Safety Assessment of Tunnel Shield Construction for Existing Adjacent Bridges

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Disturbances to the surrounding soil layer from a tunnel shield passing under a viaduct can cause uneven pile foundation settlement, negatively affecting the structure’s normal functionality. Therefore, its safety must be evaluated before tunneling. In this study, a numerical model is developed on Midas to simulate the differential settlement of pile foundations under a viaduct during the entire construction tunneling process. The evaluation basis is derived by verifying the various bearing capacities of the viaduct. The results indicate that when the shield excavation face is 12 m from the pile foundation axis, vertical displacement begins to appear and increases gradually as the excavation face approaches the pile foundation. The bearing capacity of the girder support and midspan section is lower than the section resistance and meets the specifications. In the anti-cracking calculation, when the bearing differential settlement is 5 mm, there is no tensile stress in the normal section of the main beam, and the tensile stress of the oblique section meets the specifications. Furthermore, in situations when the pressure must be verified, the calculation results meet the specifications. Based on this evaluation criteria, it is ultimately determined that the viaduct can be used normally after shield construction. This safety assessment method can serve as a reference for similar construction projects.

1. Introduction

With the rapid growth of urbanization and urban population, urban roads, additional subway lines, and subway systems are increasingly being planned and constructed to alleviate the associated traffic pressure in densely populated urban areas [1]. Under the influence of urban planning and the surrounding environment, the construction of tunnels near buildings has become a closely monitored issue. The shield construction method is widely used because of its advantages, including fast construction, high mechanization, and minimal disturbance to strata [2, 3].

When there are no surrounding buildings, the ground deformation caused by tunnel excavation can be analyzed via the Peck and modified Peck formulas. Subway lines are generally planned along and near urban roads, along which there are overpasses, viaducts, and bridges. Under these conditions, a new shield tunnel will inevitably be adjacent to the pile foundation of these existing structures. Ground subsidence caused by tunnel excavation will redistribute the soil and produce lateral and axial forces on pile foundations. When the tunnel construction is close to a pile foundation, the relevant stress release around the pile foundation during tunnel excavation will be more obvious, resulting in a decrease in the foundation’s bearing capacity and an increase in settlement, which poses a risk to the normal and safe use of the pile foundation. When the settlement of bridge piles exceeds a certain limit, it will also adversely affect the normal operation of the bridge. Therefore, in the shield construction process, it is necessary to ensure the normal operation of the surrounding buildings to avoid their excessive displacement or uneven settlement [4–6].

Sagaseta [7] summarized the strain field caused by formation loss via the image source-sink method by assuming that the soil layer is an incompressible elastic semi-infinite body. Wei Gang [8] derived the longitudinal
settlement formula of the ground during shield construction by assuming that the soil is undrained and using the Mindlin solution of elastic mechanics. Furthermore, it was found that the additional thrust of the shield machine causes the front ground to uplift and rear settlement; moreover, the soil parameters have a great influence on ground deformation. Lee and Ng et al. [9] carried out centrifuge tests on soil and simulations of the tunnel excavation process and found that tunnel excavation has a greater impact on the longitudinal length of the tunnel within the diameter range. Additionally, it was found that tunnel excavation has an impact on the pile foundation of surrounding buildings. Chen [10] pointed out that the biggest problem in urban tunnel construction is the impact on surrounding buildings, with buildings being usually supported by pile foundations; therefore, it is very important to calculate the change in pile foundations. Huang [11] studied the influence of shield tunneling on pile pile foundations under high groundwater content conditions and analyzed three different conditions: without considering the influence of groundwater, considering the influence of groundwater, and fluid-solid coupling interaction. The results show that the actual monitoring data and the calculated values are more consistent with each other when the water-soil interaction is considered.

To accurately predict the pile foundation deformation and bending caused by ground displacement, Cheng [12] introduced a new numerical method to simulate tunnel construction by determining the tunnel boundary displacement size and direction. The results of this method are more consistent with the monitoring data than other methods, and the bridge pile foundation deformation during tunnel excavation is better predicted. Gordon [13] studied the change in the pile body during the tunnel advancing process, analyzed the pile-soil coupling consolidation and compared the results with theoretical analysis results. Considering the pile-soil interaction, HE [9] proposed a three-dimensional (3D) elastic finite element model to analyze the influence of shield excavation on single and multiple piles. Huang and Zhang [14] used a simplified two-stage analysis method to analyze the influence of shield tunneling on the surrounding pile foundation and calculated the bending and axial stress of the pile foundation using the attenuation function considering the shielding effect. Wang [15] studied the influence of tunnel excavation on pile foundations by analyzing the lateral and vertical deformations of the pile body and concluded that adverse shielding effects could be reduced after stratum reinforcement. Zhao [16] examined the protective effect of different reinforcement schemes on a bridge during shield construction. The validity of the reinforcement scheme was verified via numerical simulations based on an orthogonal design using FLAC, a two-dimensional (2D) numerical modeling software for advanced geotechnical analysis. Based on the SPA method, Zhai Qiang [17] considered factors such as shield construction parameters and bridge conditions as evaluation indices and calculated the weight of each factor for the safety assessment and sensitivity analysis of the bridge. Wu [18] used a cloud model to discretize the influence factors on bridge safety and determined the bridge risk sensitivity factors via a mathematical method, providing a new method for the safety assessment of structural damage to adjacent bridges induced by shield construction.

In general, previous studies have focused on the deformation characteristics of bridge pile foundations and on comparing different reinforcement schemes during shield excavation; however, the safety of the bridge itself has not been adequately investigated. For example, Zhai and Wu analyzed the factors affecting the safety of a bridge structure but did not assess the safety of the bridge structure. In this study, considering the Hefei S1 line interval tunnel shield construction project under the Yangtze River West Changjiang Road Viaduct as the background, and in accordance with the “Highway Bridge Concrete and Prestressed Concrete Bridge and Culvert Design Code,” the bridge structure and bearing capacity in the uneven pile foundation settlement are examined. Additionally, a safety assessment for normal operation of the bridge is performed. The findings of this study can guide practical engineering projects and provide a reference for similar engineering cases.

2. Project Overview

The Hefei S1 line runs through the West Changjiang Road Viaduct bridge pile. The round interval tunnel construction limit is φ5.2 m, the tube piece inner and outer diameters are 5.4 and 6.0 m, respectively; each ring tube piece has a width of 1.5 m. The viaduct is a φ1.5 m bored pile with a pile length of 21.8–23.8 m, buried at 11.8–13.9 m in medium weathered muddy sandstone. The minimum horizontal clearance between the tunnel and the pile foundation is approximately 2.5 m. The affected piers were numbered P177, P178, P179, and P180, and the calculated spans were 30, 40, and 30 m, respectively. Figure 1 illustrates the location of the inter-district tunnel in relation to the West Changjiang Road Viaduct.

3. Numerical Simulation Model

3.1. Finite Element Model. The Midas GTS numerical simulation software was used in this study. Figure 2 depicts the structural model of the West Changjiang Road Viaduct and inter-district tunnels. The model dimensions were 200 m × 100 m × 50 m (x × y × z). The distance from the structure to the edge of the model was more than 25 m. The bridge piers, bearing platforms, and soil layers 1–5 were simulated as 3D solids, whereas the single tunnel segment and shield machine housing were simulated as 2D plates and the bridge pile foundations as 1D beams. The bridge superstructure was represented as loads applied on each pier according to the actual dimensions of the viaduct. The shield was excavated, and the pipe slab was assembled by passivating the excavation area and activating the single tunnel segment, respectively.

3.2. Description of the Computation Steps. The model calculation steps are as follows: (1) analysis of the initial stress field; (2) activation of the viaduct structural unit; (3) clear
displacement calculation; and (4) tunnel excavation. The construction simulation process is as follows: (1) activate the shield housing; (2) excavate the soil and apply the tunnel face pressure; (3) propel the shield and generate the single tunnel segment; (4) continue excavating the soil and applying tunnel face pressure; (5) continue propelling the shield and generating new single tunnel segments; and (6), repeat steps 1–5 until the excavation is complete. Figure 3 shows a simulation of shield excavation. The actual size of the pipe piece (1.5-m-wide ring) could not be simulated owing to the large size of the model. Therefore, the excavation length was set to 6 m per ring, wherein every 6 m interval was a working condition, and two lines comprised 40 working conditions. As shown in Figure 4, monitoring points were placed at the center of the pile cap and at the top of the pile to monitor the displacement during the simulated construction.
3.3. Material Calculation Parameters. From top to bottom, the stratigraphic soils in the excavated area are artificial fill, clay, silt, full efflorescence argillaceous sandstone, and strong efflorescence argillaceous sandstone, with thicknesses of 3.2, 6.3, 1.1, 1.6, and 36.8 m, respectively. Grade C30 concrete was used for the viaduct pile foundations and the bearing platform, C40 for the bridge pier, and C50 for the viaduct and single tunnel segments. The modified M–C constitutive model was used to simulate each soil layer, while the elastic constitutive model was used for pile foundations, bearing platform, and bridge pier. The calculation parameters of each material are listed in Table 1 and Table 2.

3.4. Model Assumptions. The following assumptions were made in the model used in this study.

(1) The West Changjiang Road Viaduct has been in use for several years. Therefore, we assumed that the consolidation settlement of the foundations and post-work settlement of pile foundations were completed.

(2) We assumed that the soil layers were homogeneously stratified and isotropic within the modeling range, and that the deformations and forces in the tunnel lining were within the elastic range.

![Figure 2: (a) Graphical model and (b) finite element meshing of the inter-district tunnel and viaduct bridge piers.](image1)

![Figure 3: Shield construction model.](image2)
(3) The soil layer where the inter-district tunnels are located is a weakly permeable layer, and various measures are considered during the tunneling process to ensure that the groundwater does not percolate in large quantities into the shield’s working surface. Therefore, the influence of groundwater infiltration and flow was ignored in the finite element model.

(4) The slow consolidation (post-work settlement) and creep of the soil at the end of the shield were excluded from the model.

(5) The loads of the bridge superstructure were equivalently distributed on the top of the corresponding piers; the loads on piers P177, P178, P179, and P180 were 8,600, 11,466, 11,466, and 8,600 KN, respectively.

4. Results

As is evident from Figure 5(a), all pile foundations settle to a certain extent during the initial stage of shield excavation. The initial settlement of P179 is the largest at 0.55 mm because the excavation pressure of the tunnel boring machine disturbs the soil mass and results in stress redistribution, thereby causing the pile foundation to settle. After S3 was met, only P179 exhibited a significant vertical displacement. After S10 (when the left excavation face is 12 m from the pile foundation axis), the vertical displacement increased by 0.43 mm (from 0.55 to 0.98 mm). However, the remaining pile foundations did not exhibit an increase in vertical settlement. As seen in Figure 5(b), after S30 (when the right excavation face is 12 m from the pile foundation axis), only P178 exhibited a significant vertical displacement, which increased by 1.39 mm (from 0.23 to 1.62 mm). Upon completion of the double-line construction, the maximum and minimum vertical settlements were 1.66 and 0.16 mm, which occurred at P178 and P180, respectively. Therefore, it is concluded that the influence of shield excavation on the vertical settlement of a pile foundation is related to the distance between the excavation face and pile foundation axis; the smaller the distance, the greater the influence. The final vertical settlement values of pile foundations P177, P178, P179, and P180 were 0.17, 1.66, 1.01, and 0.16 mm, respectively.

5. Safety Assessment Based on the Bridge Status

Highway bridges must meet the requirements of the normal use limit state under external influence. Accordingly, it is necessary to evaluate the bearing capacity of their components to ensure that the calculated values do not exceed the limits in the specification. The “General Specification for Design of Highway Bridges and Culverts” [19] requires a combination of permanent load, live load, temperature load, and bearing settlement as the external loads. In addition to this specification, we used the “Public Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges Concrete and Prestressed Concrete Bridges and Culverts” [20] (Name “specification” below) as the standard for the calculation and “Bridge Doctor V3.6” for the box girder analysis and calculation. The permanent load is calculated according to the structural size, whereas the live load is calculated according to the actual use of the bridge (urban trunk road). To verify the calculations, we used the highway grade I load. The longitudinal overall impact coefficient of the vehicle load is calculated according to the “General Specification for Design of Highway Bridges and Culverts.”

Monitoring points were set on the central points of pile tops P177, P178, P179, and P180 of each pier of the West Changjiang Road Viaduct. The calculated vertical and differential settlement values for each construction stage are listed in Table 3.

According to the pile foundation settlement and bridge usage, the maximum vertical nonuniform settlement of the bridge structure caused by shield tunneling was 1.49 mm. To consider the safety reserve scenario, an adverse scenario where the bridge bearing differential settlement is 5 mm was considered to calculate the forces on the main beam. According to the “Public Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete
Table 1: Soil calculation parameters.

<table>
<thead>
<tr>
<th>Soil layer name</th>
<th>Soil thickness (m)</th>
<th>Volumetric weight, $\gamma$ (KN/m$^3$)</th>
<th>Cohesive forces, $c$ (KPa)</th>
<th>Angle of internal friction, $\phi'$ (°)</th>
<th>$E_{ref}^{s0}$ (MPa)</th>
<th>$E_{ref}^{ed}$ (MPa)</th>
<th>$E_{ref}^{ur}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Artificial fill</td>
<td>3.2</td>
<td>18.6</td>
<td>5.0</td>
<td>7.0</td>
<td>4.0</td>
<td>4.0</td>
<td>20.0</td>
</tr>
<tr>
<td>Clay</td>
<td>6.3</td>
<td>19.4</td>
<td>30.0</td>
<td>16.5</td>
<td>5.0</td>
<td>5.0</td>
<td>25.0</td>
</tr>
<tr>
<td>Silt</td>
<td>1.1</td>
<td>20.1</td>
<td>16.0</td>
<td>22.0</td>
<td>7.0</td>
<td>7.0</td>
<td>35.0</td>
</tr>
<tr>
<td>Full efflorescence argillaceous sandstone</td>
<td>1.6</td>
<td>20.9</td>
<td>30.0</td>
<td>21.0</td>
<td>10.0</td>
<td>10.0</td>
<td>50.0</td>
</tr>
<tr>
<td>Strong efflorescence argillaceous sandstone</td>
<td>36.8</td>
<td>21.0</td>
<td>90.0</td>
<td>22.0</td>
<td>15.0</td>
<td>15.0</td>
<td>75.0</td>
</tr>
</tbody>
</table>

Note. $E_{ref}^{s0}$ is the secant modulus obtained from the standard triaxial drainage test. $E_{ref}^{ed}$ is the tangent modulus of the lateral loading test. $E_{ref}^{ur}$ is the engineering strain unloading/reloading modulus. The above parameters are determined according to the survey report and engineering experiences.

Table 2: Material calculation parameters.

<table>
<thead>
<tr>
<th>Name</th>
<th>Material</th>
<th>Volumetric weight, (KN/m$^3$)</th>
<th>Elastic modulus, $E$ (MPa)</th>
<th>Poisson Ratio $\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile foundation</td>
<td>C30</td>
<td>24</td>
<td>30000</td>
<td>0.2</td>
</tr>
<tr>
<td>Bearing platform</td>
<td>C40</td>
<td>24</td>
<td>32500</td>
<td>0.2</td>
</tr>
<tr>
<td>Tunnel segments</td>
<td>C50</td>
<td>25</td>
<td>34500</td>
<td>0.2</td>
</tr>
<tr>
<td>Shield machine shell</td>
<td>Steel</td>
<td>78.5</td>
<td>210000</td>
<td>0.3</td>
</tr>
<tr>
<td>Grouting area</td>
<td>Grouting slurry</td>
<td>22</td>
<td>120</td>
<td>0.24</td>
</tr>
</tbody>
</table>

Figure 5: Vertical settlement curves for pile foundations under different working conditions. (a) Left and (b) right excavation lines.

Table 3: Calculated vertical and differential settlement for each construction stage.

<table>
<thead>
<tr>
<th>Bearing platform number</th>
<th>Double-line construction completed</th>
<th>Vertical displacement (mm)</th>
<th>Differential settlement (mm)</th>
<th>Character space, $L$ (m)</th>
<th>Differential settlement rate (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P177</td>
<td></td>
<td>0.17</td>
<td>1.49</td>
<td>30.0</td>
<td>0.0496</td>
</tr>
<tr>
<td>P178</td>
<td></td>
<td>1.66</td>
<td>0.65</td>
<td>40.0</td>
<td>0.0163</td>
</tr>
<tr>
<td>P179</td>
<td></td>
<td>1.01</td>
<td>0.85</td>
<td>30.0</td>
<td>0.0283</td>
</tr>
<tr>
<td>P180</td>
<td></td>
<td>0.16</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note. Vertical settlement is the value taken at the top surface of the central point of the bearing.
Bridge finite element unit

Table 4: Symbol definition in checking formula.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{st}$</td>
<td>Normal tensile stress of concrete at the edge of section of crack resistance checking calculation of members under the action of frequent combination</td>
</tr>
<tr>
<td>$\sigma_{pc}$</td>
<td>Normal precompression stress of concrete caused by preloading</td>
</tr>
<tr>
<td>$\sigma_{lt}$</td>
<td>Principal tensile stress in component concrete</td>
</tr>
<tr>
<td>$\sigma_{tp}$</td>
<td>Normal tensile stress of concrete caused by prestress</td>
</tr>
<tr>
<td>$\sigma_{kc}$</td>
<td>Principal compressive stress in component concrete</td>
</tr>
<tr>
<td>$f_{tk}$</td>
<td>Standard value of axial compressive strength of concrete</td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>Standard value of axial tensile strength of concrete</td>
</tr>
</tbody>
</table>

Table 5: Bending load effect of normal section.

<table>
<thead>
<tr>
<th>Section</th>
<th>Maximum bending load effect</th>
<th>Minimum bending load effect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5mm effect size (KN-m)</td>
<td>5mm effect size (KN-m)</td>
</tr>
<tr>
<td>Side span bearing section</td>
<td>19325</td>
<td>112073</td>
</tr>
<tr>
<td>Intermediate span bearing section</td>
<td>18610</td>
<td>111955</td>
</tr>
<tr>
<td>Intermediate span section</td>
<td>164969</td>
<td>184327</td>
</tr>
<tr>
<td>Side span midspan section</td>
<td>95785</td>
<td>144256</td>
</tr>
<tr>
<td></td>
<td>-61663</td>
<td>-181456</td>
</tr>
<tr>
<td></td>
<td>-62568</td>
<td>-181456</td>
</tr>
<tr>
<td></td>
<td>89360</td>
<td>184327</td>
</tr>
<tr>
<td></td>
<td>46833</td>
<td>144256</td>
</tr>
</tbody>
</table>

Note. Symbol represents positive and negative bending moment.

5.1. Bending Capacity of Cross Section Calculations under a 5-Mm Bridge Bearing Differential Settlement. As stipulated in Sections 5.2.2–5.2.5 of the specification (JTG D62-2004), the bending strength of cross-section check for the bridge should be carried out under external influence conditions. The calculation results of the maximum and minimum bending load effects of the girder when the bridge bearing differential settlement is 5 mm are shown in Table 5.

Table 5 illustrate that when bridge bearing differential settlement is 5 mm, side span bearing section maximum bending load effect is 19325 KN-m, which is less than section resistance 112073 KN-m; side span bearing section minimum bending load effect is 61663 KN-m (negative moment), which less than section resistance 181456 KN-m (negative moment), etc. The calculation results show that each section effect values less than the section resistance. Therefore, the bending capacity of cross-section calculations in the main beam at a 5 mm bridge bearing differential settlement meets the code requirements.

5.2. Crack Resistance Calculations under a 5-Mm Bridge Bearing Differential Settlement

5.2.1. Normal Sections. As stipulated in Section 6.3.1 of the specification (JTG D62-2004), the normal section crack resistance check for the bridge should be carried out under external influence conditions. According to the aforesaid specified conditions, partially prestressed concrete bridge elements are required to meet the criteria of $\sigma_{st} - \sigma_{pc} \leq 0.7f_{tk}$ and $\sigma_{lt} - \sigma_{pc} \leq 0$ for the short- and long-term effects of the load combination, respectively.

Figure 6 illustrates that the stress of the main beam is compressive stress under the short-term load effect, and the minimum stress of the upper edge is 2.809 MPa (compressive stress), the minimum stress of the lower edge is 1.108 MPa (compressive stress), and there is no tensile stress. Therefore, the crack resistance calculation of the normal section of the bridge under the short-term load effect meets the requirement of $\sigma_{st} - \sigma_{pc} \leq 0.7f_{tk}$, $0.7f_{tk} = -1.855$ MPa (tensile stress). From Figure 7 shows that the stress of the main beam under the long-term load effect is also a compressive stress. The minimum stress at the upper edge is 4.766 MPa (compressive stress), and the minimum stress at the lower edge is 2.642 MPa (compressive stress). There is no tensile stress, so the crack resistance check of the normal section of the bridge under the long-term load effect meets the requirement of $\sigma_{lt} - \sigma_{pc} \leq 0$. Therefore, the minimum stress in the main beam at a 5 mm support differential settlement meets the code requirements.
5.2.2. Oblique Sections. As stipulated in Section 6.3.1 of the specification (JTG D62-2004), the oblique sections crack resistance check for the bridge should be carried out under external influence conditions. According to the aforementioned specifications, partially prestressed concrete bridge elements are required to meet the criteria of 
\[ \sigma_{tp} \leq 0.5f_{tk} \]
for the short-term effects of the load combination.

Figure 8 illustrates that the calculated maximum principal tensile stress of the girder is \(-1.106\) MPa at midspan. Because the distortion of the calculation results at the support is not included in the statistical range, \(0.5f_{tk}\) is greater than the maximum tensile stress in the middle of the span, that is \(-1.106\) MPa. Therefore, the oblique sections crack resistance check in the main beam at a 5 mm support differential settlement meets the code requirements.

5.3. Compressive Stress Calculations under a 5 mm Bridge Bearing Differential Settlement

5.3.1. Normal Sections. As stipulated in Sections 7.1.3, 7.1.4, and 7.1.5 of the specification (JTG D62-2004), the normal sections compressive stress calculations for the bridge should be carried out under external influence conditions. For uncracked members, the concrete compressive stress in the positive section of the prestressed concrete flexural members in the service phase must satisfy the following:

\[ \sigma_{kc} + \sigma_{pt} \leq 0.5f_{ck} \]

Figure 9 illustrates that the upper and lower edges of the main beam are compressed under the standard combination. The maximum compressive stress of the upper edge is 9.998 MPa, and the maximum compressive stress of the lower edge is 11.940 MPa. The maximum compressive stress of the upper and lower edges appears at the bridge support, which meets the requirement of \(\sigma_{kc} + \sigma_{pt} \leq 0.5f_{ck}\). \(0.5f_{ck}\) is 16.2 MPa. Therefore, the normal sections compressive stress calculations in the main beam at a 5 mm support differential settlement meets the code requirements.

5.3.2. Oblique Sections. As stipulated in Section 7.1.3, 7.1.4 and 7.1.5 of the specification (JTG D62-2004), the Oblique sections Compressive stress calculations for the bridge should be carried out under external influence conditions. For the bending members of prestressed concrete in the service phase, the main compressive stress of the oblique sections must satisfy the following:

\[ \sigma_{cP} \leq 0.6f_{ck} \]

Figure 10 illustrates that the maximum compressive stress of the inclined section of the main girder under the standard combination appears at the upper edge, and the
maximum compressive stress is 11.940 MPa at the support, which meets the requirement of $\sigma_{cp} \leq 0.6f_{ck}$, $0.6f_{ck}$ is
19.44 MPa. Therefore, the normal sections compressive stress calculations in the main beam at a 5 mm support
differential settlement meets the code requirements.

Thus, based on the results presented in Sections 5.1, 5.2, and 5.3, the bending capacity, the crack resistance and compressive stress calculations for the normal and oblique sections reveal that all bearing capacity calculations meet the code specifications under a 5 mm bridge bearing differential settlement. As indicated in Table 4, shield tunneling causes a maximum differential settlement of 1.33 mm, which is less than 5 mm; hence, the viaduct can be used normally during shield tunneling.

6. Conclusion

Based on the background of shield tunnel excavation underneath a viaduct, we analyzed whether the Yangtze River West Changjiang Road Viaduct can be used normally after shield tunneling. The pile foundation model of the double-line tunnel shield crossing viaduct was established using Midas, and the vertical displacement trend of the pile foundation was analyzed. The bearing capacity of the bridge during the service stage under uneven pile foundation settlement was analyzed via the “Bridge Doctor.” The calculation results indicate that the bridge structure can be used normally under the influence of shield construction. Based on the safety assessment of the viaduct, the following conclusions can be drawn:

(1) When shield excavation is at 12 m from the pile foundation axis, vertical settlement begins to appear. When the excavation surface passes through the pile foundation axis, the vertical settlement changes the most. The shorter the distance between the pile foundation axis and the excavation surface, the greater the vertical variation. During the construction stage of the actual construction process, the displacement monitoring of the pile foundation should be strengthened.

(2) When assessing the bending capacity of the bridge, the maximum and minimum bending effect values of the bearing and the midspan section are less than the section resistance, and the bridge can be used normally.

(3) In the crack resistance calculation for the bridge normal section, the stress of the upper and lower edges is the compressive stress, with no tensile stress. In the calculation for the oblique section, except for the support distortion result, the results meet the code requirements.

(4) In the compressive stress calculation for the bridge’s normal and oblique sections, the stress results of the upper and lower edges of the main beam meet the code requirements.

(5) This study can guide practical engineering projects and provide a reference for similar engineering cases.

Data Availability

Data sharing is not applicable to this article as no new data were created or analyzed in this study.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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