinforcement parameters.” The structures of the bridges, the culverts, and the reinforcement bodies were simulated, as shown in Figure 16.

![Figure 16. Main structure model and solid simulation diagram.](image-url)
4.3. Ground Subsidence

The displacement cloud chart in the Z direction of the stratum under the unreinforced and reinforced conditions is shown in Figure 17. The 13th excavation section (underpass culvert section) of the left line tunnel was selected for observation and analysis. Based on the nephogram, stratum floating could be found in local areas of the culvert exit with amounts up to 22.4 mm under the unreinforced condition. Moreover, the maximum subsidence could reach as much as 219 mm because its sand layer was located above the tunnel, and the groundwater was abundant. In other words, the shield passing underneath the culvert without ground reinforcement may lead to foundation instability or ground collapse. In the reinforced condition, the maximum subsidence of the stratum can be controlled at 8.86 mm.

Figure 17. Z-direction displacement cloud image before and after reinforcement: (a) no reinforcement; (b) with reinforcement.

Figure 18 shows the transverse ground settlement curves of the 13th excavation section of the tunnel on the left line with and without reinforcement. In the case of no reinforcement, the maximum surface settlement appeared above the rear tunnel vault with a settlement value of 11.6 mm and a formation loss rate of 0.19%. In the right area of the left tunnel vault, the surface settlement value decreased sharply to 2.65 mm owing to the release of the stress concentration. If the difference of the settlement value is too large, the corresponding superstructure produces extra stress, and when it exceeds a certain limit, cracks, tilt, and even damages occur. Strengthening the section reduced the formation loss rate by more than 50%. The maximum surface settlement was 5.30 mm. This maximum settlement value was close to the surface settlement value above the left tunnel vault with a soft-rock height ratio of 0.5 in the measured monitoring data. It did not appear to have a significant differential settlement value; thus, the MJS method of reinforcement significantly controls the influence of the surface stress concentration of release.

Figure 18. Transverse surface settlement curve of the left and right tunnel excavation at the 13th step.
4.4. Deformation of Tunnel Segments

The structural safety of the tunnel is essential to prevent excessive disturbance to the mounted segments from the concentrated release of stress in the process of the shield tunneling under-passing of the culvert at a close distance. Moreover, the axial deformation of the segments requires significant control. In Figure 19, the AB and CD survey lines of the right tunnel along the tunnel axis were selected to analyze the displacement changes in the right tunnel segment before and after the excavation of the left tunnel. Specifically, the AB and CD survey lines were used to analyze the deformation in the Z and X directions, respectively.

![Figure 19. Drawing of prior-dug tunnel segment monitoring: (a) schematic of the right tunnel survey line; (b) position map of the survey line.](image)

The displacement curves of the right segments before and after the excavation of the left tunnel in the Z-direction are shown in Figure 20. The maximum subsidence of the 19th–21st ring segment of the right tunnel was 25.1 mm before the excavation of the left tunnel in the unreinforced condition. Meanwhile, the maximum relative displacement of the adjacent segments was 17.4 mm. This indicates that a large subsidence of segments was caused by the concentrated release of stress when the shield tunnel was out of the culvert section, which might further lead to a large dislocation of the adjacent segments, threatening tunnel safety. In the unreinforced condition, additional longitudinal displacement of the 4th–26th ring segments of the right tunnel was caused by the excavation of the left tunnel. The maximum additional longitudinal displacement was -4.7 mm. After reinforcement with the MJS method, an additional vertical displacement was found only at the 8th–21st ring segments of the right tunnel produced together with a significant drop in the displacement value. The maximum additional vertical displacement was only -0.5 mm. Hence, the MJS method can markedly control the disturbance of the newly built tunnel to the proceeding tunnel. The vertical displacement of the segment corresponding to the right tunnel first increased and then decreased in the under-passing culvert process of the left tunnel. The maximum additional displacement could be observed at the 15th ring segment at the center of the culvert.

The displacement curve of the right segments before and after the excavation of the left tunnel in the X-direction is shown in Figure 21. In the absence of reinforcement, the maximum displacement in the Xdirection was -2.7 mm before the excavation of the left tunnel; the maximum displacement in the X direction after the excavation of the left tunnel was -4.6 mm; the maximum additional displacement was -2.1 mm, which was located near the 13th ring segment. In general, three disturbances might be generated during the construction process of the newly built tunnel to the preceding tunnel. More precisely, the first disturbance results from an increase in the soil pressure on the segment of the preceding tunnel owing to external loads, such as the excavation pressure and the jack force exerted by the shield machine on the heading face of the newly built tunnel. The second
disturbance stems from a decrease in the surrounding rock pressure of the preceding tunnel owing to soil mass unloading after the soil excavation of the newly built tunnel. The third disturbance emanates from the soil mass around the preceding tunnel rebounding locally after the shield machine leaves the heading face when the segment of the newly built tunnel is mounted. The above disturbances increase with increasing axial distance between the preceding tunnel and the newly built tunnel. This shows that the segment of the preceding tunnel is most affected by the second disturbance in the construction of the newly built tunnel, and the segment is displaced toward the newly built tunnel. After being reinforced by the MJS method, the lateral displacement of the preceding tunnel was significantly decreased, and the maximum displacement before and after the excavation of the newly built tunnel was only $-0.6 \text{ mm}$ without an additional displacement. Moreover, The MJS method can “increase” the distance between the two tunnels and reduce the disturbance caused by double-track tunnel excavation to the leading tunnel.

![Figure 20. Tunnel displacement curve in the Z direction.](image)

![Figure 21. Tunnel displacement curve in the X direction.](image)
4.5. Pile Foundation Deformation and Internal Force Analysis

The subsidence cloud charts of the pile foundations of the culvert after the shield excavation in the reinforced and unreinforced conditions are shown in Figure 22. Before reinforcement, tunnel excavation leads to a decrease in soil pressure along the pile, affecting the soil-friction effect. Moreover, it might result in pile foundation sinking, with a maximum subsidence of 43.69 mm and a minimum subsidence of 20.28 mm. After reinforcement based on the MJS method, the subsidence of the pile foundation was controlled to more than 3.14 mm and less than 8.51 mm. The maximum subsidence was reduced by 80%. Moreover, the subsidence difference of the pile foundation was reduced from 23.41 mm to 5.37 mm, which could effectively avoid the cracked upper structure caused by the excessively large subsidence difference of the pile foundation.

Figure 22. Settlement cloud image of bridge and culvert pile foundation: (a) not reinforced; (b) reinforced.

The horizontal displacement cloud charts of the pile foundations of the culvert in the X direction after shield excavation in the reinforced and unreinforced conditions are shown in Figure 23. According to the cloud chart, the displacement of the pile foundation in the X direction occurs at the bottom of the pile, moving towards the shield tunnel. This is because the loss of the upper stratum is caused by the shield-tunnel excavation, and the soil mass above the tunnel is unloaded. Before reinforcement, the maximum horizontal displacement of the pile foundation was 26.6 mm. After reinforcement, the maximum horizontal displacement was only 4.2 mm, which was a reduction of 84.2%.

Figure 23. Cloud diagram of horizontal displacement of bridge and culvert pile foundation: (a) not reinforced; (b) reinforced.

A pile foundation closest to the shield tunnel at the 13th excavation step of the left tunnel was selected, as shown in Figure 24. The curve shows the changes in the bending moment of the pile foundation before and after the shield construction. In the unreinforced condition, the negative bending moment of the pile foundation was increased by more
than 150 kN·m, and the positive bending moment was increased by more than 50 kN·m after the shield construction. Conversely, in the reinforced condition, the bending moment curves of the pile foundation before and after shield excavation nearly overlapped; that is, the additional bending moment could be ignored. This shows that the piles reinforced by the MJS method can “offset” the soil pressure generated by the shield excavations.

![Bending moment curves](image)

**Figure 24.** 13th excavation step of the rear tunnel corresponds to the bending moment curve of the pile foundation.

### 4.6. Analysis of Deformed Culverts

The subsidence cloud chart of the culvert after shield tunnel excavation is shown in Figure 25. The subsidence of the culvert presents a rising trend from entering the culvert to exiting the culvert as the continuous shield excavation leads to multiple disturbances in the soil. As a result, subsidence in the culvert increased. In the unreinforced condition, the maximum and minimum subsidence values were 39.92 mm and 22.46 mm, respectively. The subsidence difference between adjacent locations can reach 4.5 mm, posing a certain threat to the culvert structure. Moreover, the locations where the culvert has a large subsidence value are close to the abutment, indicating that most of the disturbances caused by the shield construction to the culvert are transmitted through the pile foundation. In the reinforcement plan, the subsidence distribution of the culvert was relatively uniform as a raft was set between two abutments to improve the integrity of the culvert. The maximum value was only 5.81 mm, indicating that the influence of the shield excavation on the culvert can be greatly lowered owing to the reinforcement with the MJS method.

![Culvert displacement images](image)

**Figure 25.** Culvert settlement cloud image after the completion of the underpass: (a) not reinforced; (b) reinforced.
5. Conclusions

The near-range underpass of the shield tunnel has a significant influence on buildings, which might lead to considerable disturbances on the ground and significant displacement of the segments and buildings. This paper focuses on the slurry-shield tunneling construction under-passing the pedestrian culvert of the North Ring Road at the upper-soft, lower-hard composite strata in the Tongdewei-Shangbu section of the northern extension of Guangzhou Metro Line 8. The influences of the MJS method on ground subsidence, displacement of segments, bridges, and culverts, as well as the control of inner stress using field monitoring and numerical simulation analysis, were evaluated. The main conclusions are as follows.

1. After the construction of the left tunnel, when the soft-rock height ratio was 1.0, the surface attained the maximum subsidence value of 7.97 mm, showing that the ground subsidence value was in a safe range. However, when the soft-rock height ratios of the adjacent excavation faces ranged from 0 to 0.2 and from 0.5 to 1 special attention was given to the differential ground subsidence that might be generated in the subsequent shield construction.

2. The concentrated release of stress might occur at the beginning and end stages of the shield construction underneath, which has a greater impact on the structure than the geological conditions of the shield-tunneling face. Hence, ground subsidence at the beginning and ending stages needs more concern, even if the shield tunnel is under-passing the building under favorable geological conditions.

3. Reinforcing with the MJS method could effectively control the concentrated release of stress that may be caused by the shield-tunneling project. Moreover, it not only reduces the disturbance to the preceding tunnel during the excavation of the double-line tunnel, but also minimizes the pressure loss of the tunnel’s surrounding rock and the maximum displacement of the adjacent segments caused by the shield excavation. Thus, the safety of tunneling projects can be enhanced.

4. The displacement of the pile foundation in the under-passing project was reduced by more than 80% based on reinforcement using the MJS method. Moreover, no additional bending moment was applied to the pile foundation, indicating that the strength of the MJS reinforcement body can be lowered to reduce the construction cost in practice.

5. The ground subsidence of the left tunnel vault was 5.30 mm in the numerical simulation, which was consistent with that of the newly built tunnel vault of the monitoring section at a soft-rock height ratio of 0.5 in field monitoring. This proves the effectiveness of the numerical simulations conducted in this study.

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