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RESEARCH ARTICLE

Investigation of the first quasi-rectangular metro tunnel constructed by the 0-θ method

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ABSTRACT Quasi-rectangular shield tunneling is a cutting-edge trenchless method for constructing metro tunnels with double tubes, owing to its advantages in saving underground space and reducing ground disturbance. However, the conventional quasi-rectangular shield tunneling method is not applicable when constructing a tunnel without a center pillar, such as a scissor crossover section of a metro line. Therefore, the 0-θ tunneling method, which combines the quasi-rectangular shield and pipe jacking methods, was investigated in this study to solve the aforementioned construction challenges. This study presents a case study of the Sijiqing Station of the Hangzhou Metro Line 9 in China, in which the 0-θ method was first proposed and applied. Key techniques such as switching between two types of tunneling modes and the tunneling process control in complex construction environments were investigated. The results demonstrated that the 0-θ method can address the technical challenges presented by the post-transition line with a high curvature and a scissors crossover line. In addition, the adoption of the 0-θ method ensured that the transformation between shield tunneling and pipe jacking was safe and efficient. The ground settlement monitoring results demonstrated that the disturbance to the surrounding environment can be limited to a safe level. This case study contributes to the construction technology for a metro tunnel containing both post-transition lines with a small turning radius and a scissors crossover line. A practical construction experience and theoretical guidance were provided in this study, which are of significance for both the industry and academia.

KEYWORDS quasi-rectangular tunnel, 0-θ method, pipe jacking, shield tunneling, underground space

1 Introduction

With the rapid development of metropolitan areas such as New York, Tokyo, Hong Kong (China), and Hangzhou, the space above the ground has become increasingly crowded owing to the increase in buildings, population, and traffic. Thus, the development of underground spaces presents significant potential for alleviating the shortage of surface space and easing urban traffic pressure [1–3]. Shield tunneling, especially shallow tunneling, is the main method for constructing metro, road, and utility tunnels owing to its high working efficiency and effectiveness in ground disturbance control [4–6]. However, conventional circular tunnels, which are built by shield tunneling or pipe jacking methods, are not as efficient as quasi-rectangular tunnels in terms of space utilization efficiency. In addition, quasi-rectangular tunneling is proven to induce less ground disturbance compared to the conventional circular tunneling method [7]. Therefore, the quasi-rectangular tunneling method has been extensively applied in recent times.

Figure 1 depicts the cross-section of a metro line with two conventional single circular tunnels and a quasi-rectangular tunnel. As shown in Fig. 2(a), a center pillar is constructed in the quasi-rectangular tunnel to reduce the bending moments within the tunnel vault. Previous

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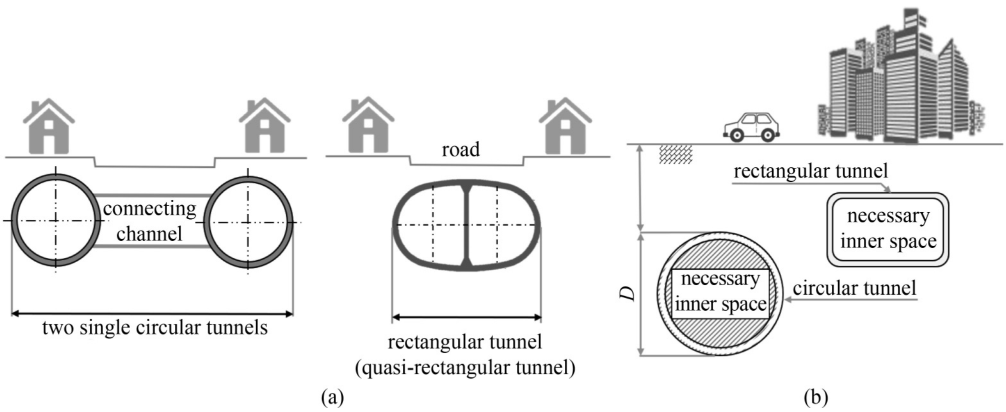

Fig. 1 Comparison of the quasi-rectangular and circular tunnels: (a) comparison of the excavation boundary range; (b) comparison of the space utilization.

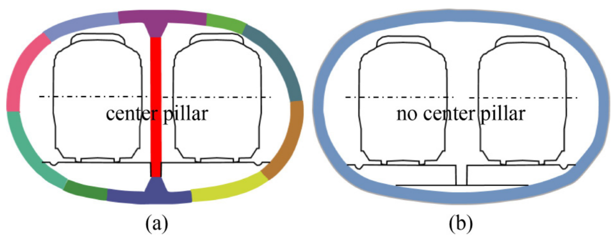

Fig. 2 Quasi-rectangular lining structure: (a) quasi-rectangular lining with a center pillar; (b) quasi-rectangular lining without a center pillar.

studies have shown that a quasi-rectangular tunnel can save more than 20% of space compared to a conventional circular tunnel [8]. Therefore, the quasi-rectangular tunneling method has gained increasing attention from industry and academia. Figure 3 illustrates the construction process of quasi-rectangular shield tunneling; it should be noted that the propulsion hydraulic cylinders act on the assembled tunnel segment linings, which provide a thrust force to push the shield machine forward. The first quasi-rectangular tunnel in the world was successfully constructed in Ningbo, China [9]; in this project, several scholars investigated the ground disturbance of quasi-rectangular shield tunneling. Zhang et al. [10] investigated the ground surface deformation caused by the combined effects of multiple factors, such as grouting pressure, the friction between the shield skin and soil, ground volume loss, and found that grouting pressure and ground volume loss played a decisive role in controlling the ground settlement. The additional soil stress caused by quasi-rectangular shield construction was studied by Chen et al. [11]; the results demonstrated that the additional stress field of the quasi-rectangular shield is similar to that of the large-diameter circular shield, whereas the additional stress of the soil in the double-circular shield section is close to twice that of the quasi-rectangular shield. The characteristics of ground deformation caused by quasi-rectangular shield tunneling were investigated based on field monitoring of surface deformation and soil-layered settlement by Si et al. [12]. It was found that the influence of the shield attitude on the deformation of the surrounding stratum was more significant than that of a single circular shield. In addition, the surface deformation law of the ordinary double-circle shield and quasi-rectangular shield construction in the soft soil area in Ningbo was studied by Qiu [13], who concluded that the ground disturbance zone of quasi-rectangular shield tunneling was smaller than that of double-circle shield tunneling and the ground loss for

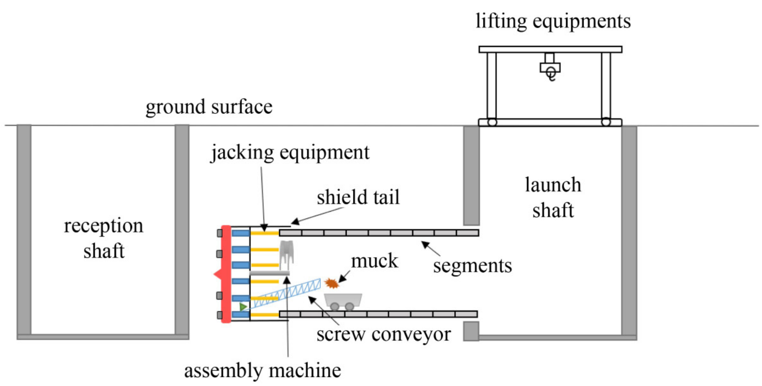

Fig. 3 Shield propulsion diagram.

quasi-rectangular shield tunneling was reduced by approximately 30%. Previous studies have demonstrated that quasi-rectangular shield tunneling has several advantages over traditional circular shield tunneling. In a conventional quasi-rectangular shield tunnel, the center pillar is installed to reduce the bending moment developed in the tunnel vault and maintain the stability of the tunnel lining. However, in the scissors crossover section of a metro line, the center pillar must be eliminated because the train vehicles must cross the central wall for direction turning (Fig. 4). Under such circumstances, the quasi-rectangular pipe jacking method, which has a similar cross-section but does not use the center pillar structure (Fig. 2(b)), is developed to construct the scissors crossover section.

However, when a turn-back line contains the scissors crossover section in addition to the long-distance curved post-transition section (Fig. 5), the pipe jacking method alone cannot be employed to complete the construction of the entire turn-back line [14–16]. For such a project background, the turn-back line tunnel must be constructed by first adopting the pipe jacking method (for the scissors crossover section with prefabricated pillarless quasi-rectangular pipe segments) and then transferring it to the quasi-rectangular shield method for constructing the subsequent long-distance flat curve section with a center pillar. However, there are no practical projects or studies on the use of a hybrid method combining the quasi-rectangular shield method and pipe jacking method for an actual tunnel project [17–24].

Therefore, this study presents the first application of the hybrid tunneling method for constructing a turn-back section of a quasi-rectangular metro tunnel. Because the cross-section shapes of the quasi-rectangular pipe jacking tunnel and shield tunnel resemble the characters “0” and “θ”, respectively, this hybrid tunneling method is named the 0-θ method. The case study was conducted on the first quasi-rectangular tunneling project in the world that utilized the 0-θ method to construct the turn-back line of the Hangzhou Metro Line 9 tunnel project in China. Details regarding the project and construction procedures using the 0-θ method are first comprehensively investigated, and the key technologies and challenges in the construction phase, as well as countermeasures, are thoroughly studied and discussed. The results of this study provide a meaningful technical reference for the tunneling industry and urban underground space development.

2 Construction background

2.1 Project background

As the terminal station of the line, the length of the turn-back line of Sijiqing Station is 231.1 m, including the scissors crossover section of 67.2 m, followed by the post-transition section with a length of 163.9 m. The scissors crossover section starts at Sijiqing Station and crosses the Qiushi Bridge to the switching shaft. The post-transition section starts from the switching shaft, crossing the ancient seawall and the Xinkai River to the end of the line (Fig. 6). Owing to the heavy ground traffic and intertwined nature of the existing metro and municipal pipelines, the open-cut method (or trenching method) cannot be implemented. Therefore, the quasi-rectangular tunnel technology is used for the construction of the turn-back line for the first time.

2.2 Construction method

A quasi-rectangular earth pressure balance (EPB) dual

Fig. 4 Scissors crossover section of a metro line.

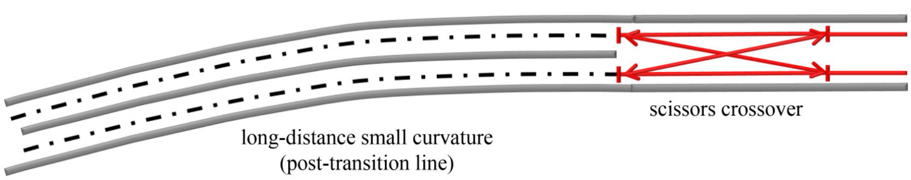

Fig. 5 Diagram of the turn-back line interval with scissors crossover line and flat curve.

Fig. 6 Project location layout.

mode (pipe jacking/shield) tunneling machine of length 11.83 m and height 7.27 m was used in this project. Quasi-rectangular pipe jacking in the scissors crossover section (Sijiqing Station to the switching shaft) of the tunnel was used, as it meets the pillarless structural requirements of the scissors crossover (Fig. 7(a)). To build a pillarless structure, the lining of the pipe jacking was designed to be thicker and wider than that of the shield for improved strength. In addition, because the tail gaps of the pipe jacking and shield are approximately 20 and 300 mm, respectively, a thicker pipe section can occupy the tail gap and shell thickness of the shield to construct the pillarless structure (Figs. 7(a)–7(c)). The parameters concerning section size of the lining and tunneling machine are listed in Table 1. However, owing to the flat curve of part of the turn-back line (switching shaft to reception shaft), it is difficult to implement ordinary pipe jacking. Therefore, pipe jacking is switched to the conventional quasi-rectangular shield mode that contains a center pillar in the switching shaft (Figs. 7(b) and 7(c)); thus, the post-transition section excavation can be completed (Fig. 7(d)). This combined construction method is called the 0-θ method (Zero-Theta method) in this study.

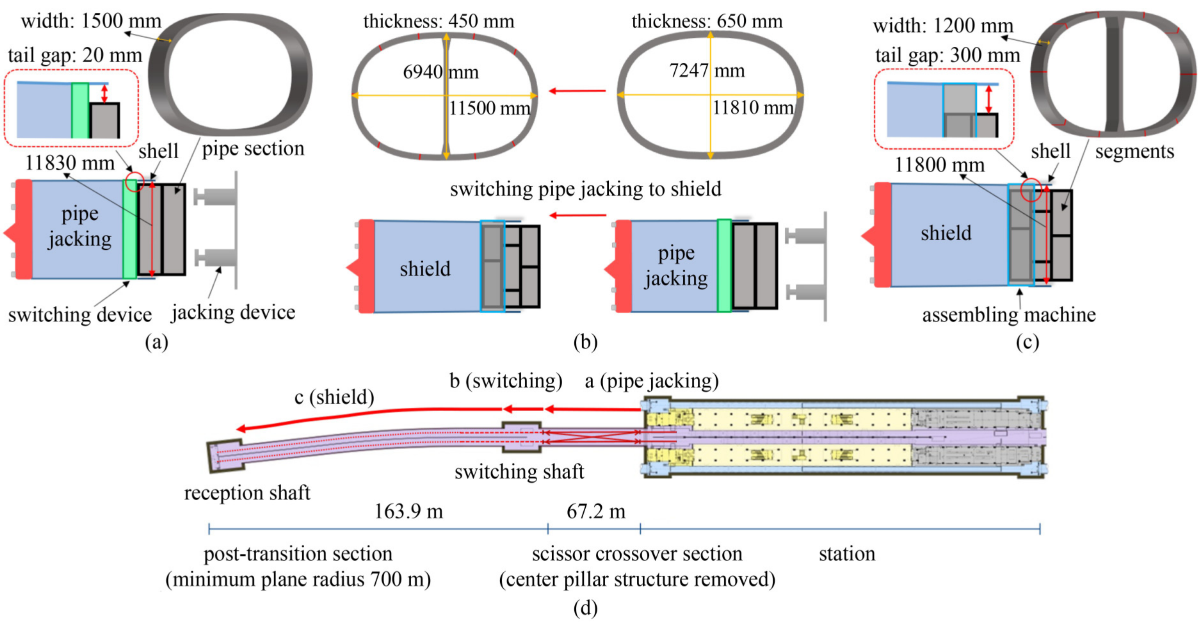

Fig. 7 Sijiqing excavation plan and turn-back line interval: (a) quasi-rectangular pipe jacking tunneling; (b) switching pipe jacking to shield; (c) quasi-rectangular shield tunneling; (d) turn-back line interval.

Table 1 Section size of the tunneling machine and lining in different work modes

work mode	lining thickness (mm)	lining width (mm)	lining outer size (mm)	lining structure	outer size of tail shell (mm)	tail gap (mm)
pipe jacking	650	1500	11810 × 7247	pillarless	11830 × 7270	20
shield	450	1200	11500 × 6940	pillar	11800 × 7240	300

2.3 Geological conditions

The maximum longitudinal slope of the scissors crossover section is −0.2% ('−' represents downward and '+' represents upward), and the overburden depth ranges from 10.23 to 10.47 m. The main soil strata through which the tunnel is aligned are L2-sandy silt, L4-sand with sandy silt, and L5-sandy silt (Fig. 8). The minimum plane radius of the post-transition section is 700 m, and the maximum longitudinal slope is −0.2%, implying the overburden depth changes from 10.94 to 11.37 m. The main strata encountered by the tunnel cross-section are L3-sandy silt, L4-sand with sandy silt, and L5-sandy silt, as shown in Fig. 8. Detailed physical parameters of the strata are listed in Table 2. The depth of the groundwater level in the project varies from 1.3 to 4.4 m below the surface.

3 Design of the dual-mode tunneling machine

3.1 Dual-mode tunneling machine

In this project, an EPB tunneling machine (Fig. 9) was adopted based on the geological conditions. This EPB machine has a unique quasi-rectangular cross-section with an outer dimensions of width 11.83 m and height 7.27 m. The cutterhead system consists of two large X-shaped cutterheads and two small circular cutterheads, which can collaboratively excavate a quasi-rectangular cross-section. This EPB shield tunneling method was employed to construct a quasi-rectangular metro tunnel with double train tracks aligned in a single tunnel tube. The assembly system of this EPB machine is specifically designed to meet the requirements of assembling the pillar structure segments, which achieves a high-precision control of movement with eight degrees of freedom. Notably, this type of tunnel is demonstrated to be more space-saving compared to metro tunnels constructed with two separated smaller circular tubes. The normal EPB tunneling machine can be directly used to construct shield tunnels in the post-transition section (namely, working in shield mode), but it must be retrofitted to function in pipe jacking mode to construct the tunnel in the crossover section. The mode-switching technology is critical to this hybrid method of shield and pipe jacking and is discussed in Subsection 3.2.

3.2 Tunneling mode switching technology

3.2.1 Comparison of the shield and pipe jacking working modes

Before introducing the working mode switching

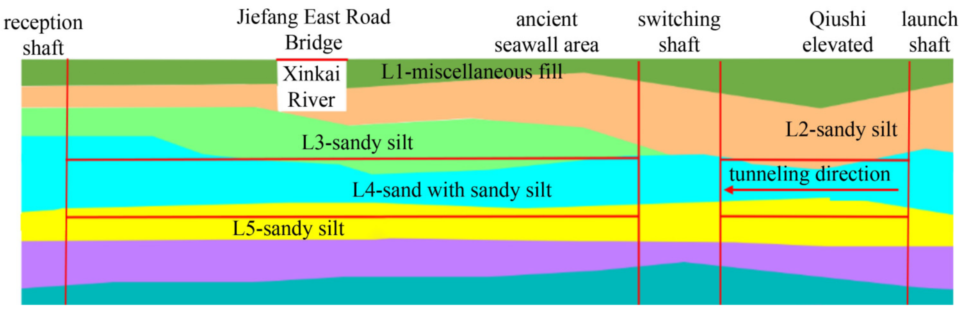

Fig. 8 Quasi-rectangular turn-back line-interval strata sections.

Table 2 Physical parameters of each stratum

stratum	thickness (m)	lateral earth pressure coefficient	unit weight (kN/m^3)	cohesion (kPa)	internal friction angle (°)
L1-miscellaneous fill	2.3	0.50	17.50	8.00	15.00
L2-sandy silt	8.3	0.52	19.20	4.00	26.00
L3-sandy silt	5.9	0.45	19.20	6.00	30.00
L4-sand with sandy silt	6.3	0.37	19.70	5.00	34.00
L5-sandy silt	3.5	0.52	19.40	7.00	24.00

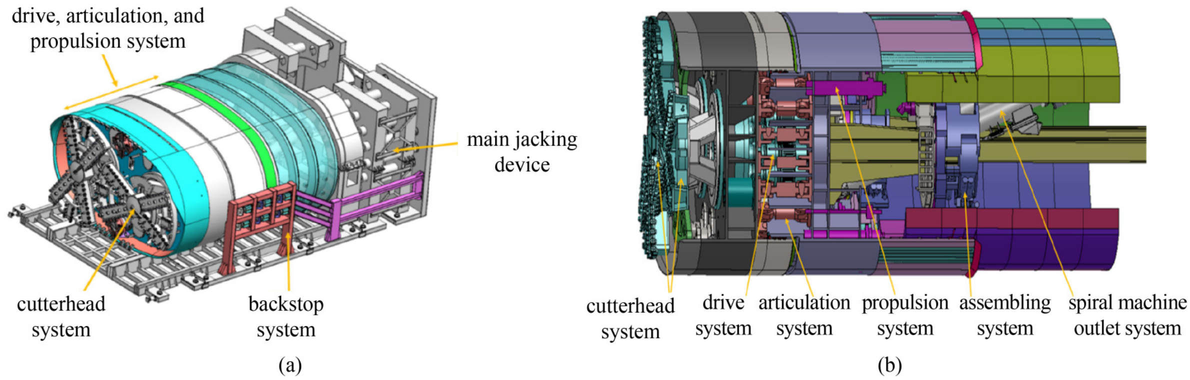

Fig. 9 Pipe jacking and shield mainframe structure: (a) pipe jacking mode mainframe structure; (b) shield mode mainframe structure.

technology, it is necessary to compare the two working modes. When the EPB tunneling machine operates in the shield mode, the shield tail is connected to the back shell, and the propulsion cylinder functions as the propulsion system. The articulated cylinder is used to meet the needs of the shield machine turning; a schematic diagram of the main machine structure is shown in Fig. 10.

When the EPB tunneling machine operates in the pipe jacking mode, an additional jacking system is prepared for the different propulsion and force transmission methods. Simultaneously, a dual-mode switching device connects with the back shell, and the propulsion cylinder becomes the force transmission support system. Considering that the articulating cylinders are used in both modes with different functions (they are used to satisfy the requirements of the machine for alignment deflection in the pipe jacking mode), they are only used for maintenance work. The spiral machine has a new tie rod, its outlet is rotated toward the middle, and a new guide slot is created. Considering that the assembling machine is not used in the pipe jacking mode and the advantage of shortening the length of the machine for ease of lifting, the large translation beam and assembling machine is removed. A schematic of the main structure of the machine is shown in Fig. 11.

3.2.2 Dual-mode switching design for shield and pipe jacking

Considering the differences in the force transfer mechanism and shell structure configuration between the shield and pipe jacking modes (Table 3), the shield tail in the jacking mode was removed and a new switching device was created (connecting frame). The connecting frame was welded to the backside of the back shell, and the dimensions of the outer shell were the same as those of the back shell.

When the tunneling machine functions in the shield mode, the jacking force of the propulsion cylinder is transferred to the front shell through the back shell. To ensure that the transmission method remains stable, a ring plate is set at the back of the top block of the propulsion cylinder; therefore, the back part of the ring plate is a pipe section. Under such conditions, the jacking force of the jacking cylinder can be transmitted to the ring plate through the pipe section, moved from the ring plate to the propulsion cylinder, and finally reach the back shell through the propulsion cylinder. Therefore, a constant force transmission in both working modes can be maintained. This transmission system makes the propulsion cylinder work as a propulsion system in the

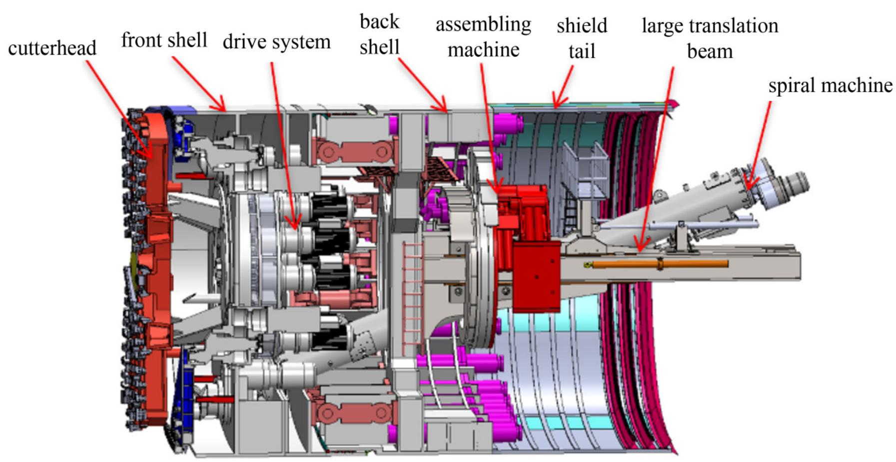

Fig. 10 Schematic diagram of the main machine structure in the shield mode.

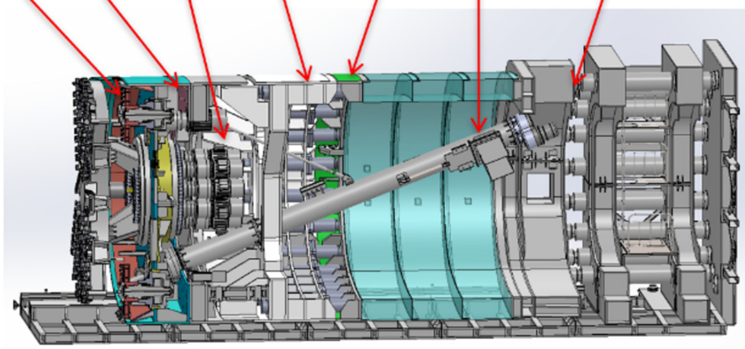

Fig. 11 Schematic diagram of the main structure of the machine in pipe jacking mode.

Table 3 Comparison of the shield and pipe jacking working modes

difference item	shield mode	pipe jacking mode
propulsion method and force transmission method	The propulsion cylinder is on the front of the segments and pushes forward. The jacking force is transmitted from the propulsion cylinder to the machine.	Using the main jacking device against the reaction wall of the launch shaft, the front pipe section is jacked forward. The first pipe section is pressed against the ring plate of the machine's back shell, and the jacking force is transferred from the pipe section to the back shell.
shell structure	front shell, back shell, and tail shell	front shell, back shell, and connection frame
articulating device	meets the needs of shield machine turning	meets the needs of the pipe-jacking machine to correct the alignment deflection
backup frame	The tunnel lays the track and follows behind the main machine to move forward.	placed on the first few pipe sections, which are at rest with respect to the pipe sections
other structures	the assembly system, including the assembling machine, large translation beam, single and double beam for segment assembly	No assembly system. Pipe sections are assembled in front of the main jacking device in the launch shaft.

shield mode and as a force transfer support system in the jacking mode.

To ensure the strength of the connection between the outer shell and connecting frame, stiffener plates were employed and inserted in the gap between the propulsion cylinders. The detailed structure of the switching device is illustrated in Fig. 12.

Moreover, the connecting frame, integrated with the EPB tunneling machine, should be easily detached from the concrete segment (pipe section) when pipe jacking is complete. For this purpose, four detaching holes are set at designated positions on the connection frame, as shown in Fig. 13. Four hydraulic cylinders of the jacking system of the EPB tunneling machine can extend through the detaching hole and directly reach the concrete segments. Therefore, the reaction force derived from the concrete segments pushes the EPB tunneling machine forward, and the connection frame can finally properly detach from the concrete segment.

4 Comprehensive technical measures for the construction of the quasi-rectangular dual-mode tunneling machine

4.1 Critical working parameter of tunneling

4.1.1 Earth pressure determination and control

Owing to the several silty and sandy soil layers in the

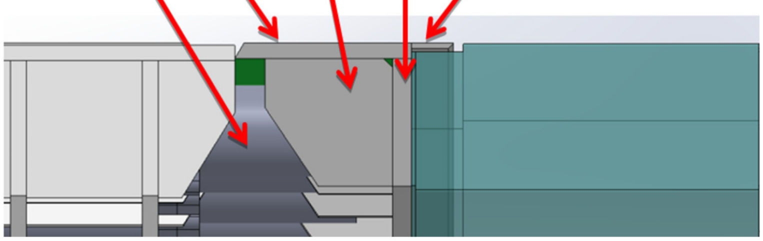

Fig. 12 Structural diagram of the switching device.

stratum, the earth pressure applied to the excavation face should be properly determined. The torque on the machine is always large when tunneling in sandy soil. Without considering the relevant construction details and parameters, it is highly likely for the cutter to get stuck and be unable to turn. Tables 4 and 5 list the construction parameters of the shield and pipe jacking indicators in the current project, respectively.

The EPB machine uses the earth pressure to maintain the stability of the excavation face of the soil layer and control the surface settlement; hence, setting the earth pressure is the key to tunneling construction. According to Eqs. (1)–(3), the water−earth pressure (P) between the top and bottom of the cutterhead is loosely estimated to range between 200 and 260 kPa. For safety, the passive earth pressure (P_P) at the excavation face is also considered as the maximum earth pressure setting value, which is determined to be approximately 550 kPa using Eq. (4). Notably, when the shield crosses the geologically fragile Xinkai River section (Fig. 8), the earth pressure and penetration speed should be reduced to prevent damage to the river area caused by variation in the overburden depth.

$$P_s = \sum K_0 \gamma h_i, \tag{1}$$

$$P_w = \gamma_w h_w, \tag{2}$$

$$P = P_w + P_s, \tag{3}$$

$$P_P = \sum \gamma h_i \tan^2(45^\circ + \frac{\varphi}{2}) + 2c\tan(45^\circ + \frac{\varphi}{2}), \tag{4}$$

where P_s denotes the lateral static earth pressure between the top and bottom of the cutterhead, P_w denotes the groundwater pressure, γ_w denotes the unit weight of water, h_w denotes the height of the groundwater level at the calculation point, K_0 denotes the lateral earth pressure coefficient of the corresponding stratum, γ denotes the unit weight of the soil of the corresponding stratum, h_i denotes the thickness of the corresponding stratum, φ denotes the internal friction angle of the soil, and c represents the cohesion of the soil (Table 2).

Table 4 Shield construction parameter indicators

area	easrth pressure (kPa)	penetration speed (mm/min)	volume of excavated earth (m^3/ring)
tunneling in normal area	200–260	10–40	84.7–86.4
tunneling in river area	100–130	10–30	

Table 5 Pipe jacking construction parameter indicators

earth pressure (kPa)	thrust force (kN)	penetration speed (mm/min)	volume of excavated earth (m^3/ring)
200–260	66200–72100	10–40	84.7–86.4

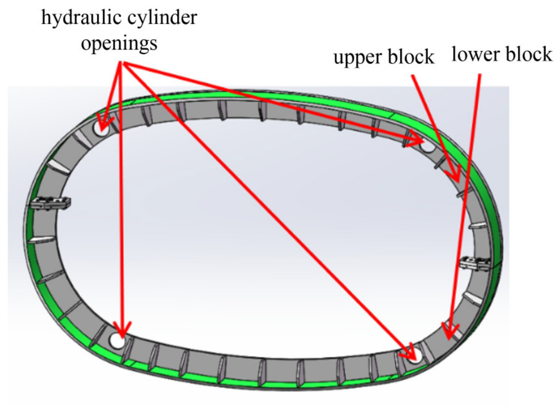

Fig. 13 Overall structure of the switching device.

4.1.2 Thrust force determination and control in pipe jacking mode

The pipe jacking machine and pipe sections move forward owing to the thrust force generated on the jack at the launch shaft, as shown in Fig. 14. If the thrust force is too small, the pipe section does not move forward; if it is too large, it may cause damage to the pipe section and the tunneling machine. Therefore, the maximum jacking force (F) must be calculated to control the construction parameters. The maximum thrust force of the pipe jacking machine includes two parts: the resistance force N_r from the excavation face and the frictional resistance force between the soil-pipe interface N_f. According to Eq. (5), the maximum jacking force of the pipe jacking can be estimated to be in the range of 66200–72100 kN.

$$F = N_r + N_f = S \times P_t + f \times L \times l, \tag{5}$$

where S is the cross-sectional area of the cutterhead of size 71.98 m^2; P_t is the passive earth pressure at 1/3 height above the bottom of the cutterhead, measured in kN/m^2; f is the friction coefficient of the grouting injection process, which can be determined by an actual test (typically $f = 4-7\ kN/m^2$; L is the circumference of the cutterhead or pipe section (31 m); l is the maximum jacking distance (67.2 m).

4.2 Grouting design

4.2.1 Synchronous grouting design for earth pressure balance tunneling

In the shield stage, the excavated soil in the cross-sectional area is larger than the required tunnel dimension; therefore, the gap between the ground and the outer part of the tunnel lining must be simultaneously filled during the tunneling process. Synchronous grouting was conducted along with soil excavation; thus, grouting mortar was injected into the gap through the tubes in the tail shell. The synchronous grouting design is important in shield tunneling.

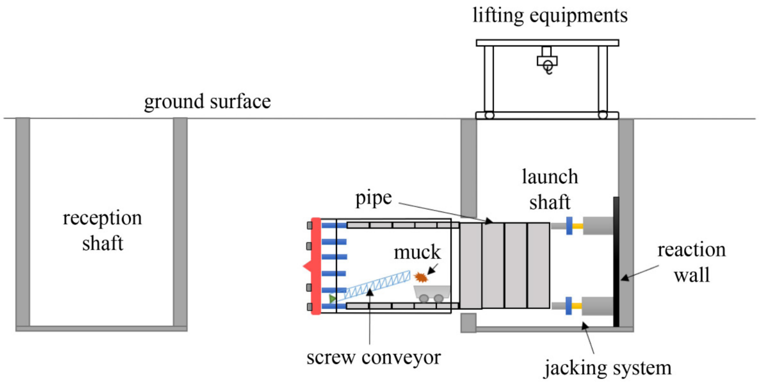

Fig. 14 Pipe jacking propulsion diagram.

The grouting mortar must meet certain technical requirements, including high fluidity and adequate early strength. Based on a slow-hardening medium with high fluidity and early-strength grouting, as well as the characteristics of synchronous grouting for quasi-rectangular shields, an early-strength grouting that adapted to the construction of quasi-rectangular shields was developed for this project. The early-strength synchronous grouting ratio and performance index are shown in Tables 6 and 7, respectively.

The filling rate for the gap was initially set to 180%, which was calculated to be a grouting volume of 11 m^3 per ring. The grouting pressure of the shield was then selected to range between 200 (earth pressure above the grouting hole calculated using Eq. (6)) to 250 kPa. The specific parameters can be further determined by the test data for the construction parameters and are adjusted in a timely manner based on the actual working conditions in shield tunneling.

$$P_{\mathrm{E}} = \sum \gamma h_i. \tag{6}$$

4.2.2 Friction-reducing grouting design for pipe jacking tunneling

In the pipe jacking stage, the frictional resistance at the soil-pipe interface must be properly reduced for a smooth jacking process; therefore, friction-reducing grouting should be sprayed circumferentially along the external pipe. In this grouting process, the lubricating grouting is continuously injected from inside the pipe; thus, the grouting forms an external sleeve that embraces the pipe section to achieve friction-reduction.

Table 6 High-flow and early-strength shield synchronization grouting ratio (kg/m^3)

sand	coal fly ash	lime	bentonite	cement	external admixture	water
1050	350	80	100	0	3	330

In this project, there is a 2 cm gap between the excavated ground cross-section and the external tunnel boundary, which allows for more space for the grouting sleeve to form. While reducing the tunneling friction resistance force, it relies on its own bearing capacity to support the soil and reduce the surface settlement to a certain level. The friction-reducing grouting is composed of sodium bentonite and an additive mixed with water, thereby forming a grouting with a high density, shear resistance, and consistency. The mixing proportions of the reducing grouting and its corresponding performance indices are illustrated in Tables 8 and 9, respectively.

During pipe jacking tunneling, the grouting volume of the outer wall of the pipe section is usually 150%–250% of the gap, which indicates that the grouting volume of a single pipe section of the pipe jacking is approximately 1.4–2.3 m^3. The grouting pressure was determined to range between 200 and 300 kPa.

4.3 Soil muck modification technique

During EPB tunneling, the soil muck excavated by the cutterhead settles in the soil tank and is subsequently discharged by the screw conveyor. As natural soil muck may result in high resistance on the cutterhead and blockage of the spiral conveyor, it is necessary to modify the soil muck to achieve smooth tunneling. In this project, the sandy silt soil layer, where the tunnel is aligned, is more likely to negatively impact the tunneling process, resulting in a large cutterhead torque and a high

Table 7 Performance index of a high-flow and early-strength shield synchronization grouting

slump (cm)		density (g·cm^{-3})	bleeding rate (%)	shear strength (Pa)		serviceable time (initial setting time) (h)	fluidity (mm)	compressive strength of 7 d (MPa)
0 h	8 h			0 h	4 h			
14	> 5	> 2.0	< 1%	> 300	> 800	0–20	> 200	> 0.15

Table 8 Pipe jacking friction-reducing grouting ratio (kg/m^3)

raw material	content
sodium carbonate	5
carboxymethyl cellulose (CMC)	1.2
bentonite	100
water	550

Table 9 Pipe jacking friction-reducing grouting performance index

property	value
funnel viscosity (s)	80
fluid loss (mL)	12.6
effective viscosity (mPa·s)	21
specific gravity (kg/m^3)	1.05

possibility of blockage. Therefore, the soil muck should be properly modified during EPB tunneling.

To modify the soil muck, additives such as bentonite and a foam agent are injected into the soil tank. The injection of the mixed additive is used to reduce the internal friction and improve the fluidity of the soil muck, thereby alleviating the cutterhead torque, reducing the load to the EPB tunneling machine and the wear of the cutter, and preventing mud cake generation. Moreover, soil muck modification can further reduce the underground water loss and facilitate settlement control in strata with high permeability.

Critical information regarding the soil muck modification in the current project is as follows. Bentonite is injected at the front side of the cutter and the breast plate of the soil bin; hence, multi-point, sub-regional, and independent control can be achieved (Fig. 15). When the oil pressure in the spiral machine is too high, it can be injected with an appropriate amount of bentonite grouting. The volume ratio of bentonite to water in the grouting is set to 1:3. The permeability coefficient of sandy soil is high, which indicates that the pore water pressure simultaneously develops and dissipates faster. The time difference between the development and

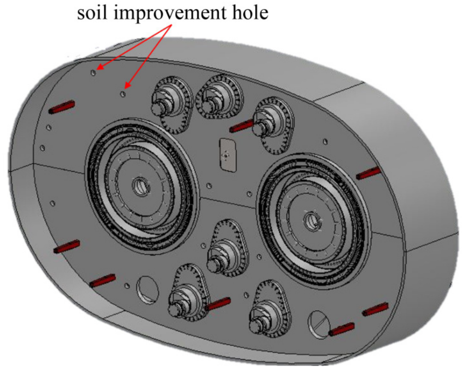

Fig. 15 Soil improvement hole.

dissipation of the pore water pressure is found to be the time-lapse of dewatering. Therefore, the speed of jacking should be compatible with the rate of dewatering.

4.4 Attitude maintenance and deflection error correction in tunneling

4.4.1 Pipe jacking attitude maintenance and deflection correction

During the pipe jacking tunneling process, the machine may undergo attitude changes in three directions: pitch, transverse, and roll. In particular, the occurrence of pitch and transverse change eventually leads to an offset of the actual tunneling axis from the designed tunneling axis in the vertical and horizontal directions, respectively; rolling results in a rotation angle, as shown in Fig. 16. Therefore, when an attitude error occurs, it must be corrected promptly. The attitude measurement of the machine head must be conducted at the end of each pipe jacking step to ensure that the deflection can be corrected in time. Because a quasi-rectangular pipe jacking machine is adopted in the current study, the lateral level of the pipe must be maintained. The cutter reversal and ballast iron methods can be adopted to correct the rotation of the machine head and pipe section. It should be noted that the deviation of the rotation angle for jacking should be within ±15′.

4.4.2 Attitude maintenance control and deflection correction for shield tunneling

The shield machine can be controlled by correcting the jacking equipment. As shown in Fig. 17, the jacking equipment consists of 32 cylinders distributed along the circumference of the shield shell. The jacking cylinders have a long stroke cylinder, which is suitable for general segments with a staggered assembly. The propulsion system cylinders are supplied with a proportional pump, and its propulsion speed can be manually adjusted; this speed is stepless and can be accurately adjusted.

During the tunneling process, it is also necessary to control the shield rotation angle within 0.6° through the comprehensive application of a single or combined deflection correction countermeasures. Otherwise, it is prone to risks such as fragmentation of segments and difficulties in assembly, as shown in Fig. 18. Rotation prevention and corrective methods for the shield machine are carried out as follows. 1) The attitude at the base is kept horizontal to ensure that the rotation angle of the machine is 0° at the exit of the tunnel. 2) Timely correction of the rotation through the forward and reverse rotation of the cutterhead during tunneling. 3) Addition of heavy objects such as lead blocks to one side of the shield to achieve rotation correction. Additional grouting holes

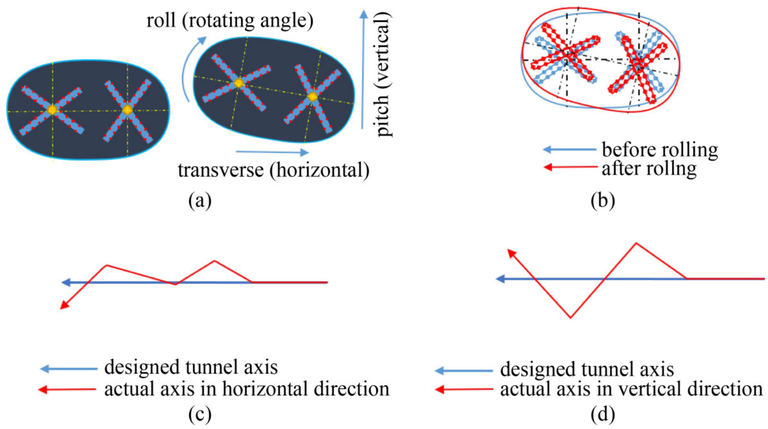

Fig. 16 Attitude deviation diagram: (a) the attitude deviation of tunneling machine; (b) the machine's rotation angle; (c) the machine's attitude deviation in horizontal direction; (d) the machine's attitude deviation in vertical direction.

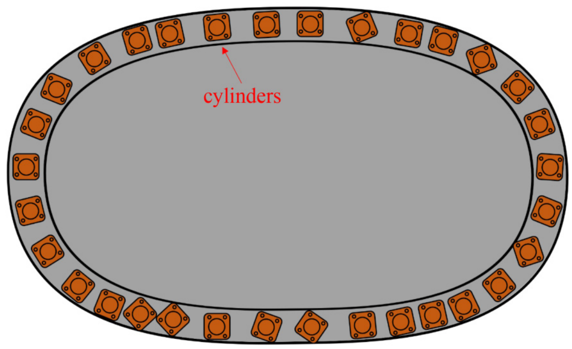

Fig. 17 Diagram of the cylinder distribution for the quasi-rectangular machine.

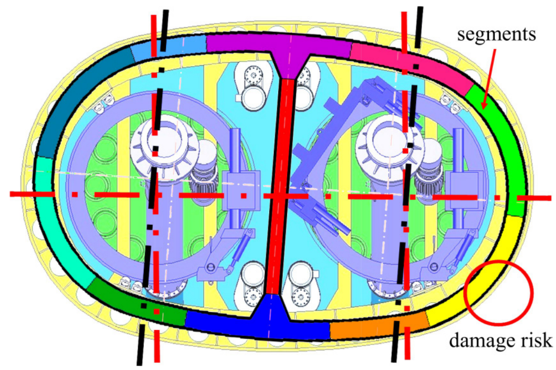

Fig. 18 Risk of deflection of shield machine rotation angle.

should be opened in the shield shell, enabling the shield rotation to be controlled and adjusted by injecting the single liquid grouting into the grouting holes.

As disclosed by the site investigation for the current project, the soil of the stratum in the ancient sea pond area is significantly poor (Fig. 19), and its thickness is nearly equal to the burial depth of the tunnel. Under such conditions, uplifting of the shield machine or tunnel lining tends to occur. Therefore, it is necessary to continuously monitor the uplift of the quasi-rectangular tunnel during crossing.

When the quasi-rectangular tunnel tends to float, the following measures must be taken to reduce floating. 1) Iron blocks are used in the tunnel for ballast, and the space below the rail bearing in the tunnel can surcharge the loaded iron blocks, which is synchronized with shield tunneling. Shield segments are used for ballast in the area

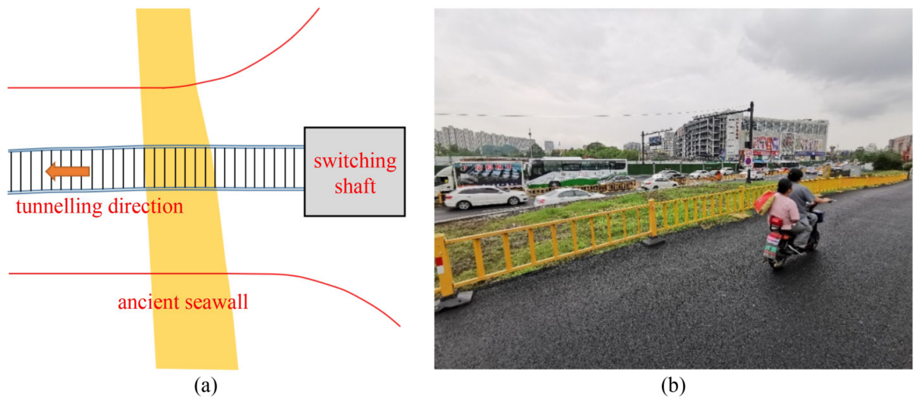

Fig. 19 Plane position and surrounding environment of the ancient seawall: (a) plane position relationship between the ancient seawall and tunnel; (b) environment around the ancient seawall.

between the iron block surcharge loading area and shield assembly working surface. 2) To strengthen the connection of segments in the tunnel, shear dowels are used in each ring when the quasi-rectangular shield crosses the ancient seawall area. Simultaneously, a pretension force is longitudinally imposed on the segments through tension bars to further strengthen the integrity of the formed tunnel and synchronized with the shield tunneling to reduce the uplift scale.

4.5 Ground disturbance control

4.5.1 Ground disturbance control of pipe jacking tunneling

The pipe jacking tunnel passes beneath the Qiushi Bridge, which is an elevated bridge for an urban roadway of width 35 m. The closest distance between the bridge pier and pipe jacking tunnel is approximately 18.17 m, as shown in Fig. 20. In addition, there are several municipal pipelines in the shallow ground above the tunnel, including sewage and rainwater pipes. Among these, the closest sewage pipe lays approximately 5.95 m above the tunnel vault, as illustrated in Fig. 20. These facilities must be considered during the tunneling process. Therefore, careful and rigorous ground disturbance control during tunneling is required to minimize its negative impacts on existing facilities.

The ground settlement and heave caused by tunnel construction should be controlled within allowable limits; during pipe jacking, ground settlement and uplift should be controlled to within 30 and 10 mm, respectively. The surface settlement data for the various cross sections (CS-1, CS-2, and CS-3) in the launch area, middle area, and Qiushi Bridge area during tunneling were analyzed (Fig. 21). Figure 22 demonstrates that the recorded maximum surface settlement during tunneling lies between 15 and 17.5 mm, which is far less than the allowable maximum settlement of 30 mm. Additionally, the surface settlement was predicted by using the empirical Peck formula. The shape of the Peck formula in the lateral direction is mainly related to the settlement trough width factor (i), whereas in the vertical direction, it is associated with the ground loss rate (δ). According to Ref. [25], the shape of the Peck formula was corrected in the lateral and vertical directions by introducing two coefficients, α (correction factor for settlement trough width) and λ (correction factor for ground loss rate), respectively (Eqs. (7)–(11)). The predicted settlement values were close to the actual settlement when λ was within 1.2–1.4, and α ranged within 1.4–1.6. Based on the formula prediction and actual measurements, the surface settlement was proven to be effectively controlled

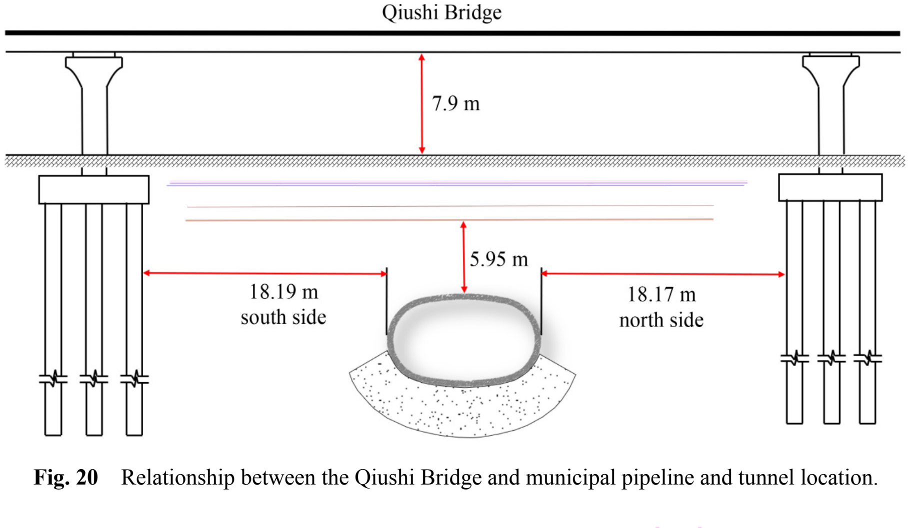

Fig. 20 Relationship between the Qiushi Bridge and municipal pipeline and tunnel location.

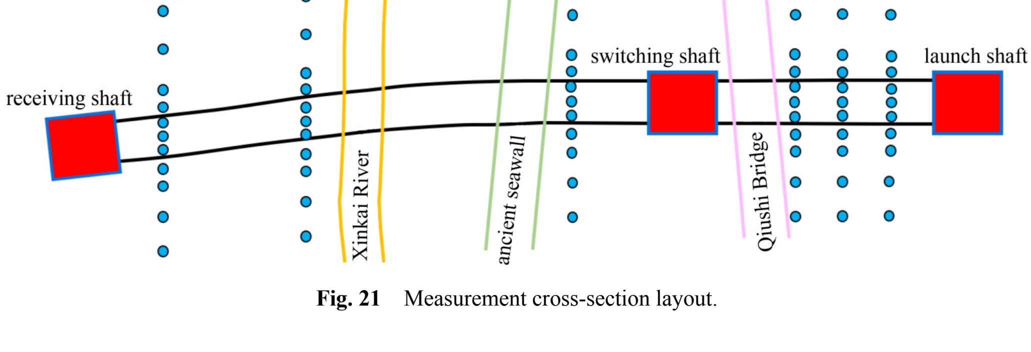

Fig. 21 Measurement cross-section layout.

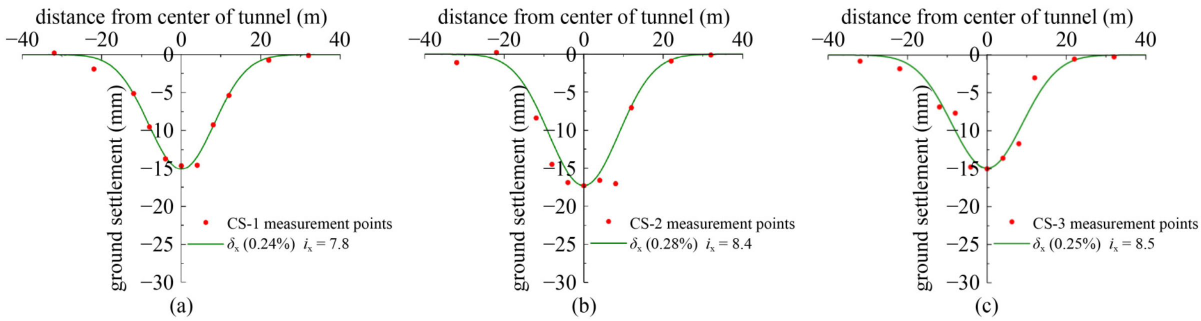

Fig. 22 Field measurement and Peck formula prediction of the pipe jacking ground settlement: (a) CS-1; (b) CS-2; (c) CS-3.

during the entire pipe jacking process; hence, the disturbance to the surrounding environment was concluded to be small.

Quasi-rectangular Peck formula:

$$S(x) = \frac{V_{sx}}{\sqrt{2\pi}\times i_x}\times\exp\left(-\frac{(x+2.3)^2}{2i_x^2}\right) + \frac{V_{sx}}{\sqrt{2\pi}\times i_x}\times\exp\left(-\frac{(x-2.3)^2}{2i_x^2}\right), \tag{7}$$

$$V_{sx} = \lambda\cdot\delta\cdot\pi\cdot r^2 = \delta_x\cdot\pi\cdot r^2, \tag{8}$$

$$\delta_x = \lambda\cdot\delta, \tag{9}$$

$$i_x = \alpha\cdot i, \tag{10}$$

$$i = r\times\left(\frac{Z}{2r}\right)^{0.8}, \tag{11}$$

where $S(x)$ is the surface settlement at a lateral distance x from the centerline of the tunnel (m), i is the settlement trough width factor, and i_x is the modified quasi-rectangular tunnel settlement trough width factor (m), V_s is the ground loss per unit length of the double circle shield tunnel, and V_{sx} is the modified ground loss rate per unit length of the quasi-rectangular tunnel (m^3/m). δ is the ground loss rate in the double-circle Peck formula and r is the tunnel excavation radius (m). For non-circular tunnels, the calculation can be performed according to $r = \sqrt{W/\pi}$ (4.614 m), where W (71.98 m^2) is the tunnel excavation area. Z is the tunnel-axis depth (m).

Secondary consolidation settlement is likely to occur while tunneling in soft ground, which may cause further settlement of the pipe jacking tunnel and disturbance to the surrounding facilities. In this project, secondary grouting injection should be performed in a timely manner according to the measured settlement data. The position of the grouting hole and grouting volume can be determined on-site to better control the deformation of the elevated bridges and pipelines. The ground beneath the tunnel in this interval is reinforced by adopting the Metro Jet System (MJS) as the tunnel in this area is a turn-back line switch rail section at a later stage (scissors crossover section). The unconfined compressive strength of the reinforced soil exceeds 1.2 MPa. As shown in Fig. 23, the reinforcement depth was 11 m below the bottom of the tunnel. The mixing proportion of the secondary grouting was 50% cement grouting and 50% ordinary Portland cement (P.O 42.5).

4.5.2 Ground disturbance control of shield tunneling

As shown in Fig. 24, the shield tunnel is constructed across the Xinkai River with a width of 12 m and a depth of 4.4 m. Owing to the shallow burial depth and significant changes in the overburden, as well as the presence of confined water in the crossing area, mishaps such as shield tail leakage during shield tunneling may easily occur. Therefore, it is necessary to take additional measures to prevent shield tail leakage in the river-crossing section.

A quantitative and sufficient amount of grease was injected to improve the seal of the shield tail. The amount of segment wedge was controlled in advance to ensure the alignment of the assembled segments and advancement axis; thus, the leakage of the shield tail owing to the uneven gap of the shield tail can be reduced. Shield tail grease injection points on both sides of the leakage point were used to inject the grease. In the event of ineffective shield tail grease sealing, sponge strips were placed under the segments to reduce the shield tail

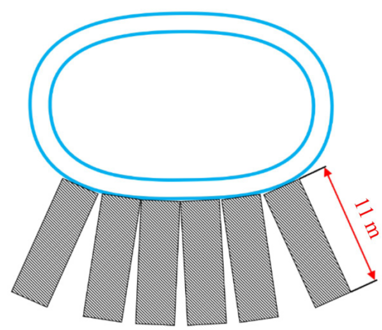

Fig. 23 MJS reinforcement diagram.

Fig. 24 Plane position of the new river and the surrounding environment: (a) relationship between the plane position of the new river and the tunnel; (b) surrounding environment of the Xinkai River.

gap. If necessary, polyurethane hoops were used to seal the shield tails.

During shield tunneling, the ground surface deformation was measured to provide feedback on the effectiveness of the tunneling control. As shown in Fig. 21, three cross-sections, CS-4, CS-5, and CS-6, located in the ancient seawall area, Xinkai River area, and receiving area of the shield section, respectively, were selected. The measured surface settlement data for the three aforementioned cross sections are shown in Fig. 25. The maximum surface settlement of the quasi-rectangular shield section is observed to be in the range of 24–28.1 mm, which is less than the maximum allowable settlement of 30 mm. On the ground near the Xinkai River (CS-5), the maximum surface settlement reaches approximately 28.1 mm, very close to the allowable value of 30 mm, which is mainly owing to the poor geological conditions in the area. For other areas, such as CS-6, the surface settlement control effect was significantly better than that of the Xinkai River area. Figure 25 demonstrates that the actual settlement values are near the predicted results when using Eqs. (7)–(11), with λ ranging between 0.91 and 1 and α ranging between 1 and 1.1.

The monitored data demonstrated that the surface settlement was controlled within a reasonable range in both areas with dense buildings and areas with poor geological conditions. This further indicates that the ground disturbance can be sufficiently controlled when implementing the 0-θ method. Meanwhile, by comparing Figs. 22 and 25, it can be observed that the surface settlement control in the pipe jacking section is generally better than that of the shield section; thus we can conclude that the disturbance to the surrounding environment caused by pipe jacking is smaller than that of the shield in this project. In addition, this comparison also demonstrates that the range of the two correction factors (λ and α) for the quasi-rectangular shield is significantly smaller than that for the quasi-rectangular pipe jacking. This is mainly owing to the fact that the building gap of the quasi-rectangular pipe jacking is significantly smaller than that of the quasi-rectangular shield. Consequently, the respective settlement curves also demonstrated differences.

5 Conclusions

This study investigated the world's first quasi-rectangular metro tunnel constructed using the 0-θ method, a hybrid tunneling method combining the shield and pipe jacking methods. A special earth balance pressure tunneling machine was developed for technically adjusting the

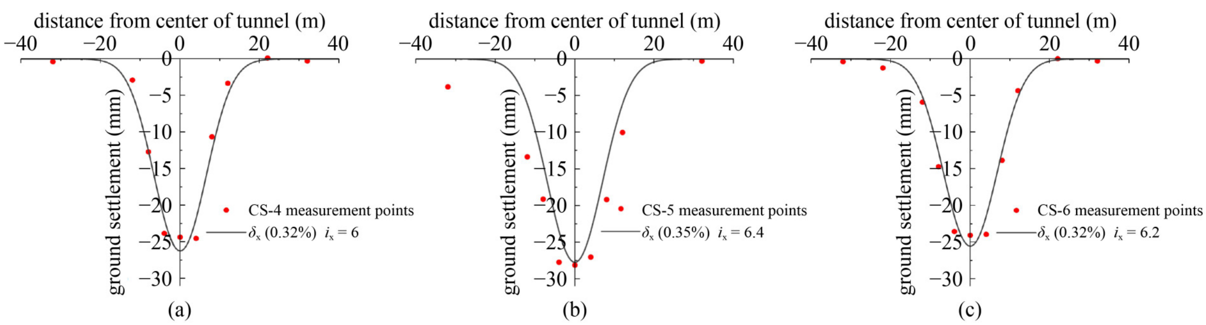

Fig. 25 Field measurement and Peck formula prediction of the shield ground settlement: (a) CS-4; (b) CS-5; (c) CS-6.

tunnel construction under the shield and pipe jacking modes. The key technologies for mode switching were investigated, and the challenges and corresponding countermeasures for construction in complex environments were presented. The applicability of this 0-θ method was validated through a practical case study.

The conclusions of this study are as follows.

1) The 0-θ method is highly applicable for constructing metro tunnels that contain scissors crossover and flat curve sections. Furthermore, it can be used in rail transit distribution structures, such as crossover/turn-back lines and the main body of stations, as well as structures such as shallowly buried tunnels, ramps, and air ducts of underground expressways. This method has been proven to reduce the construction cost and facilitate station extensions in the future, and could have even more advantages in the future.

2) The 0-θ method requires convenient switching between the two tunneling modes of pipe jacking and shield; hence, a dedicated switching device has been designed. It functions by changing the force transfer support system in the pipe jacking mode to the propulsion system in the shield mode; meanwhile, the segment assembly machine is installed. In this manner, switching between the pipe jacking and shield modes is completed.

3) Ancillary construction measures should be implemented for safety control during the implementation of the 0-θ method. These measures include tunneling parameter control, proper grouting, attitude control, uplifting, and leakage control. In this study, the disturbance of the ground surface can be controlled in the range of 15−28.1 mm, which meets the control standard of the industry specification (surface settlement ⩽ 30 mm and uplift ⩽ 10 mm). Thus, the proposed construction measures are proven to be effective.

4) Currently, the 0-θ method requires the construction of a dedicated switching shaft to switch between the two working modes, resulting in higher costs. The 0-θ construction technology without switching shafts is a meaningful research direction for the tunneling industry.

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Conflict of Interest The authors declare that they have no conflict of interest.

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