

Research Article

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Impact of fully rotating steel casing bored pile on adjacent tunnels

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Abstract: Based on the theoretical model of a soil plug column, the stress analysis of the soil plug column during the spinning process of steel casing is carried out, and the critical depth of the soil column is determined using the stress and torsional shear ratio of the soil column. The effect of factors such as casing wall thickness, surface load, and steel casing spinning speed on the critical depth of soil columns has been explored, and more reasonable construction process parameters have been obtained quantitatively. Combined with the construction of small net distance test piles at a distance of 2.5 m from the tunnel, the impact of the construction process on the existing shield tunnel has been explored. The results indicate that during the construction process, when the wall thickness of the steel casing does not exceed 0.012 m, the surface load does not exceed 15 kPa, the spinning speed of the steel casing is maintained at 5/4/2/4 m/h or 5/3/2/3 m/h (corresponding to soil depths of 2.5/9.5/6/14 m), and the soil height of the soil column is controlled within 11 m, it is not easy to generate soil plug inside the steel casing, and the soil column has strong torsional shear resistance. According to the measured data of adjacent tunnels, it has been found that the construction method of fully rotating steel casing bored pile can effectively reduce the impact on adjacent shield tunnels, and has a good microdisturbance effect, which can control tunnel deformation not exceeding 1 mm and maintain within the warning value range.

Keywords: steel casing, fully rotating bored pile, critical depth of soil column, quantification, minor disturbances

1 Introduction

In recent years, with the continuous development of world urbanization, a large number of pile foundation projects have inevitably appeared around rail transit lines. For example, there are pile foundation construction projects on the side of existing shield tunnels, and the impact of pile foundation construction on adjacent tunnels is becoming increasingly prominent. Obviously, the construction of a pile foundation is prone to disturb the surrounding soil, resulting in significant additional stress on adjacent tunnels, resulting in problems such as misalignment, uneven settlement, damage to pipe segments, and leakage of water in the tunnel structure. Domestic and foreign scholars mainly studied the impact of pile foundation construction on adjacent tunnels based on the analytic solution method, model test, numerical simulation, and field measurement.

Analytical solution methods are widely used in analyzing the impact of pile foundation construction on tunnels. Song et al. [1] analyzed the impact of different construction sequences of pile groups on multiple existing shield tunnels through theoretical analysis and numerical simulation methods, and obtained a relatively optimal construction sequence plan for pile groups. Based on the extended shear displacement method and the two-parameter elastic foundation beam model, Wang et al. [2] proposed a simplified analytic solution of tunnel longitudinal settlement caused by an axial load of the single pile or pile group, and verified the analytic solution with three-dimensional finite element analysis method. The research results show that the proposed analytic solution can be easily used for the preliminary estimation of tunnel longitudinal settlement caused by the axial load of the single pile or pile group. In order to clarify the impact of pile foundation construction on the lateral stress and deformation characteristics of tunnel segments, Zhang et al. [3] proposed a method for calculating the additional confining pressure caused by

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the construction of bridge pile steel sleeves in existing tunnels, and conducted three-dimensional numerical simulation analysis of tunnel segments based on the shell spring model. The research results indicate that pile foundation construction can cause the lateral rotation of tunnel segments to occur first clockwise and then counterclockwise. Based on the pile foundation engineering of a nearby subway tunnel in Hangzhou, Ding et al. [4] combined the Mindlin solution with a two-stage analysis method to solve the additional stress and longitudinal deformation of the existing tunnel caused by pile foundation construction. The research results show that the existing tunnel undergoes a process of first uplift, then settlement, and finally deformation and stability. The side friction caused by pile foundation construction is the main factor causing the deformation of the adjacent tunnel. Although analytical solutions have the characteristics of simplicity and convenient engineering applications, their applicability may be greatly reduced due to the fact that the derivation process of analytical solutions generally requires the introduction of a large number of assumptions.

The model testing method is also an important method for analyzing the impact of pile foundations on tunnels. Kong et al. [5] used indoor model tests and finite element numerical calculation methods to study the impact of pile foundation construction on adjacent existing tunnels. In the study, two variables, namely, the distance between the pile and tunnel and the pile top load, were mainly considered. Weng et al. [6] mainly considered the pile top load and the distance between the pile and tunnel, and used a centrifugal model to test the impact of the pile foundation loading process on adjacent existing tunnels. The research results showed that the deformation of the tunnel caused by the pile top load is mainly caused by longitudinal settlement deformation, and the lateral deformation is relatively small. Jin et al. [7] mainly considered the pile length, the distance between pile and tunnel, and relative depth between pile and tunnel and conducted research on the impact of pile foundation loading on adjacent existing tunnels using centrifugal model tests and numerical simulations. The research results show that the longitudinal bending moment of the tunnel decreases with an increase of the distance between the pile and tunnel. When the pile and tunnel are at the same depth, the longitudinal bending moment of the tunnel is the largest. When the pile end is above the tunnel, the longitudinal bending moment of the tunnel is greater than when the pile end is below the tunnel. Mahajan et al. [8] used a scaled model test to study the effect of pile foundation loading on adjacent tunnels at different distances between the pile and tunnel. The research results showed that when

the distance between the pile and tunnel exceeds 12 times the pile diameter, the effect of pile foundation loading on adjacent tunnels is relatively small. The model testing process is generally unable to simulate complex on-site construction environments and construction steps, which makes the corresponding research results less valuable for promotion.

Among all the methods for evaluating the impact of pile foundations on tunnels, the numerical calculation method is the most widely used method. Yoo [9] used a three-dimensional finite element method to simulate the impact of pile-supported bridge construction on the tunnel. The research results showed that when the horizontal net distance between the pile and tunnel is less than 1.0 times, the tunnel diameter and the vertical net distance is less than 0.5 times the tunnel diameter, the impact of pile-supported bridge construction on the tunnel can be ignored. On the basis of summarizing a series of three-dimensional numerical calculation results, Lueprasert et al. [10] proposed an evaluation method for tunnel deformation, which includes the maximum lateral shrinkage rate and maximum elongation in the longitudinal direction of the tunnel. Considering a nearby construction project as the background, Lv [11] used numerical simulation methods to analyze the impact of bridge pile foundation construction on the internal force and displacement of the tunnel structure, so as to further optimize the bridge design and construction plan and reduce the impact of nearby construction. Considering the Mrta tunnel project as the background, Heama et al. [12,13] used a three-dimensional finite element numerical calculation method to clarify the impact of pile foundation loading on existing tunnels. The research results indicate that when the horizontal net distance between piles and tunnels is greater than 3.5 m, the influence of tunnel deformation and additional stress is relatively small, and the tunnel deformation and additional stress increase with the increase of the number of loaded pile foundations. Taking the tunnel project of Wuxi Metro Line 2 as the background, Liu et al. [14] used the finite element numerical calculation method to study the influence mechanism of static piling on the tunnel structure. The research results showed that static piling will lead to compression deformation in the transverse direction and tensile deformation in the longitudinal direction of the existing tunnel. Nematollahi and Dias [15] compared the performance of different soil constitutive models in calculating the impact of pile foundation loading on tunnels, and the research results showed that the Mohr Coulomb model is not suitable for this type of calculation. Heama et al. [16] compared the performance of 3D finite element and 2D finite element techniques in calculating the impact of pile foundation loading on tunnels. The research results showed

that if only the tunnel response is concerned, the 2D finite element calculation method can be used. If pile foundation responses also need to be considered, the 3D finite element calculation method must be used. Lueprasert et al. [17] used numerical calculation methods to study the effect of pile foundation loading at different pile end positions on adjacent tunnels. Wang and Yuan [18] used numerical calculation method and field test method to study the interaction mechanism between piles and shield machines. Lin et al. [19] used a new numerical calculation method to compare the impact of the construction of static pressure piles, bag grouting piles, and bored piles on the surrounding strata. The results showed that the construction of bored piles had the least disturbance to the surrounding strata, bag grouting piles, and the construction of static pressure piles will cause the greatest disturbance to the surrounding strata. Although numerical calculation methods are widely used, the accuracy and reliability of the calculation results have a great relationship with the experience of the calculation personnel.

The on-site monitoring method is also a very important means to clarify the impact mechanism of pile foundation construction on tunnels. Huang et al. [20] combined steel casing and mud circulation soil sampling methods to reduce the impact of bored pile construction on adjacent tunnels. Zuo et al. [21] used on-site measurement methods to study the impact of punching pile construction on the surrounding soil. The results showed that the horizontal and vertical effective influence range of punching pile construction is three times the pile diameter, and the corresponding stress effect is smaller compared to the displacement effect. Xu and Wang [22] studied the adverse impact of adjacent pile foundation construction on the shield tunnel by using the on-site monitoring method. The research results showed that the maximum displacement of the tunnel segment occurs on the profile corresponding to the test pile. The segment mainly occurs with horizontal displacement, and the settlement is about 0.5 times of the horizontal displacement. Considering the viaduct reconstruction project of Beitang Road in Hangzhou as the background, combined with on-site measured data, Ding et al. [23] analyzed the impact of full casing bored pile construction on adjacent tunnels. The research results showed that during pile foundation construction, the displacement of the upper soil layer is relatively large, which has a significant impact on the settlement of the shallower buried roadbed, while the displacement of the deep soil layer is relatively small, which has a relatively small impact on the deeper buried tunnels. In order to improve the protection effect of subway tunnels, Wang et al. [24] used on-site measurement methods to study the deformation characteristics of subway caused

by pile foundation construction under different net distances and pile foundation types. The research results showed that when the net distances between piles and tunnels were 5, 12, and 20 m, full casing-full rotation, full rotation-half casing, and conventional rotary-drilling rig construction techniques should be used, respectively. Based on on-site measured data, Benoto bored pile and mud circulation soil sampling technology were used for pile foundation construction. Gao et al. [25] evaluated the impact of pile construction on adjacent tunnels. Considering the Changzhou tunnel project as the background, Yang et al. [26] used on-site measured data to analyze the disturbance mechanism of near distance pile foundation construction on adjacent tunnels. The research results showed that the main construction impact range of casing piles is six times the pile diameter.

Based on the existing research results, the current research mainly focuses on the impact of pile foundation loading on adjacent tunnels. However, in practical engineering, the engineering community is more concerned about the impact of pile foundation construction on adjacent tunnels, especially how to adopt effective construction parameters to control the impact on tunnels. Although the construction technology of steel casing bored pile has little impact on adjacent buildings and structures, the protection of surrounding buildings and structures during the construction process cannot be ignored for small net distance conditions. This study aims to conduct a quantitative study on the construction of small net distance full rotary steel casing bored piles in conjunction with Hangzhou Metro Line 1. The objective is to establish appropriate construction process parameters through theoretical calculations, thereby minimizing the adverse effects of soil disturbance on the existing tunnel during construction.

This study seeks to formulate a theoretical model for soil plug columns and assess the influence of fully rotating steel casing bored piles on soil subjected to varying construction factors. The objective is to ascertain more rational construction process parameters. Furthermore, by integrating on-site pile testing construction and monitoring data from adjacent tunnels, we investigate the effects of cast-in-place pile construction on nearby existing shield tunnels.

2 Project overview

As shown in Figure 1, the project is located in Hangzhou, Zhejiang Province, China. The exhibition center project includes eight steel structure exhibition hall main buildings, one central corridor, two login halls in the east and west, an underground parking garage, and three connecting passages



Figure 1: Project location [27,28].

connecting the north and south underground warehouses. The foundation form of this project is the pile raft foundation, which supports the basement pile foundation in the exhibition hall and subway protection area using the construction process of fully rotating steel casing bored pile, while the ordinary basement uses prestressed high-strength concrete pipe piles. The Hangzhou Metro Line 1, which has been put into operation, runs across the whole building from the central corridor. As shown in Figure 2, the foundation pit is divided into three parts, namely, the non-subway protection area, the subway protection area, and the connecting passage area.

The diameter of the subway shield tunnel is 6 m, and the minimum net distance between the two tunnels is

about 6 m. The distance from the ground to the top of the tunnel is approximately 12 m, while the minimum clearance between the bored pile and the tunnel is 2.5 m. Considering the requirements for minor disturbance construction in the subway protection area and the presence of thick sandy silt and silty clay layers in the area, the traditional mud protection wall drilling and pouring pile technology can easily lead to shrinkage or slot wall collapse, thereby affecting the safety of tunnel operation [24]. Therefore, this project plans to use fully rotating steel casing bored piles [3,23], which have a minimum length of 75 m and a diameter of 1 m. Additionally, a 32 m-long steel casing will be employed for the upper section. The engineering profile is shown in Figure 3, and the geological

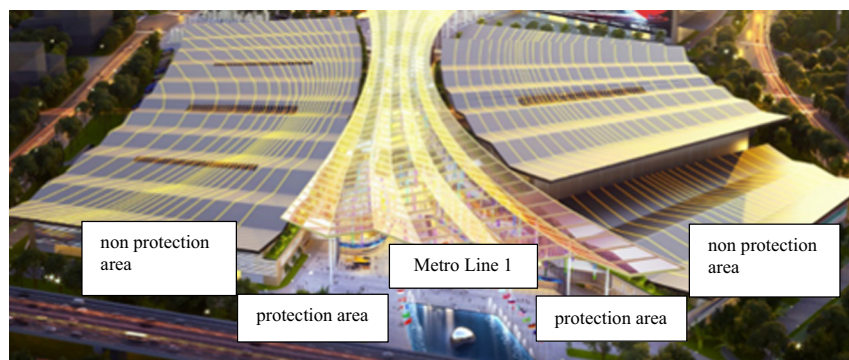


Figure 2: Layout plan of the exhibition hall and metro line.

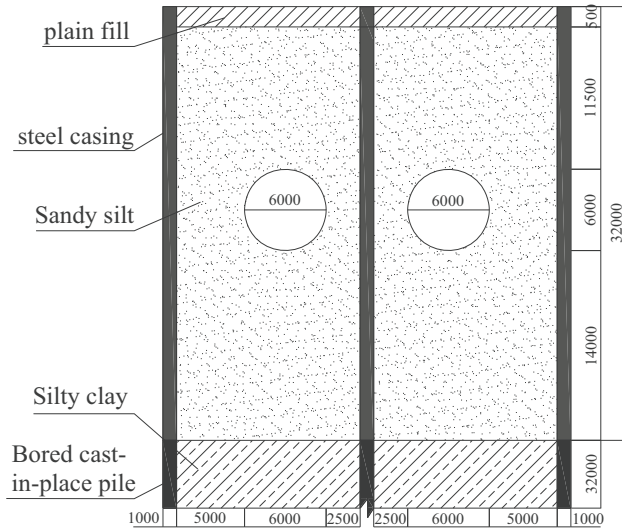


Figure 3: Geological profile.

conditions within the construction range of the steel casing are shown in Table 1.

The primary challenge presented by this project is the net distance of 2.5 m between the bored pile and the tunnel. Throughout the construction phase, several factors, including ground overload, soil stress release during hole formation, the speed of hole formation, and the uninterrupted operation of the subway, can significantly affect the subway tunnel. Hence, a careful selection of appropriate construction parameters for bored piles is of utmost importance in order to minimize the adverse effects caused by the construction of closely spaced bored piles on existing shield tunnels.

3 Analysis of construction technology for bored piles

3.1 Theoretical model of the soil plug column

In this study, the potential occurrence of soil blockage during the rotation and compression process of the steel

casing, known as the soil plug column, is investigated. First, it is necessary to determine the relationship between the cumulative vertical compressive stress of the soil column at different depths of the steel casing and the soil layer stress through force analysis. This article mainly considers the influence of the casing wall thickness and surface load on the vertical compressive stress during the rotation and compression process of the steel casing. Additionally, the study aims to determine whether the soil plug column is prone to torsional shear failure during this process, and it is necessary to determine the relationship between the cumulative torsional shear moment of the soil plug column at different depths and the torsional shear strength of the soil column through force analysis, so as to determine the critical depth corresponding to the shear of the soil column.

The stress analysis of the soil plug column corresponding to the spinning depth z of the steel casing is as follows [1]:

$$AdP_i = \sum_{i=1}^n \frac{a^3}{1 + k_i^3} \left[\left(\frac{E_i t}{D} + \frac{\mu_i}{1 - \mu_i} P_i \right) \tan \phi_i + c_i \right] U dz_i + \gamma_i A dz_i. \quad (1)$$

Let

$$\varepsilon_{1i} = \frac{a^3}{1 + k_i^3} \left(\frac{E_i t}{D} \tan \phi_i U + c_i U + \gamma_i A \right), \quad (2)$$

$$\varepsilon_{2i} = \frac{a^3}{1 + k_i^3} \frac{\mu_i}{1 - \mu_i} \tan \phi_i U. \quad (3)$$

Then, equation (1) is transformed into

$$AdP_i = \sum_{i=1}^n (\varepsilon_{1i} dz_i + \varepsilon_{2i} P_i dz_i). \quad (4)$$

The general solution for equation (4) is

$$P = \sum_{i=1}^n \left[e^{\frac{\varepsilon_{2i}}{A} z_i} \left(-\frac{\varepsilon_{1i}}{\varepsilon_{2i}} e^{-\frac{\varepsilon_{2i}}{A} z_i} + C_1 \right) \right]. \quad (5)$$

Considering that when $z = 0$ m, $P = P_0$, then

$$C_1 = P_0 + \frac{\varepsilon_{1i}}{\varepsilon_{2i}}. \quad (6)$$

Table 1: Soil layer-related parameters

Soil layer name	Thickness z (m)	Elastic modulus E (MPa)	Unit weight γ (kN/m ³)	Poisson's ratio μ	Cohesive force c (kPa)	Internal friction angle ϕ (°)
Plain fill	0.5	3.0	18.6	0.20	10.0	20.0
Sandy silt	31.5	10.0	20.0	0.25	9.8	31.0
Silty clay	32.0	8.0	20.0	0.31	42.0	21.0
Calcareous siltstone	56.0	300.0	20.0	0.30	200.0	32.0

Substituting equation (6) into equation (5) yields

$$P = \sum_{i=1}^n \left[\frac{\varepsilon_{1i}}{\varepsilon_{2i}} \left(e^{\frac{\varepsilon_{2i}}{A} z_i} - 1 \right) + P_0 e^{\frac{\varepsilon_{2i}}{A} z_i} \right]. \quad (7)$$

The self-weight stress of the soil layer is calculated as follows:

$$p = P_0 + \sum_{i=1}^n \gamma_i z_i. \quad (8)$$

The stress ratio of soil column is

$$R_p = \frac{P}{p}, \quad (9)$$

where A is the cross-sectional area of the soil column (m^2); P is the vertical stress at depth z_i (kPa); k_i is the spinning speed of the steel casing at depth z_i (m/h); a is the correction amount for the rotational pressing of the steel casing (m/h), taken as $a = 0.95$ m/h; E_i is the elastic modulus of the soil (MPa); t is the wall thickness of the casing (m); μ is the Poisson's ratio of soil mass; c_i is the soil cohesion (kPa); φ_i is the internal friction angle of the soil ($^\circ$); U is the perimeter for the inner wall of the casing (m); p is the self-weight stress of the soil layer (kPa); P_0 is the surface load (kPa); γ_i is the weight of the soil mass (kN/m^3); z_i is the calculated thickness of the i -th layer of soil (m); and R_p is the stress ratio of the soil column.

When the steel casing rotates and presses down to a certain depth, the torsional shear moment between the steel casing and the soil column will also increase to a certain extent. When the torsional shear moment exceeds the torsional shear strength of the soil column, the soil column will be cut off, and at this time, the soil column will rotate with the steel casing. In order to accurately determine the shear failure of soil plug columns under different rotational compression conditions, this article analyzes the ratio of cumulative torsional shear moment and torsional shear strength of soil columns at different depths of the steel casing, and finally, obtains the critical depth of the soil column.

The cumulative torsional shear moment of a soil plug column can be expressed as [1]

$$M_N = \frac{D}{2} \int_0^z \frac{a^2 k}{1 + k^3} \left[\left(\frac{Et}{D} + \frac{\mu}{1 - \mu} P \right) \tan \varphi + c \right] U dZ. \quad (10)$$

In the case of layered soil, the integration of equation (10) in discrete segments can be employed to obtain

$$M_N = \frac{DU}{2} \sum_{i=1}^n \frac{a^2 k_i}{1 + k_i^3} \left[\left(\frac{E_i t z_i}{D} + \frac{\mu_i}{1 - \mu_i} \left(\frac{\varepsilon_{1i}}{\varepsilon_{2i}} \left(e^{\frac{\varepsilon_{2i}}{A} z_i} - 1 \right) + \frac{A}{\varepsilon_{2i}} \right) \right) \tan \varphi_i + c_i z_i \right]. \quad (11)$$

The torsional shear strength of soil plug columns can be expressed as

$$M_K = \frac{\pi D^3}{12} \sum_{i=1}^n \left[c_i + \left(\frac{\varepsilon_{1i}}{\varepsilon_{2i}} \left(e^{\frac{\varepsilon_{2i}}{A} z_i} - 1 \right) + P_0 e^{\frac{\varepsilon_{2i}}{A} z_i} \right) \tan \varphi_i \right], \quad (12)$$

where M_N is the cumulative torsional shear moment of the soil column (kN m) and M_K is the torsional shear strength of the soil column (kN m).

The torsional shear ratio of soil columns is

$$R_M = \frac{M_N}{M_K}. \quad (13)$$

When $M_N \geq M_K$, it indicates that the soil plug column has undergone shear failure at depth z , and the depth of the soil layer at this time is recorded as the critical depth of the soil column. The stress ratio of soil column as shown in equation (9) and the torsional shear ratio of soil column as shown in equation (13) can be used to describe the degree of soil blockage in soil columns.

3.2 Analysis of influencing factors

In order to further explore the soil plug column during the rotation and compression process of the steel casing, this article focuses on the influence of the wall thickness of the steel casing, surface load, and rotation and compression speed on the soil plug column. The relevant influencing factor variables are shown in Table 2.

3.2.1 Influence of steel casing wall thickness

To investigate the effect of steel casing wall thickness on the soil plug column, this study assumes that the spinning speed of steel casing at different soil depths (2.5 m $\rightarrow$ 5 m/h, 9.5 m $\rightarrow$ 4 m/h, 6 m $\rightarrow$ 3 m/h, 14 m $\rightarrow$ 4 m/h) are fixed values. Equations (7) and (8) were used to calculate the cumulative vertical compressive stress and soil self-weight stress of soil columns at different depths, and the variation of soil column stress ratio R_p with soil depth is shown in Figure 4. The analysis of Figure 4 reveals a correlation between the

Table 2: Relevant variables of influencing factors

Variable	Value
Steel casing wall thickness t (m)	0.020; 0.016; 0.012
Surface load P_0 (kPa)	10; 15; 20; 25
Spinning speed of steel casing k_i (m/h)	5/4/3/4; 5/4/2/4; 5/3/
(single spinning depth of casing $z_i = 2.5$ m/9.5 m/6.0 m/14.0 m)	3/4; 5/3/3/3; 5/3/2/3

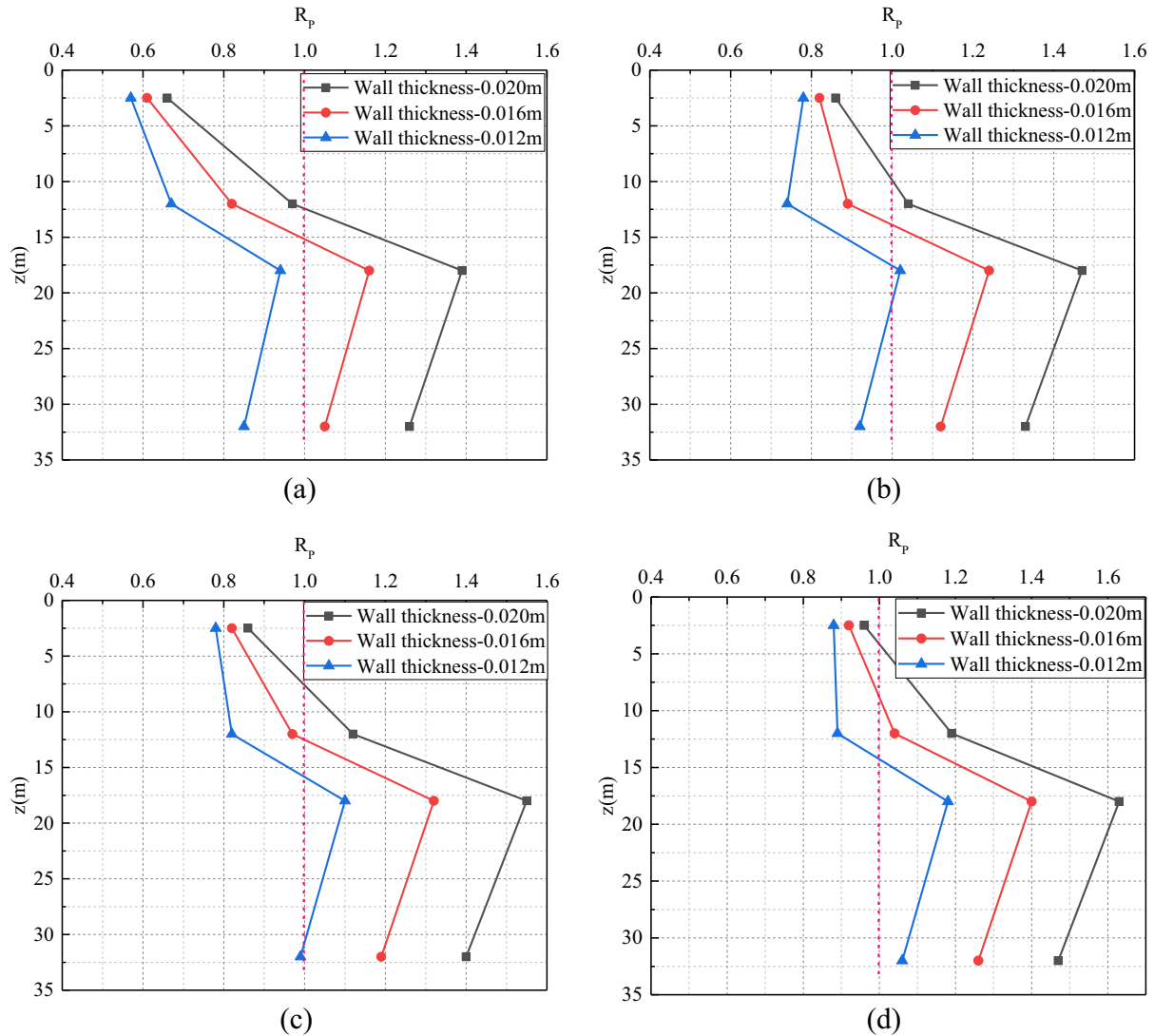


Figure 4: Stress analysis of soil column stress ratio R_p under at different wall thicknesses: (a) 10 kPa–5/4/3/4 m/h, (b) 15 kPa–5/4/3/4 m/h, (c) 20 kPa–5/4/3/4 m/h, and (d) 25 kPa–5/4/3/4 m/h.

wall thickness of the steel casing and the depth of soil at which the stress ratio R_p exceeds 1. As the wall thickness of the steel casing increases, the corresponding soil depth decreases. In other words, there exists an inverse relationship between the spinning depth of the soil plug column and the wall thickness of the steel casing. The larger the wall thickness, the more obvious the blocking effect of the soil column (i.e., $R_p > 1$). Therefore, it is recommended that the wall thickness of the steel casing be $t \leq 0.012$ m.

3.2.2 Influence of surface load

In addition, this study also assumes that the spinning speeds of the steel casing at different soil depths (2.5 m → 5 m/h, 9.5 m → 4 m/h, 6 m → 3 m/h, 14 m → 4 m/h) are fixed values,

and further explores the influence of different surface loads on the stress ratio R_p of the soil column. The findings are presented in Figure 5, which illustrated that as the surface load increased, the stress ratio R_p also increased. It was observed that the shallower the depth of the soil layer corresponding to $R_p > 1$, the earlier the manifestation of the soil plug column. Notably, when the wall thickness of the steel casing was set at $t = 0.012$ m and the surface load $P_0 \leq 15$ kPa, the soil plug column during the spinning process of the steel casing was not particularly significant ($R_p \leq 1$).

3.2.3 Influence of spinning speed on steel casing

Based on the exploration results in the previous section, considering the wall thickness of the steel casing $t = 0.012$ m

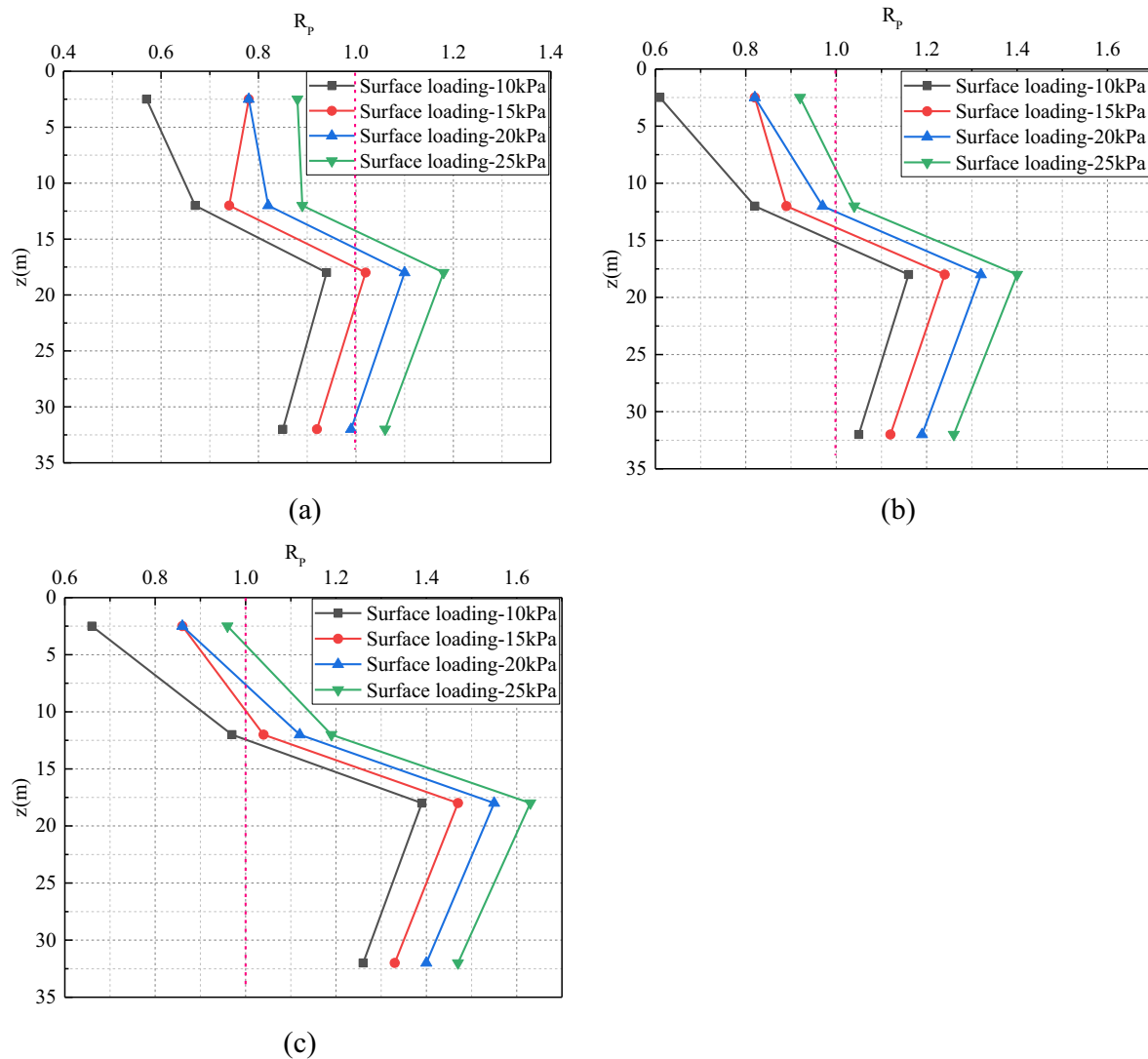


Figure 5: Analysis of the stress ratio R_p of soil columns at different surface loadings: (a) 0.012 m–5/4/3/4 m/h, (b) 0.016 m–5/4/3/4 m/h, and (c) 0.020 m–5/4/3/4 m/h.

and the surface load $P_0 \leq 15$ kPa as the prerequisite conditions, the cumulative torsional shear moment and torsional shear strength of the soil column at different depths were calculated using equations (11) and (12), further exploring the relationship between the torsional shear ratio R_M of the soil column and the depth of the soil layer under different spinning speeds of the steel casing. The results are shown in Figure 6. It is evident that the critical depth range for the soil plug column is ≤ 11 m, which indicates that the height of the retained soil should not exceed 11 m. Beyond this depth, the torsional shear ratio (R_M) becomes

greater than 1, leading to the shearing off of the soil column, thus causing a blockage. Furthermore, by maintaining a spinning speed of the steel casing at 5/4/2/4 m/h or 5/3/2/3 m/h, the soil column exhibits notable torsional shear resistance and is less susceptible to being severed.

Based on the above theoretical research, this article suggests that the wall thickness of the steel casing should be $t \leq 0.012$ m, the surface load $P_0 \leq 15$ kPa, the spinning speed of the steel casing should be maintained at $k_i = 5/4/2/4$ m/h or $5/3/2/3$ m/h, and the soil column retention height should be controlled within 11 m.

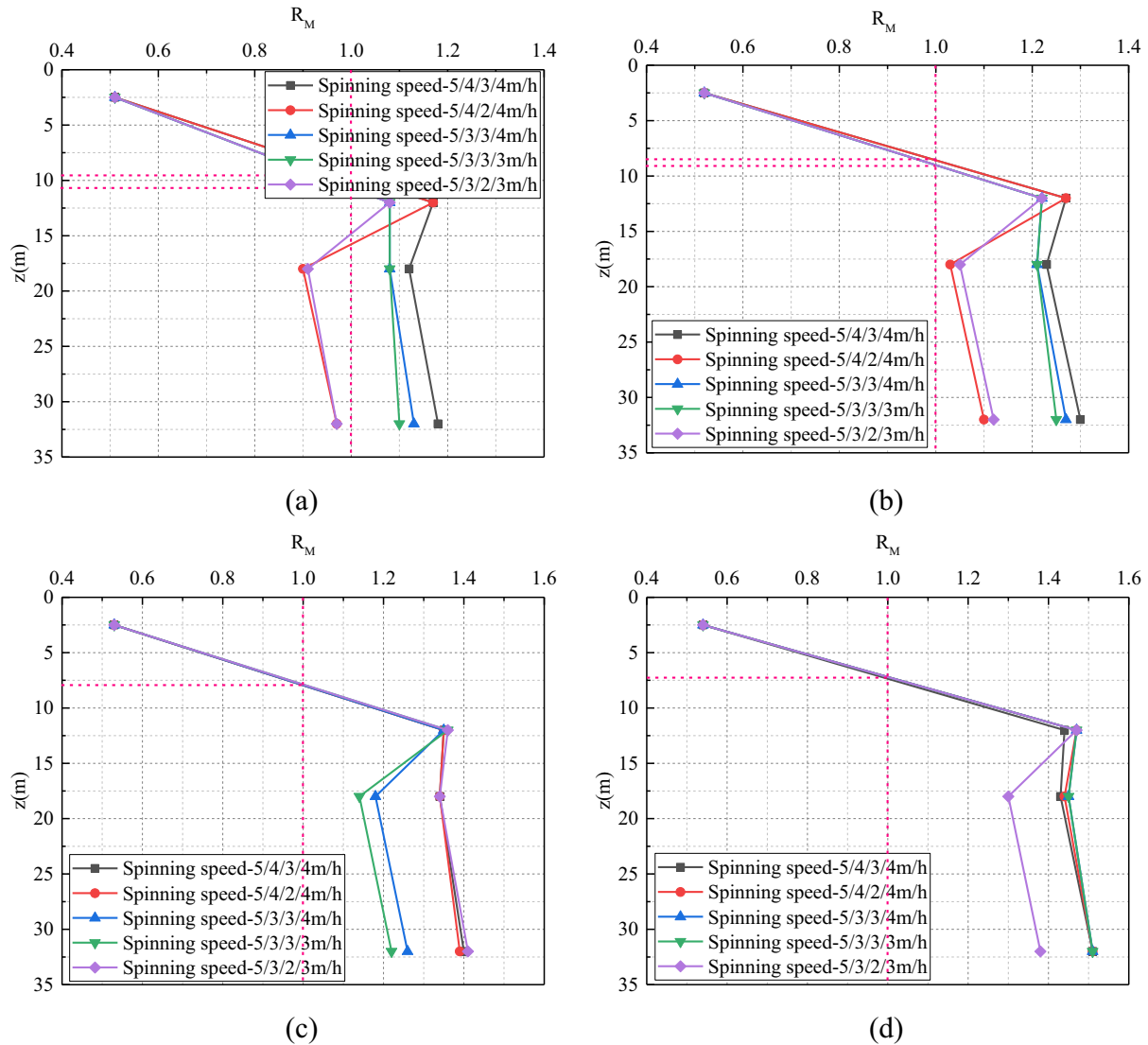


Figure 6: Analysis of stress ratio R_p of soil columns at different spinning speeds: (a) 0.012 m–10 kPa, (b) 0.012 m–15 kPa, (c) 0.012 m–20 kPa, and (d) 0.012 m–25 kPa.

4 Analysis of the impact of the construction of fully rotating steel casing cast-in-place piles on adjacent tunnels

4.1 Construction plan for test piles with a net distance of 2.5 m

In order to further explore the impact of small spacing cast-in-place pile construction on the shield tunnel, on-site pile testing construction was carried out at a net

horizontal distance of 2.5 m from the tunnel, as shown in Figure 7. The specific construction points are as follows:

- (1) All temporary facilities within 30 m from the subway tunnel were removed, and the surface construction load was controlled to around 10 kPa.
- (2) The wall thickness of the steel casing was 12 mm, with an outer diameter of 1 m. The steel casing was divided into five sections, totaling 35.5 m. The first four sections were all 8 m in length, and the fifth section was 3.5 m. When the sinking depth of the steel casing reached 32 m, the fifth section was removed, and lap welding was used for the welding of the steel casing.

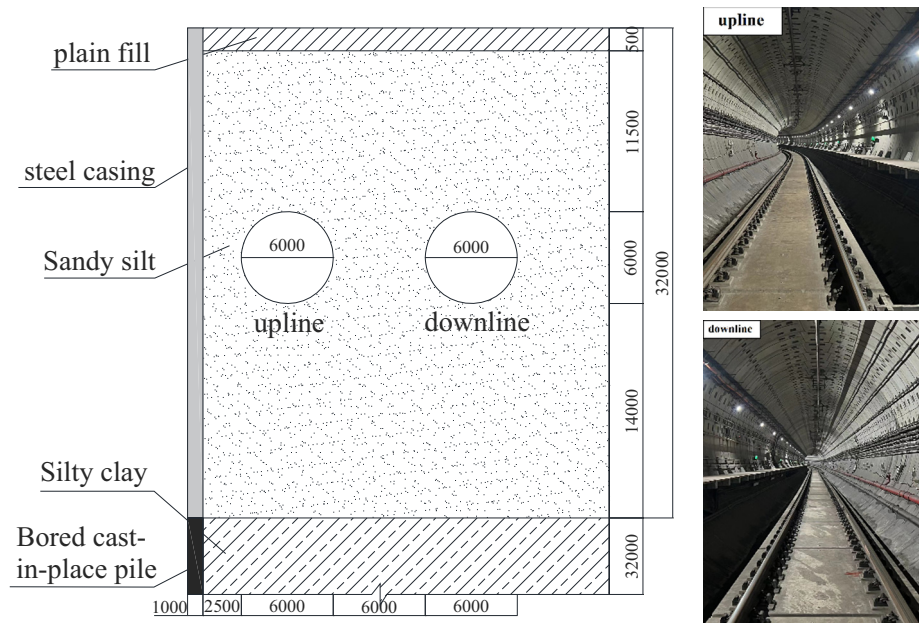


Figure 7: Schematic diagram of 2.5 m test pile location.

Table 3: Parameters related to steel casing spinning

Depth z_i (m)	Soil	Spinning speed k_i (m/h)	Height of soil left inside the cylinder h (m)
0–2.5	Plain fill, sandy silt	5	/
2.5–12	Sandy silt	4	7
12–18		2	7
18–32		3	10

- (3) As shown in Table 3, the spinning speed of the steel casing was segmented, and some soil was retained inside the casing.
- (4) After the steel casing was lowered, a rotary drilling rig was used to collect soil until a hole was formed (no less than 6.5 m into moderately weathered rock layers).

As shown in Figure 8, the on-site construction process mainly includes a fully rotating drilling rig in place, first section casing spinning, second section casing spinning and soil sampling, third section casing spinning and soil sampling, fourth section casing spinning and soil sampling, rotary drilling hole soil sampling, pouring pile concrete pouring, and steel casing extraction.

4.2 Analysis and monitoring of displacement and deformation of shield tunnel

Hangzhou Metro Line 1 crosses the middle block of the project. In order to ensure the safety of the shield tunnel

and its structural stability, very strict control requirements were put forward for the construction of the project. A fully automated monitoring system was used to monitor the subway protection area. The composition and monitoring principle of the automated monitoring system is shown in Figure 9a. The automated monitoring process mainly includes the following steps: measuring reference points and bias points; obtaining adjustment group data from the SQL database and sending it to the “GeoMos Monitoring Automatic Adjustment Assistant” software; “GeoMos Monitoring Automatic Adjustment Assistant” software is attached to traverse adjustment; returning the accurate coordinate data after adjustment to the SQL database; station orientation; start monitoring.

Specific monitoring implementation process: first, lay out the control network. The automated monitoring system adopts an independent coordinate system, and in order to ensure high measurement accuracy, 12 reference points are set up in the upline and downline tunnels. Second, carry out station layout. In order to ensure monitoring accuracy, the layout of the total station should fully

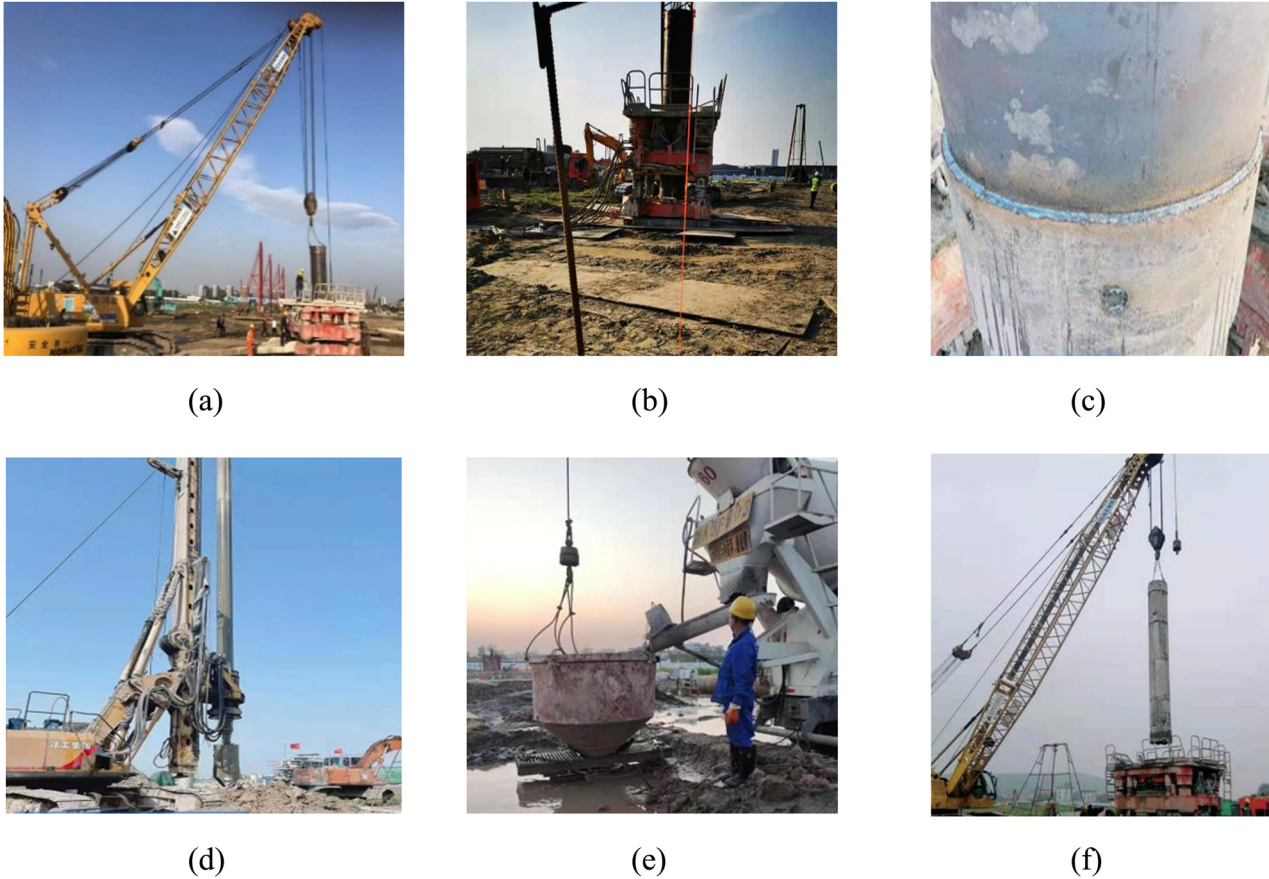


Figure 8: Test pile construction process: (a) drilling rig in place, (b) steel casing spinning, (c) welding of steel casing, (d) borehole soil sampling, (e) pouring of piles, and (f) steel casing extraction.

consider the position relationship between the measuring station and control points. The five instruments on the upline are, respectively, arranged at the positions of 699th ring, 677th ring, 507th ring, 407th ring, and 307th ring. Then, lay out the bias points. The L-shaped prism corresponding to bias point is placed on the total station bracket to ensure consistency between the offset point and the station changes. Finally, lay out the monitoring points, as shown in Figure 9b.

The monitoring objects of the tunnel include settlement of the tunnel track bed, horizontal displacement of the tunnel, and horizontal convergence of the tunnel. Based on the relevant regulations, it was ascertained that the pile foundation is oriented toward the 5-ring area. Consequently, the placement of monitoring points followed a pattern of two sections per five rings within this area. Furthermore, on either side of the pile foundation, an extension of 25 rings was made, with monitoring points arranged at a rate of 1 section per 5 rings in these extended regions.

Figure 10 shows the settlement monitoring results of the track bed before and after the construction of the fully rotating bored pile. The entire monitoring process starts from the first section of casing spinning until the completion of the concrete pouring and pile pouring. The settlement of the track bed “+” represents uplift, “-” represents subsidence, and the mileage K45 + 182 is the corresponding section of the 2.5 m test pile. The analysis of the monitoring data reveals that, at K45 + 182, the track bed experiences the highest cumulative settlement, with an upward displacement of 1.0 mm and a downward displacement of 0.9 mm. As the distance from the test pile increases, the settlement of the track bed decreases. Furthermore, the spinning process of the steel casing has a negligible impact on the roadbed settlement, indicating the favorable effectiveness of this construction technique in minimizing soil microdisturbance.

Figure 11 shows the horizontal displacement monitoring results of the shield tunnel during the entire construction process of the fully rotating bored pile (the horizontal displacement “+” indicates being away from the pile, and “-”

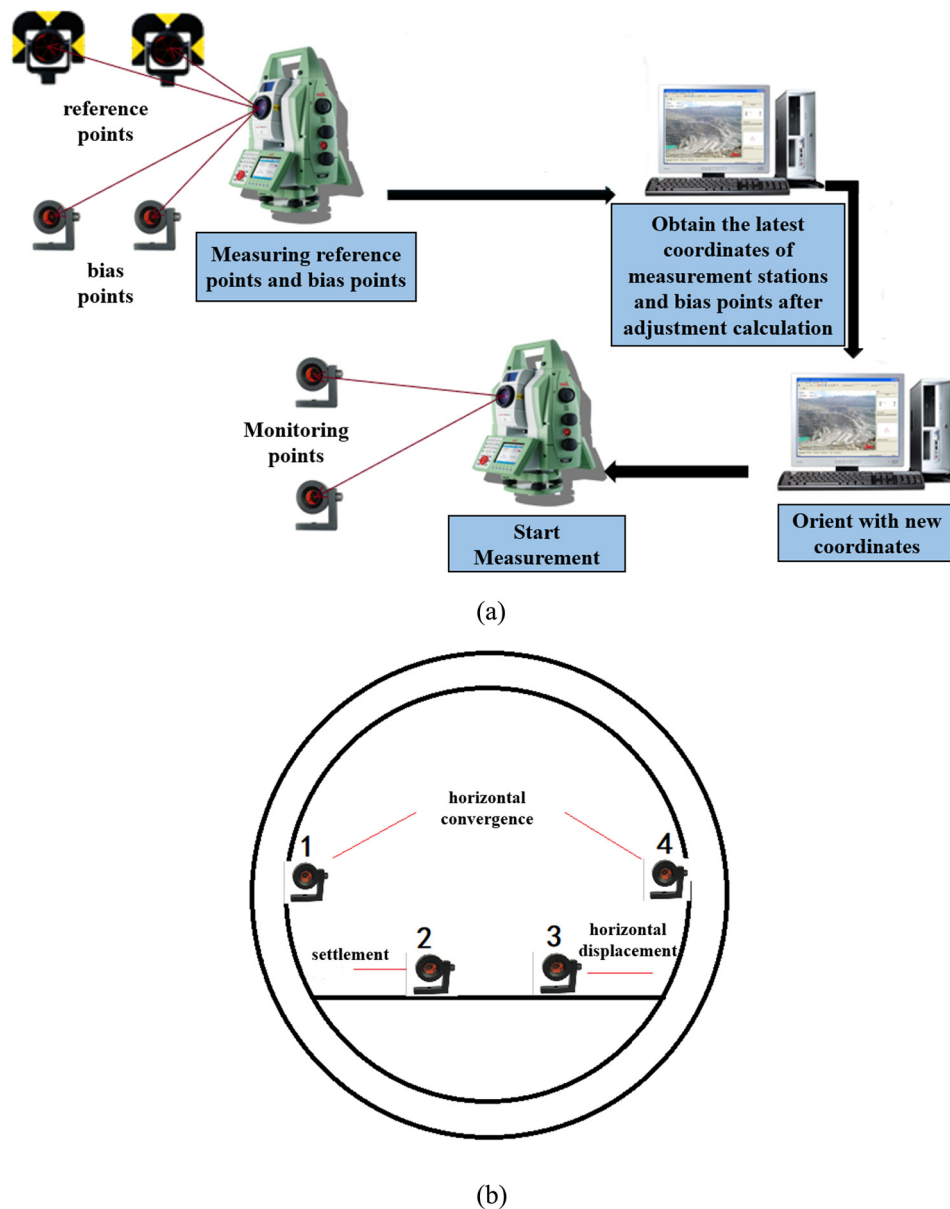


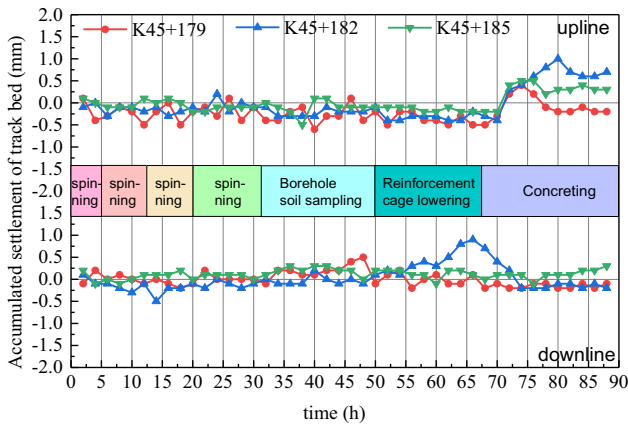
Figure 9: Automated monitoring system and layout of monitoring points: (a) composition of automated monitoring system and (b) layout of monitoring points.

indicates being close to the pile). From the monitoring data, it was found that during the construction of test piles, the parts of the tunnel's upline and downline near the test piles showed a phenomenon of being far away from the test pile, and the deformation was small. The cumulative maximum horizontal displacement of the upline was 0.6 mm, and the cumulative maximum horizontal displacement of the downline was 0.5 mm.

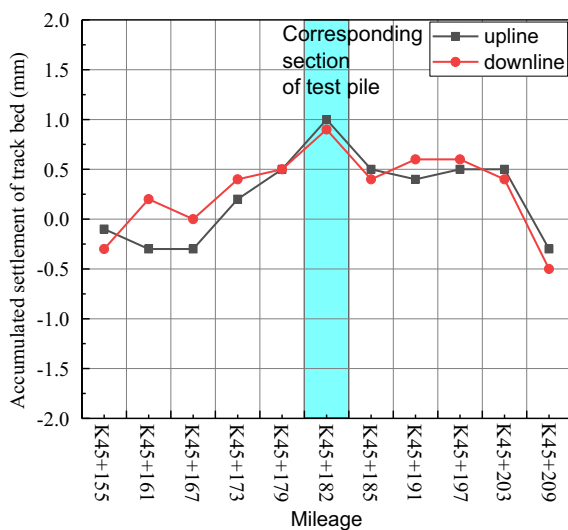
From Figure 12, it can be seen that the cumulative convergence deformation of the upline and downline is less than 1.0 mm, and the convergence deformation values

of all sections are positive (horizontal convergence “+” represents tunnel expansion, and “-” represents tunnel shrinkage). That is to say, during the construction of bored piles, the tunnel exhibits lateral expansion and “horizontal duck egg” deformation. This phenomenon primarily arises from the creation of stress relief holes along the tunnel's side during the drilling process. These holes cause localized loss of formation material, leading to an increase in the tunnel's transverse diameter.

Based on the monitoring data shown in Table 4 and compared with the control indicators, it was found that all



(a)



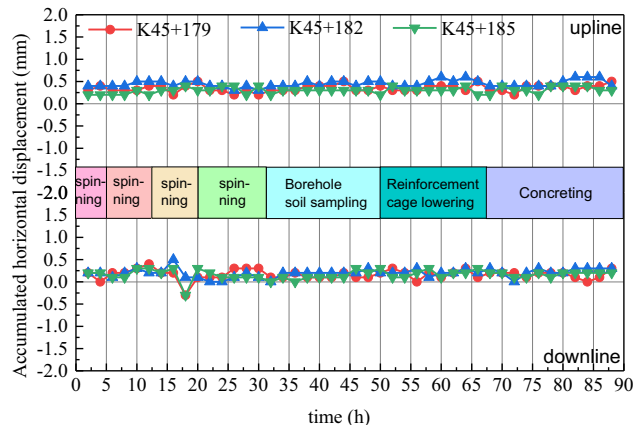
(b)

Figure 10: Accumulated settlement of the track bed: (a) the variation of cumulative settlement over time and (b) accumulated settlement changes with mileage.

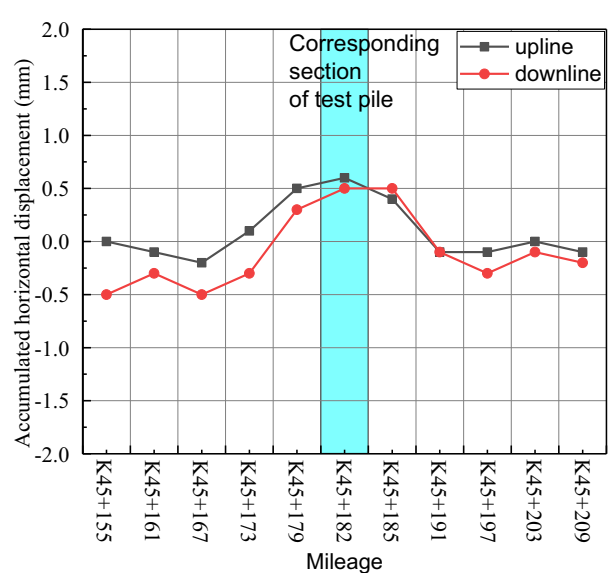
monitoring results were lower than the monitoring warning values, and the data changes were relatively stable. The observation of tunnel displacement and deformation revealed that the implementation of the full rotary steel casing bored pile construction approach effectively mitigates the impact on the shield tunnel and successfully minimizes microdisturbances.

5 Discussion

Based on the tunnel monitoring results, it can be seen that the settlement, horizontal displacement, and convergence deformation do not exceed 1 mm, which proves that the pile foundation construction process has a good microdisturbance effect. This is mainly because the construction



(a)



(b)

Figure 11: Accumulated horizontal displacement: (a) the variation of cumulative horizontal displacement over time and (b) accumulated horizontal displacement changes with mileage.

control parameters are obtained by controlling the soil column stress and torsional shear ratios of the soil plug column, which can indirectly control the soil plug. By controlling the soil blockage, the impact of close pile foundation construction on adjacent tunnels will obviously be greatly reduced.

During the construction process of steel casing cast-in-place piles, the steel casing has a lateral soil-squeezing effect. When the height of the soil column inside the casing is large, there is also a squeezing effect at the bottom of the steel casing. When the height of the soil column inside the casing is small, there is an unloading effect on the soil at the bottom of the steel casing [2–4]. How to achieve comprehensive control of lateral and bottom soil by controlling

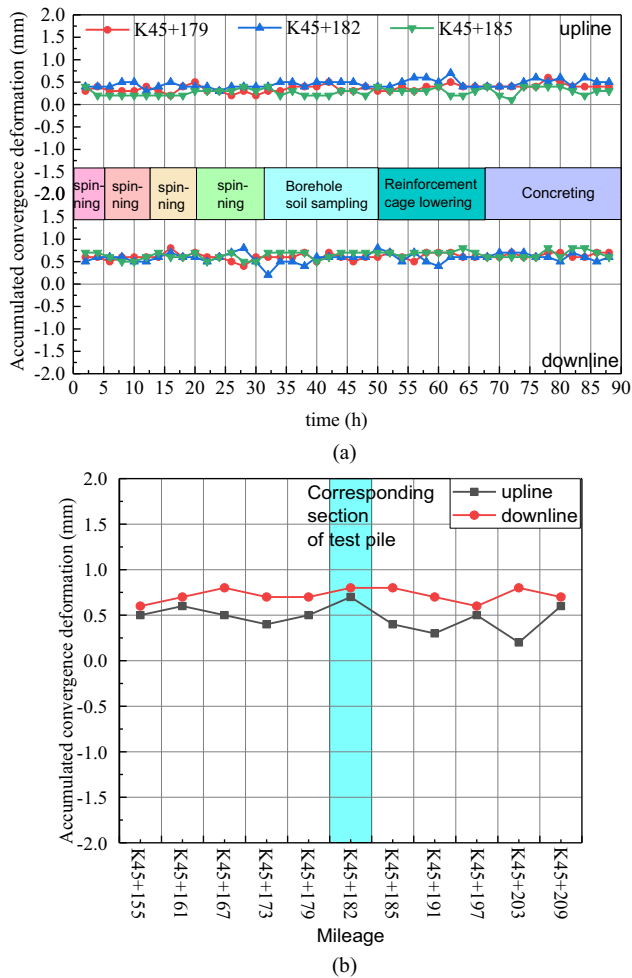


Figure 12: Convergence deformation: (a) the variation of convergence deformation over time and (b) convergence deformation changes with mileage.

construction parameters, thereby further reducing the impact of steel casing pile construction on adjacent tunnels, is a direction worthy of further research. The construction process of steel casing cast-in-place piles can cause disturbance to the surrounding soil, resulting in a

decrease in the soil strength [29]. How to reasonably consider the impact of soil strength reduction on adjacent tunnels is also a direction worthy of further research.

Obviously, in similar engineering projects in the future, the construction control of pile foundations near the tunnel can be achieved using the construction process of fully rotating steel casing bored piles, and the specific construction control parameters can be determined using the theoretical model of soil plug columns. This can achieve rapid and low-cost determination of construction parameters while ensuring minimal disturbance to adjacent tunnels.

6 Conclusion

By establishing a theoretical model of soil plug columns, the influence of fully rotating steel casing bored piles on soil under different construction factors was analyzed, and reasonable construction process parameters were determined. Through a test pile with a horizontal net distance of 2.5 m from the tunnel, the impact of this construction process on adjacent existing shield tunnels was explored. The main conclusions were as follows:

- (1) By establishing a theoretical model of soil plug columns, which combines practical working conditions and fully considers the effects of steel casing wall thickness, surface load, and spinning speed on the critical depth of soil columns, the stress situation of soil plug columns when steel casing passes through different depths of soil layers was explored.
- (2) Based on the analysis of soil column stress ratio R_p and soil column torsional shear ratio R_M , it was recommended that the wall thickness of the steel casing be $t \leq 0.012$ m, the surface load $P_0 \leq 15$ kPa, the rotational excavation speed of the steel casing be maintained at 5/4/2/4 m/h or 5/3/2/3 m/h (corresponding to soil depth of 2.5/9.5/6/14 m), and the soil column retention height be

Table 4: Statistical table of maximum cumulative variables for metro monitoring projects (Hangzhou region)

Monitoring items		Accumulated maximum change (mm)		Control indicators (cumulative)		
		Section	Variation	Warning value (mm)	Alarm value (mm)	Control value (mm)
Upline	Settlement	K45 + 182	1.0	1.2	1.6	2.0
	Horizontal displacement	K45 + 182	0.6	1.2	1.6	2.0
	Convergence deformation	K45 + 182	0.7	1.2	1.6	2.0
Downline	Settlement	K45 + 182	0.9	1.2	1.6	2.0
	Horizontal displacement	K45 + 182	0.5	1.2	1.6	2.0
	Convergence deformation	K45 + 182	0.8	1.2	1.6	2.0

controlled within 11 m to ensure good micro disturbance effect during the construction of the steel casing bored pile.

- (3) By developing a test pile construction plan with a horizontal net distance of 2.5 m from the tunnel, combined with the measured data of the shield tunnel, it was found that the construction of the small net distance fully rotating steel casing bored pile resulted in a cumulative roadbed settlement of 1.0 mm, a cumulative horizontal displacement of 0.6 mm, and a cumulative convergence deformation of less than 1.0 mm for adjacent existing shield tunnels. The measured data of the tunnel verifies the rationality of the theoretical model of the soil plug column and the referential nature of the construction process parameters.

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Author contributions: JRZ conducted relevant theoretical analysis work. DJG conducted relevant calculations and revised the manuscript. XXZ wrote the first draft. ZCB conducted pile testing construction. MJL conducted tunnel monitoring. The authors applied the SDC approach for the sequence of authors.

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