Numerical simulation of shield underneath passing a river in shallow water-rich sand layer

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Abstract. In view of the situation that the tunnel of Hangzhou Metro Line 6 underneath passed the Yongjiu River, the influences of the construction of the new double-line tunnel on the seepage field and stress field of the shallow buried water-rich sand layer were analyzed, and the ground reinforcement measures to ensure the safety of shield construction was proposed. The results show that: (1) After the tunnel is excavated, the seepage field gradually returns to the original seepage field, but there is still a slight disturbance in the seepage field within 30 m below the arch bottom; (2) The maximum displacement and maximum principal stress of the tunnel model are located at the bottom of the arch; (3) It is necessary to fully consider the conditions of the river, control the settlement and cracking of the foundation, and prevent the occurrence of gushing.

1. Introduction

The development trend of urban rail transit network often makes new tunnels parallel or overlap with other tunnels, crossing rivers and bridges, etc., and these situations increase the construction difficult and safety risks extremely[1-2]. In the construction of multiple close tunnels, E. S. Liman studied the mutual influence during the construction of close parallel tunnels through numerical simulation[3]. I. Yamaguchi et al. investigated the influence of the short-distance double-layer tunnel during shield construction based on project monitoring data[4].

Due to the complex stratum structure of the river section, if the construction is not handled properly, it is extremely prone to accidents of tunnel gushing and river bed collapse. Z Eisenstein discussed the difference between the undersea tunnel and the ordinary tunnel in the construction process[5]. He summarized the construction experience of the construction of tunnels, and believed that the detection of water gushing and timely sealing of water seepage tunnel construction is the most important[6]. There are some cases of subway tunnels underneath passing rivers in China. During the construction of Beijing Metro Line 15, the tunnel excavation was continuous and stable, and there was no water leakage after completion[7]. During the construction of Shenzhen Metro Line 5, reinforced concrete was used for water-proof treatment, and jet grouting piles were reinforced below the river bed, so the shield tunneling successfully completed the construction of the underpass section[8].

According to the above research, it is most important to ensure the safety of tunnel construction and completion of the construction. However, there are very few numerical simulation studies on the tunnel which underneath passes the river. Since FLAC³D overcomes the shortcomings of the finite element method that cannot solve the large deformation problem, and can better consider the
discontinuity and large deformation characteristics of the rock and soil, this paper carried out the numerical simulation of tunnel underneath passing Yongjiu River based on FLAC3D software.

2. Numerical model and material parameters

Hangzhou Metro Line 6 starts at Shuangpu Station in Zhijiang New Town and ends at Fengbei Station at the end, and crosses under Yongjiu River. Line 6 is basically parallel to the proposed Line 4, and the horizontal distance of the line is only about 15 m. The outer diameter of the tunnel is 6.2 m, and the double-line single-circle shield method is adopted for construction.

The construction section of the line 6 tunnel underneath the Yongjiu River is about 72 m. In this paper, it is to focus on the section which cross the sandy silt layer with a minimum vertical net distance of 8.8 m between Line 6. To simplify the analysis, the longitudinal curvature changes of the Line 6 tunnel in this range was ignored. Firstly, MIDAS GTS NX is used to establish a unit grid model, and then the model import it into FLAC3D for fluid-solid coupling calculation, to realize the numerical simulation of the newly built double-line shield tunnel. The size of the model is shown in Fig.1(a). The longitudinal length of the model is 72 m, the lateral width is 100 m, and the stratum thickness is 60 m. Line 6 passes through the sandy silt layer, the groundwater level is 2.3 m above sea level, and the model strata is shown in Fig.1(b).

Shield tunnel segments and grouting circles are simulated by solid elements, and the structure and size of the tunnel of Line 4 are same as that of the Line 6. The inner diameter is 5.5 m, and the tube thickness is 0.35 m, and the width is 1.2 m. The net distance of the double-line shield is 15.98 m, the thickness of the grouting ring is 0.2 m, and the cross-section of the shield tunnel is shown in Fig.2. The geological parameters and model material parameters are shown in Table 1.

![Fig.1 Schematic diagram of calculation model](image)

![Fig.2 Schematic diagram of tunnel cross section](image)
Table 1 Geological parameters and model material parameters

| Materials         | Natural unit weight ρ/kN·m⁻³ | Elastic modulus E/MPa | Poisson ratio v | Internal friction angle θ° | Cohesion c/kPa | Permeability coefficient k/cm·s⁻¹ | Porosity n |
|-------------------|-------------------------------|-----------------------|-----------------|-----------------------------|---------------|-----------------------------------|------------|
| Plain fill        | 18.42                         | 20                    | 0.2             | 31                          | 21            | 7×10⁻⁵                            | 0.465      |
| Sandy soil        | 19.20                         | 32.6                  | 0.25            | 24                          | 2             | 8×10⁻⁴                            | 0.417      |
| Mucky silt        | 17.20                         | 12.5                  | 0.38            | 10                          | 13            | 5×10⁻⁷                            | 0.55       |
| Silty clay        | 17.50                         | 15                    | 0.3             | 11.5                        | 15            | 2×10⁺                             | 0.513      |
| Silt              | 18.50                         | 40                    | 0.28            | 27                          | 2             | 3.5×10⁻⁴                          | 0.435      |
| Round gravel      | 20.50                         | 90                    | 0.21            | 35                          | 0             | 2×10⁻¹                            | 0.257      |
| C50 segment       | 25                            | 40×10⁻³               | 0.2             | —                           | —             | —                                 | —          |
| Grouting circle   | 19.6                          | 5×10⁻⁴                | 0.2             | 32                          | 100           | 1.5×10⁻⁴                          | 0.21       |
| Shield shell      | 75                            | 2.06×10⁶              | 0.2             | —                           | —             | —                                 | —          |

3. Basic principles of fluid-solid coupling calculation

When using FLAC3D to simulate the fluid-solid coupling mechanism of a rock mass, the rock mass should be regarded as a porous medium. The flow of fluid in porous media obeys Darcy's law and satisfies the Biot equation. When using finite difference for fluid-structure coupling calculation, there are several criteria as follows.

3.1. Flow rule

The law of liquid flow is described by Darcy's law. For a uniform, isotropic and constant density fluid, the law can be expressed as follows:

\[ q_i = -k_i \hat{k}(s) \left[ p - \rho_f x_i g_i \right] \]

Where \( q_i \) is the seepage vector, \( p \) is the pore pressure, \( k \) is the inherent permeability coefficient tensor of porous media, \( \hat{k}(s) \) is the relative permeability coefficient, \( \rho_f \) is the fluid density and \( g_i \) is the gravitational acceleration vector.

3.2. Constitutive equation

Changes in volumetric strain cause changes in fluid pore pressure. Conversely, changes in pore pressure will also cause volumetric strain. The incremental form of the constitutive equation of porous media is:

\[ \Delta \sigma_{ij} + \alpha \Delta p \delta_{ij} = H_{ij} (\sigma_{ij}, \Delta \xi_j) \]

Where \( \sigma_{ij} \) is the stress increment rate, \( \Delta p \) is the increment of pore water pressure, \( \delta_{ij} \) is the Kronecker factor, \( H_{ij} \) is the given function and \( \Delta \xi_j \) is the Total strain increment.

3.3. Balance Law

For small deformations, the fluid balance can be expressed as follows:

\[ -q_{v,i} + q_v = \frac{\partial \zeta}{\partial t} \]

Where \( q_v \) is the flow intensity and \( \zeta \) is the fluid volume change per unit volume of porous media.

The change of fluid flow rate is related to the change of pore pressure, saturation, and solid volume strain. The corresponding equation is expressed as follows:

\[ \frac{1}{M} \frac{\partial p}{\partial t} + \frac{n}{s} \frac{\partial s}{\partial t} = \frac{1}{s} \frac{\partial \zeta}{\partial t} - \alpha \frac{\partial \varepsilon}{\partial t} \]
Where $M$ is the Biot modulus with the unit of N/m$^2$, $n$ is the porosity and $\alpha$ is the Biot coefficient. The momentum balance equation can be expressed as follows. Where $\rho$ is the bulk density.

$$\sigma_{ij} + \rho g_i = \rho \frac{dv_i}{dt} \quad (5)$$

4. Numerical results and discussions

4.1. The influence on formation seepage field

The influence of the construction of Line 6 on the formation seepage field is shown in Fig. 3(a)~(n). According to the calculation results, the tunnel excavation has some influence on the seepage field of the formation. When the left line of Line 6 is excavated to the middle ring, the pressure exerted by its face is greater than the pore water pressure there. At this time, the contour map presents a convex shape, and the disturbance zone is approximately within 2 m from the outer diameter of the tunnel. After the excavation of the left line of Line 6 is completed, the seepage field of the formation gradually returns to the original seepage field. Due to the large distance between the left and right lines, the disturbance of the excavated left line seepage field caused by the right line excavation is basically negligible, as sown in Fig. 3(f), (i), (j). After the construction of both the left and right lines, the seepage field gradually recovered to the original seepage field, but there was still slight disturbance in the seepage field within 30 m below the arch bottom.
(e) The initial seepage field

(f) The lateral seepage field when the right line is excavated to the middle ring

(g) The left line longitudinal initial seepage field

(h) The longitudinal seepage field of the left line when the excavation of the left line is completed

(i) The initial seepage field

(j) The lateral seepage field after excavation of the right line
Fig. 3 Contour map of pore pressure disturbance caused by construction of Line 6

4.2. The influence on tunnel displacement and principal stress
After the excavation is completed, the maximum combined displacement of the tunnel after excluding the influence of the starting point is about 14 mm (see Fig. 4), which is located at the bottom of the arch. The maximum principal stress is 3.69 MPa, which also appears at the bottom of the arch, as shown in Fig. 5.
5. Conclusion

In this paper, based on the method of numerical simulation by FLAC\textsuperscript{3D}, the fluid-solid coupling analysis of Hangzhou Metro Line 6 was carried out. The influences of the construction on the seepage field and stress field of the shallow buried water-rich sand layer were analyzed, the conclusions are obtained as below:

(1) After the tunnel is excavated, the seepage field gradually returns to the original seepage field, but there is still slight disturbance in the seepage field within 30m below the arch bottom.

(2) After construction, the maximum displacement and maximum principal stress of the tunnel are both located at the bottom of the arch. The maximum displacement is about 14 mm and the maximum principal stress is 3.69MPa. During the construction process, the arch bottom should be focus on prevent the occurrence of gushing phenomenon.

(3) Due to the great changes in geology, it is necessary to fully consider the conditions of the river must be fully considered to control the settlement and cracking of the foundation. During construction, the balance of water and soil pressure should be maintained at all times, and the quality of synchronous grouting should be strictly controlled.

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