A case study of newly tunnels over-crossing the existing subway tunnels

Ben Wu¹, Wei Liu¹, Peixin Shi¹, Xiangyang Xu¹, and Yingjing Liu²

Abstract
Due to the continuous expansion of congested urban areas, many new tunnels are inevitable to over-cross the existing subway lines and may even affect the operation of existing lines. It is vital to investigate the response of existing tunnel caused by over-crossing tunneling. In this study, a case history of closely spaced twin tunnels excavated above the existing tunnels in soft soil stratum was presented. The deformation of the existing tunnels induced by the excavation of the new tunnels was automatically monitored. In-situ monitoring results showed that the vertical displacement of the existing tunnels was mainly uplift and its development showed obvious phase characteristics. The increase rate of the vertical displacement in Phase I and II induced by the second over-crossing was smaller than that of the first over-crossing. A superposition method was employed to describe the uplift section characteristics of the existing tunnels. The influence ranges of tunnel excavation on the left and right lines of the existing tunnels were approximately 5.5D and 4.5D, respectively. The torsional deformation of the rail bed and the convergence of the existing tunnels are explored, and the reasons for the changes of the over-crossing sections are analyzed at the same time.

Keywords
Shield tunneling, over-crossing, in-situ monitoring, tunnel deformation, tunnel behavior

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Introduction
With the rapid economic development and urbanization of China, there have been more than 44 cities that put their urban railway transit system into operation by 2020. For the coastal cities, such as Shanghai and Hangzhou, in which soft soil stratum is distributed throughout the underground space, the tunnel construction is usually conducted by the shield tunneling method.¹⁻³ During the excavation of the shield machine, the effects of shield tunneling construction on the surrounding environment or existing structures are not negligible, especially for the running metro lines. Besides, the unreasonable setting of driving parameters will also cause the instability of tunnel face and the collapse of the ground surface.⁴⁻⁶ Wei et al.⁷ summarized the cases of shield tunneling crossing construction in Hangzhou by 2020 and concluded that the crossing cases account for about 74%, while the parallel cases account for about 26%. Therefore, it is of great significance to investigate the influence on the existing tunnels due to the construction of the new tunnels.

In previous studies, the methods for the study on the response of the existing tunnels induced by crossing

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shield tunneling mainly include theoretical analysis, model test, numerical analysis, and field observation. The theoretical method is characterized by a series of assumptions to simplify the calculation. Wu et al. proposed a soil–tunnel interaction model based on the Timoshenko beam theory and discussed the effect of the equivalent shear stiffness and the rotational stiffness of the joint between soil and station. Furthermore, Zhang et al. adopted the Kerr foundation in the soil–tunnel interaction to calculate more realistically, which can consider the compression and shear response of the ground superior to conventional Winkler or Pasternak foundation model.

Boonyarak and Ng investigated the influence of different cover depths on the interaction between new tunnels and existing tunnels, and incremental normal stress acting on the existing tunnels was measured. Recently, the effectiveness of soil improvement and grouting around tunnels on protecting existing tunnels was investigated by Meng et al. with a series of three-dimensional centrifuge tests. It was found that soil improvement had a restriction effect on the diagram wall and ground and achieved desirable results on the reduction of tunnel heave. Although the centrifuge model tests can establish the relationship between the model and the practice, they cannot reflect the real response of existing tunnels due to the scale effect.

Many researchers have investigated the responses of new tunnels crossing on existing tunnels with the method of in situ monitoring. Cooper et al. reported the monitoring data and analysis results obtained from the existing twin 3.8 m diameter Piccadilly Line running tunnels, caused by three new 9 m diameter under-crossing tunnels. The short- and long-term behaviors of existing tunnels were illustrated, respectively. Chen et al. investigated the application of the Metro Jet System (MJS) on the response of stratum and overlying tunnels during the shield tunneling and used the superposition technique Fang et al. to describe the settlement profiles of the existing tunnels. Lai et al. presented a special case study of a shield tunnel obliquely constructed in close proximity to an above existing tunnel with a small intersection angle. The settlement of existing tunnel during under-crossing tunneling was monitored with an advanced automatic monitoring system.

Case studies above mainly focuses on the response of the existing tunnel caused by the under-crossing construction of shield tunneling, while there are relatively few studies on the influence of over-crossing construction of shield tunneling. Ghaboussi et al. presented the case that a sewage tunnel over-crossed an existing subway tunnel in New York, where the minimum clear distance between the two tunnels was about 0.5 m. The underlying tunnel was found to be vertically elongated when the new tunnels advanced above. Liao et al. carefully introduced a case that shield machine crossed existing tunnels from above in Shanghai soft ground. It was summarized that the tunnel support pressure and grouting control were most important key parameters for the shield machine, which was urgently decisive to be improved. Zhu et al. analyzed the in situ measured data of heave deformation of the existing tunnels in the project of Metro Line-13 tunnels over-crossing existing Metro Line-4 tunnels in Shanghai. Gaussian distribution curve was employed to predict the longitudinal uplift of the existing tunnels and founded that the variation of the uplift of the existing tunnels caused by the over-crossing excavation was similar to that of the ground settlement.

According to the literature, current studies mainly focus on the interaction between the existing tunnels and under-crossing tunnels, and minimal attention has been given to the effect of over-crossing tunneling construction on the existing tunnels. In this paper, a case study of closely spaced twin tunnels excavated above the existing tunnels in soft soil stratum was presented. Field observation was employed to investigate the response of the existing tunnels to the new tunnels excavation. The deformation characteristics, the convergence, and the rotation of the existing tunnels were analyzed based on the monitoring data.

Project overview

Geological profile

The plan view and over-crossing section view of the existing tunnels and new tunnels are shown in Figures 1 and 2, respectively. The existing circular shaped twin tunnels belong to Line 1 of the Hangzhou metro. The internal and external diameters of tunnel linings are 5.5 and 6.2 m, respectively. The width of each segment is 1.2 m. The horizontal distance between the axes of the existing tunnels is 15.6 m. The depth of the tunnel crown is about 8.2 m.

The new over-crossing tunnels are the parts of the Hangzhou metro Line 7, from Chengzhan Station to Jiangcheng Road Station, as shown in Figure 1(b). The sizes of tunnel linings of the new tunnels are the same as those of the existing tunnels. The horizontal distance between the axes of the new tunnels is 15.8 m. The minimum vertical clear distance between the existing tunnels and new tunnels is 1.9 m, and there is a skew angle of 83° between the existing tunnels and new tunnels. The new tunnels are driven by the earth pressure balance shield (EPBS). The first over-crossing of the left tunnel of Line 7 began on July 25, 2020 and ended on August 15, 2020. The second over-crossing of the right tunnel of Line 7 began on approximately 6 months later, on January 12, 2021 and ended on February 3, 2021. The over-crossing areas of the left and right tunnels of Line 7 are 21st ring to 40th ring and 24th ring to 43rd ring, respectively.
A typical geological profile of the over-crossing section is shown in Figure 2. The profile shows that the existing tunnels and new tunnels are mainly located in soft soil, which has the engineering characteristics of low shear strength, high sensitivity, and high compressibility. The groundwater table is 2 m below the ground surface. The physical and mechanical parameters of the soils obtained by the site investigation are shown in Table 1.

Arrangement of the measurement on existing tunnels

During the excavation of the new tunnels, the deformation of the existing tunnels was monitored to study the interaction between the new tunnels and existing tunnels to optimize the working parameters of shield machine. The layout of the monitoring points along the over-crossing section is shown in Figure 3. The automatic monitoring system with resolution of 0.1 mm was employed to monitor the deformation of the existing tunnels, which consisted of the Leica TM50 automatic total station and reflecting prisms, as shown in Figures 4 and 5. The monitoring points in the over-crossing area (500th–530th ring) are arranged with encryption, and a monitoring point is set in every two to three rings, while a monitoring point is set in every five rings in other areas. For each monitoring section, five reflecting prisms are installed at different locations in the tunnel linings. One prism is installed at the tunnel crown, two prisms are installed at the tunnel springline, and two prisms are installed at the rail bed. As shown in Figure 3, the first parenthesized texts “L” and “R” denote the left tunnel and right tunnel, respectively, and the associated numbers denote the ring number. The second parenthesized texts “MP” denote the monitoring point, and the associated numbers denote the prism number. It should be noted that the horizontal displacement of the tunnel is positive with convergence, while the vertical displacement is positive with uplift.

Driving parameters of EPBS

Two S585 shield machines manufactured by Herrenknecht AG were employed in the construction of the new tunnels. The excavation diameter and length of shield machines are 6.47 and 7.50 m, respectively. The shield machines were thoroughly inspected and maintained before excavating, and all cutting tools were replaced to avoid breakdown of the shield machine over the existing tunnels.

During the excavation of new tunnels, driving parameters of shield machine in the first 10 rings of left line...
Table 1. Physical and mechanical parameters of the soils.

| Soil layers                  | $\gamma$ (kN/m$^3$) | $w$ (%) | $c$ (kPa) | $\varphi$ ($^\circ$) | $E_s$ (MPa) | $\nu$ |
|------------------------------|----------------------|---------|-----------|----------------------|-------------|-------|
| <1-1> Miscellaneous fill     | 17.5                 | —       | 8.0       | 15.0                 | 3.0         | 0.33  |
| <3-2> Sandy silt             | 19.4                 | 26.7    | 6.0       | 29.0                 | 8.5         | 0.32  |
| <3-3> Silt                   | 19.6                 | 24.8    | 5.0       | 33.0                 | 14.0        | 0.28  |
| <3-5> Silt with sandy silt   | 19.7                 | 23.6    | 5.0       | 32.0                 | 15.0        | 0.27  |
| <3-7> Sandy silt             | 19.2                 | 29.5    | 7.0       | 24.0                 | 7.0         | 0.34  |
| <7-2> Silty clay             | 19.7                 | 26.5    | 30.0      | 18.0                 | 12.0        | 0.32  |
| <8-2> Silty clay             | 18.6                 | 35.8    | 32.0      | 14.0                 | 6.0         | 0.33  |
| <10-2> Silty clay with silt  | 18.2                 | 36.8    | 12.0      | 16.0                 | 4.0         | 0.37  |

$\gamma$: unit weight; $w$: water content; $c$: cohesion; $\varphi$: friction angle; $E_s$: constrained modulus; $\nu$: Poisson’s ratio.

Figure 3. Layout of the monitoring points along the over-crossing section.

Figure 4. Automatic total station.

Figure 5. Measuring point of the reflecting prism.
were used as reference. After the completion of the construction of the left line, driving parameters of right line of new tunnels were also set according to the parameters obtained by the excavation of left line of new tunnels. Driving parameters of shield machine were optimized constantly to reduce the deformation of existing tunnels and the disturbance to the stratum. The changes of main driving parameters in the first 100 rings of left line of new tunnels are shown in Figures 6 and 7. It can be found that the change of the total thrust is roughly bounded by the 40th ring and 50th ring. The total thrust changes little in the first 40 rings, while increases in step from the 40th to the 50th ring, and gradually tends to be stable after the 50th ring. The changes of other driving parameters are similar to the total thrust, except that the driving speed decreases in the 40th to 50th ring. The reason is that the stratum strength near the 40th ring changes greatly, which leads to a decrease in the driving speed.

It can be seen from Figure 7 that the earth pressure in the chamber at all different parts of the tunnel face shows an increasing trend in the first 10 rings and then gradually tends to be stable. The comparison results show that the pressure in the upper silo is the smallest, while the pressure in the right silo is the biggest. The pressure in the right silo is greater than that in the left silo. This is because the upper silt stratum is close to the surface and loose, and the soil on the right side is squeezed by shield machine.

Based on the monitoring data of main driving parameters in the first 100 rings of left line of new tunnels, recommended values of driving parameters are obtained, which have been marked in Figures 6 and 7. More detailed driving parameters of shield machine are listed in Table 2. It can be found that most monitoring values fall within the recommended value range, the recommended values can be used as the control basis for subsequent driving parameters.

Tail grouting is the main means to fill the building gap between the soil and segments and to reduce the deformation of tunnel linings. The effect of tail grouting is mainly determined by the setting time, strength, consistency, and so on. The field grouting test was carried out before the new tunnels excavating. Based on the results of the field grouting test, the optimal mix proportion of tail grouting is determined, as shown in Table 3.

The pressure of tail grouting was controlled in 0.2–0.25 MPa. The theoretical volume of tail grouting can be calculated by

\[
V_g = \frac{\pi L (D^2 - d^2)}{4} = \frac{1.2\pi \times (6.47^2 - 6.2^2)}{4} = 3.22 \text{ m}^3
\]

where \(V_g\) is the volume of synchronous grouting, \(L\) is the length of the shield machine, \(D\) is the excavation diameter, and \(d\) is the external diameter of tunnel linings.

The grouting volume of each ring in the trial excavation is generally 180%–200% of the theoretical volume.
of tail grouting. Thus, the grouting volume of each ring is controlled in 6–6.5 m$^3$. Combined with the field grouting test, the pressure of secondary grouting is controlled in 0.3–0.4 MPa. The mix proportion of secondary grouting is shown in Table 4.

### In-situ monitoring results

**Vertical displacement development of the existing tunnels**

In this section, two typical monitoring sections L510 and R525 are selected to analyze the vertical displacement development of the existing tunnels, which are located at the centerlines of left and right lines of the new tunnels. The vertical displacement development of L510 given by the monitoring point MP1 and the increase rate are shown in Figure 8. It can be found that the vertical displacement development experienced four phases:

(a) Phase I: The shield machine had not reached the over-crossing section. Due to the shield thrust and friction between the soil and shield machine, the uplift of L510 gradually increased to 2.3 mm.

(b) Phase II: The shield machine passed through the left line of the existing tunnels. The uplift of the existing tunnels increased rapidly as a result of the excavation unloading of soil strata stress redistribution. The increase rate of the vertical displacement remained in 0–0.1 mm/d.

(c) Phase III: The shield machine drove away from the over-crossing section. The uplift of the existing tunnels continued to increase, while the increase rate was reduced due to the grouting reinforcement of the existing tunnels, indicating that the uplift deformation has been effectively controlled.

(d) Phase IV: The distance between the shield machine and the left line of the existing tunnels is greater than 1.0D. The uplift of the existing tunnels tended to be stable with the gradual emergence of grouting effect. The soil consolidation induced by the dissipation of excess pore water was the main reason for the variation of the deformation in this phase. The final uplift of L510 was approximately 3.0 mm.

![Figure 8. Vertical displacement development of L510 during the first over-crossing.](image)
The vertical accumulative displacement and incremental displacement development of R525 during the second over-crossing are shown in Figure 9(a) and (b), respectively. The accumulative displacement during the second over-crossing is similar to the first over-crossing with obvious stage characteristics. Besides, it can be seen that the incremental displacement and increase rate caused by the second over-crossing are smaller than that of the first over-crossing. This indicates that the influence on the existing tunnels of the first over-crossing is larger than the second over-crossing, and the grouting in the existing tunnels reduces the displacement to control the uplift of the existing tunnels. It should be noted that after the shield machine drove away from the right line of the existing tunnels, there was an obvious increase process of the uplift of the existing tunnels, as shown in Figure 9(a). The incremental uplift of the existing tunnels was about 0.4 mm. This may be responsible for the change of groundwater level, which caused the soil deformation. The similar phenomenon was observed by Chen et al. The uplift of the existing tunnels tended to be stable after the shield machine drove away from the over-crossing section 1.25D distance. The final accumulative uplift of R525 was about 5.8 mm, which was greater than the first over-crossing.

The deformation of the existing tunnels during the crossing construction of the shield machine is a key problem in shield engineering. Figure 10 shows the proportion of the uplift (Δ) caused by the over-crossing construction to the total uplift (S) for the four typical monitoring sections, where L510 and R510 are located at the left tunnel centerline of the new tunnels, and L525 and R525 are located at the right tunnel centerline of the new tunnels. It is found that the influence of the first over-crossing on L510 and R510 is greater than that on L525 and R525. The main reason is that L510 and R510 are located at the center of left line of new tunnels, which are more affected by shield tunneling excavation. The uplift ratios Δ/S of L510 and R510 are 23.3% and 22.8%, respectively. However, Δ/S of L525 and R525 after the second over-crossing are significantly greater than that of L510 and R510. The maximum Δ/S of 37.8% occurs at the monitoring section L525. The increase of Δ/S from the first over-crossing to the second over-crossing shows an obvious accumulative effect induced by the crossing construction of the shield machine, so more attention should be paid to protection of the existing tunnels in the second over-crossing excavation.

**Vertical displacement profiles of the existing tunnels**

Figure 11 presents the vertical displacement profiles of the existing tunnels given by the monitoring point MP1 during the first over-crossing. The vertical displacement of the left line of the existing tunnels is mainly uplift (see in Figure 11(a)). The maximum uplift is
approximately 3.5 mm, which is located 6 m away from the left tunnel centerline. The reason may be related to the oblique crossing angle of the shield machine. The vertical displacement profile of the right line shows the same distribution as the left line, as shown in Figure 11(b). The maximum uplift of the right line is approximately 4.6 mm, which is larger than that of the left line.

The vertical displacement profiles of the left and right lines of the existing tunnels caused by the second over-crossing are shown in Figure 12(a) and (b), respectively. Compared with the first over-crossing, the symmetry centers of the vertical displacement of the existing tunnels are shifted to the midpoint of the centerlines of the new tunnels (517th ring). The maximum uplifts of the left line and right line increase by 60% to 5.6 mm from 3.5 mm and 39% to 6.4 mm from 4.6 mm, respectively. It should be noted that the longitudinal uplift scope of the existing tunnels is larger than that of the first over-crossing, and more attention should be paid to the construction.

According to the monitoring data of MP1, the typical section characteristics of the existing tunnels can be observed. Because the tunnel is a flexible structure, the Gaussian distribution curve can be used to describe the settlement trough of the existing tunnels caused by the shield tunneling.\(^2,14\) The equation for describing the shape of the settlement trough is expressed as

\[
S = S_{\text{max}} \exp\left(-\frac{x^2}{2i^2}\right) = \left(AV\right)\frac{V_s}{A} \exp\left(-\frac{x^2}{2L^2}\right).
\]

where \(x\) is the distance from the tunnel centerline, \(i\) (or trough width) is the distance from the tunnel centerline to the inflection of the trough, \(S_{\text{max}}\) is the maximum settlement, \(A\) is the tunnel cross-sectional area, \(V = V_s/A\), and \(V_s\) is the volume loss due to tunneling.

For the separately constructed twin tunnels, as the existing tunnel forces are more complex, only a Gaussian distribution curve is impossible to describe the uplift section characteristics of the existing twin tunnels. In this paper, a superposition method is
introduced to calculate the accumulative uplift caused by the over-crossing excavation of twin tunnels.\textsuperscript{2,13} The equation of the method is expressed as
\begin{equation}
S = \frac{A_1}{i_1\sqrt{2\pi}} \exp\left[-\frac{(x + L/2)^2}{2i_1^2}\right] + \frac{A_2}{i_2\sqrt{2\pi}} \exp\left[-\frac{(x - L/2)^2}{2i_2^2}\right]
\end{equation}
where the subscripts 1 and 2 represent the excavation of the left line and the right line of the new tunnels, respectively, and $L$ is the horizontal distance between the centerlines of the left and right line of the new tunnels.

The uplift monitoring data of the existing tunnels and the fitting values of the first over-crossing and second over-crossing obtained by the superposition method are shown in Figures 13 and 14, respectively. The fitting parameters of the existing tunnels are listed in Table 5. The degrees of fitting $R^2$ are all above 0.8, which indicates that the monitoring data fit well with the proposed method. According to the monitoring data and fitting results, it can be found that the volume losses $V_1$ and $V_2$ of the second over-crossing (0.52%, 0.63% of left line or 0.50%, 0.57% of right line) are greater than those of the first over-crossing (0.43% of left line or 0.37% of right line). The trough widths $i_1$ and $i_2$ have the same characteristics. Many case studies have reported differences in the displacement profiles between the first and second shield tunneling constructions.\textsuperscript{2,16,20} The differences between the trough width induced by the first and the second over-crossing are also observed in this study. The volume loss and the trough width of the left line of the existing tunnels are greater than that of the right line. The fitting results of the trough widths $i_1$ and $i_2$ show that the influence range of tunnel excavation on the existing tunnels is approximately 5.5D for the left line and 4.5D for the right line. The reasons for these different characteristics are primarily due to the disturbance of the surrounding soil when the first over-crossing was underway and the reduction of surrounding soil stiffness.\textsuperscript{14}

\begin{table}[h]
\centering
\caption{Fitting parameters of the existing tunnels.}
\begin{tabular}{|c|c|c|c|c|}
\hline
 & \multicolumn{2}{c|}{First over-crossing} & \multicolumn{2}{c|}{Second over-crossing} \\
 & \multicolumn{1}{c|}{Left line} & \multicolumn{1}{c|}{Right line} & \multicolumn{1}{c|}{Left line} & \multicolumn{1}{c|}{Right line} \\
\hline
$V_1$ (%) & 0.43 & 0.37 & 0.52 & 0.50 \\
$V_2$ (%) & – & – & 0.63 & 0.57 \\
$i_1$ (m) & 4.72D & 3.99D & 5.18D & 4.42D \\
$i_2$ (m) & – & – & 5.72D & 4.38D \\
$R^2$ & 0.90 & 0.81 & 0.86 & 0.97 \\
\hline
\end{tabular}
\end{table}

\textbf{Rotation of the existing tunnels}

As the shield machine obliquely crossed the existing tunnels with an intersection angle, the effect of the excavation of the new tunnels on the existing tunnels was local, which can cause the torsional deformation of the rail bed.\textsuperscript{8} Chen et al.\textsuperscript{2} proposed the rotation ratio $\lambda$ and rotation angle ($\alpha$) to characterize the torsional deformation of the rail bed, as shown in Figure 15. The rotation ratio and rotation angle can be calculated by
where $D_1$ and $D_2$ are the values of monitoring points MP3 and MP4, with the uplift specified as “positive”; $\alpha$ is specified as “positive” in case of counterclockwise; $L$ is the horizontal distance between MP4 and MP3, with $L = 2.8\, \text{m}$ in this case. Particularly, if the deformation of rail bed is mainly uplift, the relationship between $l$ and $\alpha$ can be expressed as

$$\lambda = \frac{\Delta_1}{\Delta_2}$$

$$\sin \alpha = \frac{(1 - \lambda) \Delta_2}{L}$$

The variations of $\lambda$ and $\alpha$ of the existing tunnels after the first over-crossing are shown in Figure 16(a). It can be seen that the rotation ratio of the left tunnel of Line 1 $\lambda_L$ is roughly symmetric about the left tunnel centerline of Line 7. The peak value of $\lambda_L$ is approximately 3.0, and $\lambda_L$ is less than 1.0 only at both ends of the tunnel, which indicates that the torsional deformation of the left tunnel of Line 1 is mainly clockwise. Compared to $\lambda_L$, the rotation ratio of the right tunnel of Line 1 $\lambda_R$ is generally less than $\lambda_L$, and the peak value of $\lambda_R$ is 2.8. Due to the large rotation ratio in the over-crossing section (505th ring–515th ring), the rotation angles of the left and right tunnel of Line 1 $\alpha_L$ and $\alpha_R$ are negative, as shown in Figure 16(a). The result shows that the torsional deformation is easy to occur in the section with large uplift deformation.

The variations of $\lambda$ and $\alpha$ of the existing tunnels after the second over-crossing are shown in Figure 16(b). The distribution of the rotation ratios of Line 1 $\lambda_L$ and $\lambda_R$ are symmetric about the centerlines of the left and right tunnel of Line 7. The peak values of $\lambda_L$ and $\lambda_R$ have increased to 4.5 and 3.3, respectively. The difference is that the rotation ratio at the midpoint of the new tunnels’ centerlines (517th ring) is obviously smaller than the peak value. The variations of $\lambda_L$ and $\lambda_R$ show an obvious characteristic of “double peak.” In addition, the monitoring results show that the rail bed of the existing tunnels rotates clockwise at the new tunnels centerline, while it rotates anticlockwise on both sides of the centerlines, indicating that the local torsional deformation has occurred near the new tunnels’ centerlines.

**Convergence of the existing tunnels**

In this section, the convergence of the cross-section of the existing tunnels caused by the shield tunneling is investigated. Two typical monitoring sections L510 and R525 are chosen, which are located at the centerlines of the left and right tunnel of Line 7. Based on the
In this study, a case of closely spaced twin tunnels excavated above the existing tunnels in soft soil stratum was presented. The deformation characteristics of the existing tunnels caused by the new tunnels excavation was analyzed based on the in situ monitoring data. The key conclusions from this study can be drawn as follows:

1. The vertical displacement of the existing tunnels caused by the new tunnels over-crossing is mainly uplift. Its development experienced four phases: stable, rapid, sustained increase, and tending to be stable. The increase rate induced by the second over-crossing is smaller than that of the first over-crossing, which remained in 0–0.1 mm/d. The maximum uplifts of the left line and right line are 5.6 and 6.4 mm, which are within the range of control values.

2. The superposition method is used to separate the accumulative uplift profile into uplift profiles attributed by twin tunnels. The volume losses and trough widths of the existing tunnels caused by the second over-crossing are greater than those of the first over-crossing. The size of the zone of influence of tunnel excavation on the existing tunnels are approximately 5.5D for the left line and 4.5D for the right line. These differences mainly result from the disturbance of the surrounding soil during the first over-crossing.

Figure 16. Torsional deformation of the existing tunnels (a) the first over-crossing of the new tunnels and (b) the second over-crossing of the new tunnels.

Figure 17(a) and (b) show that the variations of the convergence of two typical over-crossing sections L510 and R525 were same, while the convergence value of R525 in the Phase III was greater than that of L510, which was affected by the second over-crossing of the shield machine. The shield tunneling and the reduced load caused by excavation are the main causes of load changes on the existing tunnels. Thus, the changes of the shape of the over-crossing sections are prompted by the combined effect of the driving thrust of the shield machine and excavation unloading.14

Conclusion

In this study, a case of closely spaced twin tunnels excavated above the existing tunnels in soft soil stratum was presented. The deformation characteristics of the existing tunnels caused by the new tunnels excavation was analyzed based on the in situ monitoring data. The key conclusions from this study can be drawn as follows:

1. The vertical displacement of the existing tunnels caused by the new tunnels over-crossing is mainly uplift. Its development experienced four phases: stable, rapid, sustained increase, and tending to be stable. The increase rate induced by the second over-crossing is smaller than that of the first over-crossing, which remained in 0–0.1 mm/d. The maximum uplifts of the left line and right line are 5.6 and 6.4 mm, which are within the range of control values.

2. The superposition method is used to separate the accumulative uplift profile into uplift profiles attributed by twin tunnels. The volume losses and trough widths of the existing tunnels caused by the second over-crossing are greater than those of the first over-crossing. The size of the zone of influence of tunnel excavation on the existing tunnels are approximately 5.5D for the left line and 4.5D for the right line. These differences mainly result from the disturbance of the surrounding soil during the first over-crossing.
The torsional deformation of the rail bed of the existing tunnels decreases with the increase of the distance from the new tunnels centerlines, and the deformation is mainly clockwise. After the excavation of the shield machine, the torsional deformation curve presents an obvious “double peak” characteristic. The monitoring section shows an obviously elliptical shape after the shield machine completely drives through the existing tunnels. The ellipticity gradually decreases after the shield drives away (>1.25D). The convergence value is controlled within the allowable deformation range.

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