CIVIL ENGINEERING JOURNAL

Article no. 47

THE CIVIL ENGINEERING JOURNAL 4-2022

STRATA SUBSIDENCE CHARACTERISTICS OF SHIELD TUNNELING IN COASTAL SOFT SOIL AREA

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ABSTRACT

In order to study the subsidence characteristics caused by large diameter shield tunnelling in coastal soft soil area, based on the project of North Oujiang shield tunnel, the displacement field, seepage field and stress field of surrounding rock during shield tunnel construction under fluid-solid coupling were analyzed by using finite difference method. The results show that when the shield tunnelling passes through the monitoring section of the tunnel, the surrounding rock in a certain range of this section above the tunnel will be uplifted. Shield tail grouting can effectively control the settlement of the strata, and the increase of the ground settlement decreases gradually. With the advance of the shield the pore water pressure increases, and the pore water pressure in the soil layer will rise sharply due to the shield tail grouting. When the shield passes through the monitoring section of the tunnel, the strata stress above the tunnel increases due to uplift extrusion, and the strata stress below the tunnel decreases due to stress release. When the shield tail grouting is completed and the shield machine gradually moves away from the monitoring section, the stress release leads to the decrease of the ground stress around the tunnel, and the strata stress distribution is funnel-shaped.

KEYWORDS

Coastal soft soil area, Shield tunnel, Numerical simulation, Displacement, Pore water pressure, Stress

INTRODUCTION

With the continuous development of shield technology, slurry shield technology is widely used in the construction of the submarine tunnel in soft clay strata. In the process of shield tunnelling, the disturbance to the surrounding soil layer will inevitably occur. If the disturbance is too large, it will pose a threat to the safety of the surrounding buildings or adjacent structures, and it is not conducive to the stability of the tunnel structure in the later period. Therefore, it is of great engineering significance to study the displacement field, seepage field and stress field in the surrounding rock during the shield tunnelling.

Scholars at home and abroad have studied the ground settlement of tunnel construction with different methods. Peck [1] induced and analyzed the surface settlement of tunnel construction based on the measured data of the project, and showed that the shape of settlement trough in tunnel construction of non-cohesive soil and cohesive soil layer presented error function or normal distribution curve shape. Due to its clear concept and simple calculation, this formula has been widely applied in practical engineering and has been continuously improved by some scholars [2-10]. Rowe and Kack [11] adopted the finite element method to simulate the ground settlement during shield construction, and analyzed the influence of segment weight, soil parameters, formation loss, and shield tail grouting on the ground settlement. Zhang and Huang [12] summarized stratum response caused by excavation disturbance, synchronous grouting, seepage and creep behavior of soft soil during shield tunnelling. Sagaseta [13] assumed that the soil was homogeneous, isotropic and linearly elastic incompressible material in a semi-infinite space. Combined with the Mindlin solution and the virtual mirroring technology of fluid mechanics, the analytical solution of formation displacement caused by formation loss was obtained. Liu and Zhang [14] applied the random medium theory to the prediction of ground subsidence caused by tunnel construction,and gave two-dimensional and three-dimensional analytical formulations for settlement prediction. Later some researchers analyzed the ground settlement of shield tunnel under the influence of different soil layers and construction conditions, and further developed the theory [15-19]. Chen et al. [20] combined with practical engineering used empirical formula (assuming that the settlement curve above the tunnel is a Gaussian distribution curve) and finite element analysis to conduct reverse analysis to the surface settlement of the tunnel. Based on the Mindlin solution, Liang et al. [21] considered the influence of the supporting force on the shield tunnel face, the non-uniform lateral friction between shield shell and soil in the process of shield tunnelling, the shield tail grouting pressure and other factors on the ground settlement. The vertical and horizontal displacement prediction formulas of the ground during shield tunnelling were obtained. Lv et al. [22] took the shield tunnel project of Guangzhou Metro Line 8 and studied the settlement of shield construction in the composite strata of upper soft and lower hard by using the finite element simulation and combining with the monitoring data of the project. They concluded that with the decrease of the height ratio of soft and hard rocks (soft rock height/hard rock height), the ground settlement decreased and the settlement trough became shallower. At present, the researches of scholars both domestic and foreign on the horizontal displacement of the stratum are relatively few, and the prediction results mostly reflect the settlement in the later stage of shield construction, and it cannot reflect the disturbance to the soil layer in the shield advancing process. And most of the studies do not consider the influence of water seepage in surrounding rock mass on stratum settlement.

The displacement field, seepage field and stress field of surrounding rock during shield tunnel construction under fluid-solid coupling were analyzed by using finite difference method combined with the actual project of North Oujiang tunnel of S2 railway line in Wenzhou city. This paper will provide theoretical guidance for the construction and design of similar underwater tunnel engineering.

NUMERICAL EXAMPLES

Numerical model and boundary conditions

The shield construction process at the typical section in the middle section of the river is simulated. The excavation diameter of shield machine is 14.9m, the outer diameter D of the tunnel is 14.5m, and the inner diameter d is 13.3m. The tunnel is buried 20m below the riverbed, and the water depth above the riverbed is 12m. A three-dimensional numerical model of tunnel construction is established by finite difference method. Due to the symmetry of the structure, half of the structure model is selected for calculation and analysis. In order to eliminate the influence of boundary effect, the distance between the left and right boundary and the lower boundary of the model and the tunnel axis is 3~5 times of tunnel diameter. Figure 1 shows the numerical model of shield construction and its dimensions.

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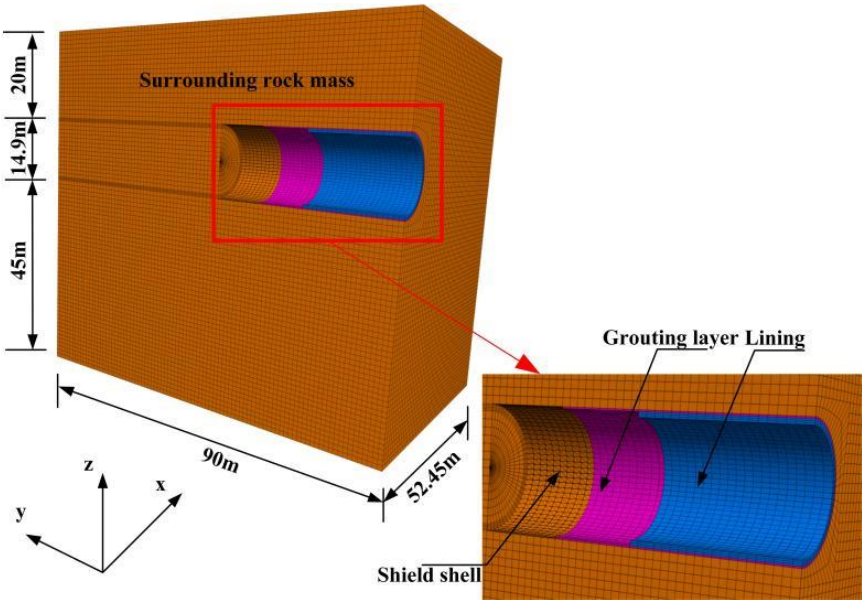


Fig. 1 – Numerical model of shield construction

The elastic constitutive models are used for the lining, grouting layer and shield of the tunnel, and the Mohr-Coulomb model is used for the surrounding rock mass, and the 8-node solid element is used for the lining, grouting layer and surrounding rock mass, and shell element is used to simulate shield shell of the shield machine. Physical parameters of surrounding rock are shown in Table 1. The selection of material parameters and elements such as shield shell, shield tail grouting body and lining are shown in Table 2. Considering the influence of staggered assembled segments and bolt connection on the stiffness of lining structure, the reduction coefficient of bending stiffness is selected as 0.8. The boundary conditions are that the top of the model is free, and the water level is above the riverbed surface depending on the specific water level. The horizontal displacement of the left and right sides and the two sides perpendicular to the axis of the tunnel are constrained, while the vertical displacement of the bottom is constrained. The sides and the bottom are set as impervious boundaries. The boundary conditions are shown in Figure 2.

Tab. 1 - Physical parameters of surrounding rock mass

The soil	Natural gravity γ/(kN/m^3)	Thickness of soil /m	Internal friction angle φ/(°)	Cohesive force c /kPa	Poisson's ratio v	Modulus of elasticity E/MPa	Porosity n	Permeability coefficient k_s/(m/s)
②^{1}Mud	16.4	14	3	6	0.4	10	0.63	7×10^{-7}
②^{2}Mud	16.7	12	3	6	0.4	10	0.6	5×10^{-7}
③^1 Silt clay	17.2	12	4	8	0.35	15	0.58	1×10^{-7}
④^{2}Clay	18.2	7	7	12	0.3	20	0.5	6×10^{-8}
⑥^{3}Tuff	25	>50	-	-	0.25	1500	0.3	2×10^{-8}

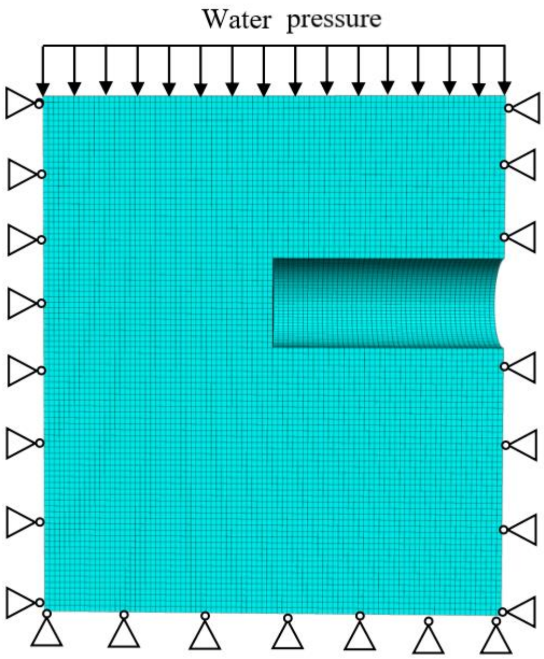


Fig. 2 – Model boundary condition

Tab. 2 - Material parameters

Material	Gravity γ/(kN/m^3)	Thickness /mm	Poisson's ratio v	Modulus of elasticity E/MPa	Unit
Lining （C50）	25	600	0.2	2.67×10^4	8-node element
Shield shell	78.5	60	0.3	2.1×10^5	Shell element
Grouting material (Beginning)	21	200	0.46	0.4	8-node element
Grouting material (Condensation)			0.2	4	

Shield construction load parameter and construction simulation

Slurry pressure in the slurry tank: The calculation results show that the support pressure at the center of the excavation face is 545.456kPa, and the change gradient of the support pressure on the excavation face is 15.537kPa/m. The grouting pressure is taken 1.1 times of the water and soil pressure, and the grouting pressure at the central axis of the tunnel is 532.434kPa, with a gradient of 16.998kPa/m. The weight of shield shell: The slurry pressure balance shield machine is selected the total weight is 2126t. The weight of the whole shield machine is evenly distributed on the whole shield shell, and the uniform weight is 71.342kPa.

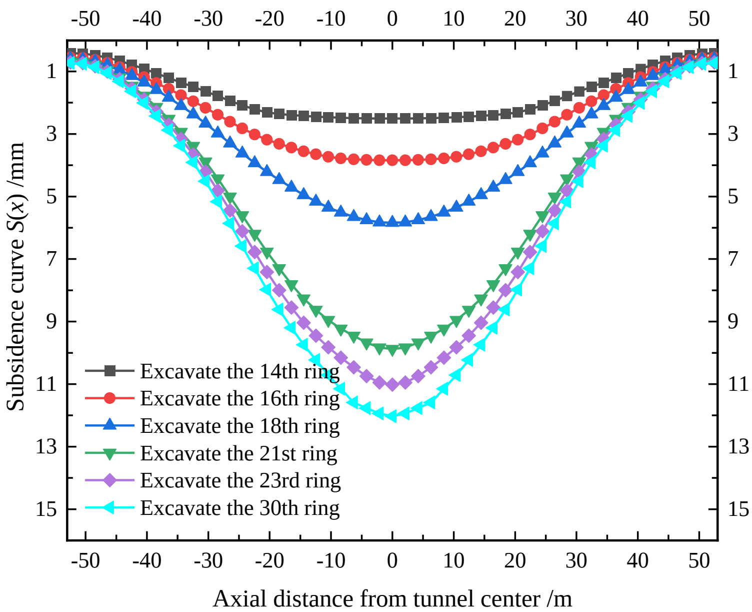
The 30 rings excavation process of shield tunnel is simulated and the length of each ring is 2m. The coupling calculation between fluid and solid is adopted to carry out mechanical calculation and seepage calculation at the same time, and the actual shield construction process is simulated and analyzed. Firstly, the in-situ stress balance is carried out for the model without considering the seepage in the surrounding rock. After tunnelling and excavating, seepage calculation and mechanical calculation are carried out simultaneously, and the fluid modulus is set as 2×10^6 kPa. The empty model is used to excavate each ring of the tunnel. The supporting pressure is set on the tunnel face, and the shell element is used to generate shield shell on the excavation ring to support the excavated soil layer. The excavation is carried out step by step. When tunnelling to the sixth ring, the shield shell element of the first ring is deleted to generate the lining of the first ring and the grouting layer of the first ring. Grouting pressure is set on the soil and lining of the upper and lower surfaces of the grouting layer of the first ring respectively to carry out fluid-soild coupling calculation. In this way, the excavation is carried out step by step to support each ring and grouting behind the ring. The grouting pressure is set behind the ring for the fluid-soild coupling calculation until the excavation of the 30th ring is completed.

CALCULATION RESULTS AND ANALYSIS OF RESULTS

The shield construction process of 30 rings of underwater tunnel excavation is numerically simulated. According to the five stages of shield tunnel construction, the displacement field, seepage field and stress field of the surrounding rock are analyzed. In order to eliminate the influence of the boundary on each field of the surrounding rock, the position of the 15th ring (Y=30m plane) is taken as the monitoring section.

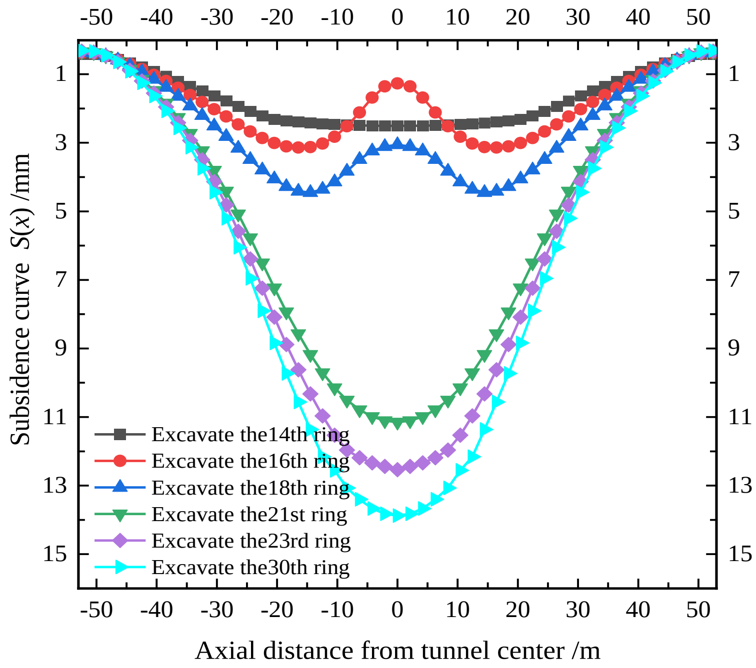
Displacement field

Taken the 15th ring section (Y=30m plane) as the monitoring cross section, the stratum settlement of monitoring section after the completion of 14th ring (shield close to monitoring section), 16th ring (shield through the monitoring section), 18th ring (transition), 21st ring (shield tail grouting stage), 23rd ring (hardening stage 1), 30th ring (hardening stage 2) are respectively selected. The formation depths of Z =0m, 10m and 18m are selected to analyze the laws of ground subsidence in the process of shield tunnelling, and the calculation results are shown in Figure 3.

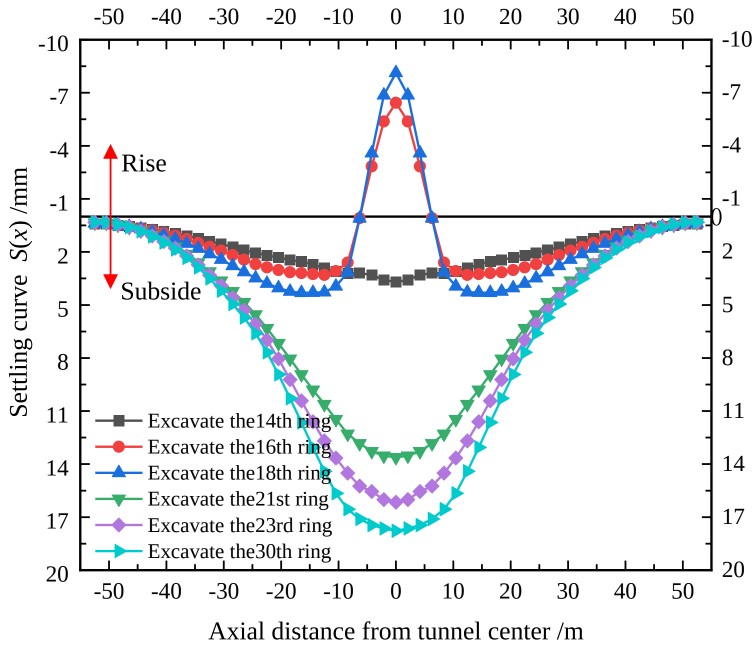


(a) Formation depth z=0m

Fig. 3 – Subsidence curve during excavation



(b) Formation depth z=10m



(c) Formation depth Z=18m plane

Fig. 3 – Subsidence curve during excavation

The Figure 3(a) shows that with shield advancing step by step, the settlement stratum (z = 0 m plane) is slowly growing, the settlement curve is basically the same in each shield stage, which all fit to Gaussian normal distribution of Peck curve, and the largest settlement occurs in the center of the tunnel axis. And after completion of the 14th ring, 16th ring, 18th ring, 21st ring, 23rd ring and 30th ring shield tunnelling, the maximum settlements on the plane z=0m are 2.6 mm, 3.8 mm, 5.8 mm, 9.9 mm, 11.0 mm and 12.3 mm respectively，which indicates that the settlement at the bottom plane of the river mainly occurs between 14th ring and 21st ring (the period of shield machine crossing and shield tail leaving the monitoring section) during shield construction, and the settlement at the bottom plane of the river is small during grouting completion and slurry consolidation, which shows selection of grouting pressure is appropriate, and strata subsidence can be controlled. Combined with Figure 3 (b) and (c), it can be seen that after the completion of the 16th and 18th rings of shield tunnelling, the settlement curve in the ground does not conform to the Gaussian normal distribution, and the curve shows the shape of gradually decreasing around and bulging in the middle, which is related to the upward floating of the machinery caused by the stress redistribution in the construction process of the shield tunnelling, so it is necessary to carry out counterweight on the shield machine in the process of shield construction to anti-floating. After the completion of shield tunnelling in the 21st, 23rd and 30th rings, the settlement curve at any depth of the stratum still conforms to the Gaussian normal distribution because the formation stress has been released due to the existence of the grouting layer. The existence of buffer layer reduces the pressure difference between the top and bottom of the tunnel, so that the floating effect will no longer occur. As can be seen from the figure, in the whole process of shield construction, with the shield gradually approaching the monitoring section, the ground will be disturbed and vertical settlement will occur. With the shield tunnelling passing through the tunnel section, A certain range of the surrounding rock above the tunnel will be uplifted, but the settlement of the riverbed will continue to increase. With the shield tail grouting gradually away from a tunnel section, the shield tail grouting can effectively control the subsidence of the ground. The subsidence of the ground is still gradually increasing, but after the condensation and hardening of the grouting layer, the subsidence of the ground will gradually tend to be stable.

Pore water pressure

The positions above the central axis of the tunnel, which are 2 m, 6 m, 10 m, 14 m, 16 m, 18 m and 20 m from the bottom of the river, are selected to study the change of pore water pressure in surrounding rock soil caused by the whole excavation process, as shown in Figure 4.

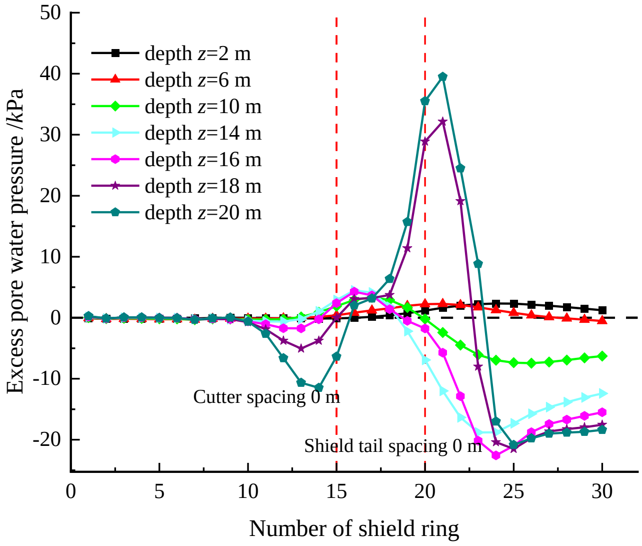


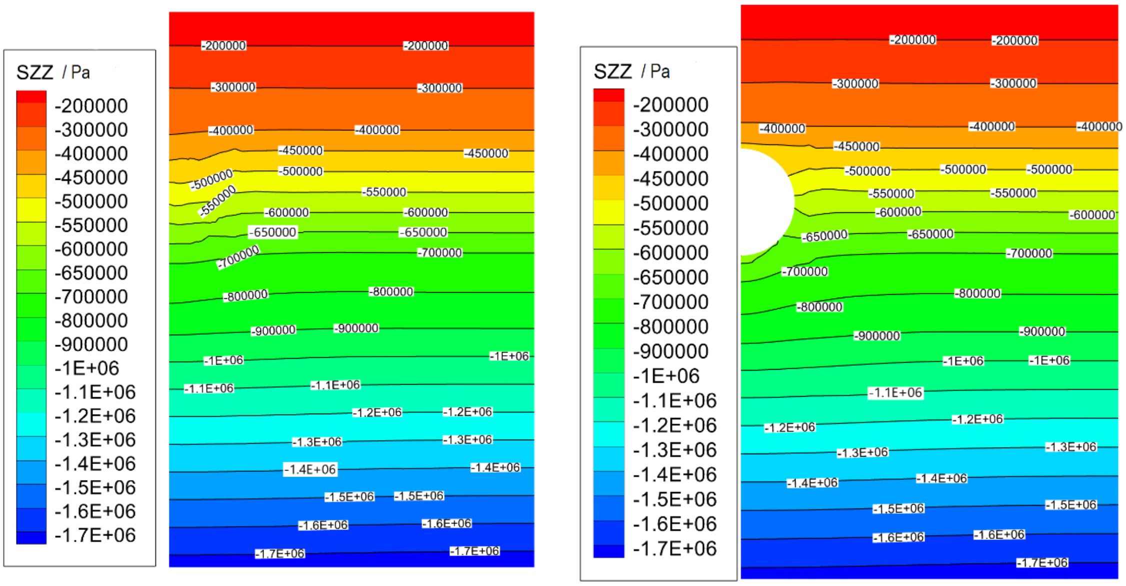
Fig. 4 – Change curve of pore water pressure during excavation

As can be seen from Figure 4, during the shield tunnelling, as the cutter head gradually approaches the tunnel excavation face, the pore water pressure in the formation will gradually decrease due to shield disturbance. When the shield passes through the tunnel face, the excess pore water pressure will be formed in the formation due to the disturbance, and the pore water pressure will increase with the shield advancing. When the grouting at the tail of the shield is removed, the pore water pressure in the soil layer within 4m of the upper part of the tunnel will rise sharply, and the maximum excess pore water pressure is 39.5kPa. As the shield tail gradually moves away, the pore pressure in the upper strata of the tunnel gradually decreases, then increases and finally tends to be stable. The final pore water pressure is less than the original hydrostatic pressure. By contrast, the pore water pressure at the top of the tunnel eventually decreases by 18.4kPa. It can be roughly seen from the figure that the influence range of pore water pressure is roughly within the range of 10m above.

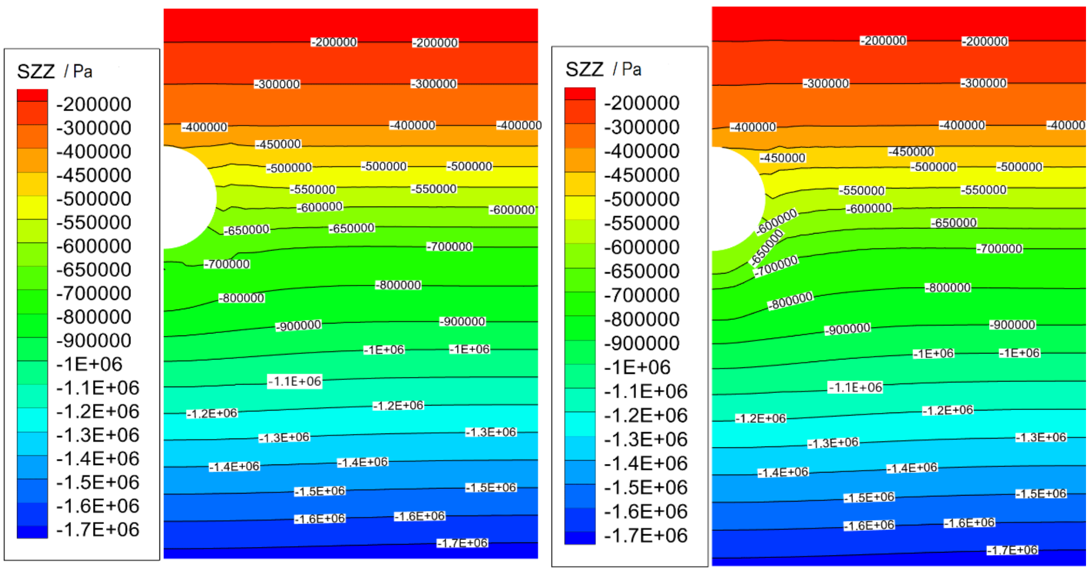
Stress field

Distribution of transverse stress field

Figure 5 is the vertical stress in the transverse direction at the position of the monitoring section (Y=30m plane) during shield construction



（a） The 14th ring *（b） The 18th ring*



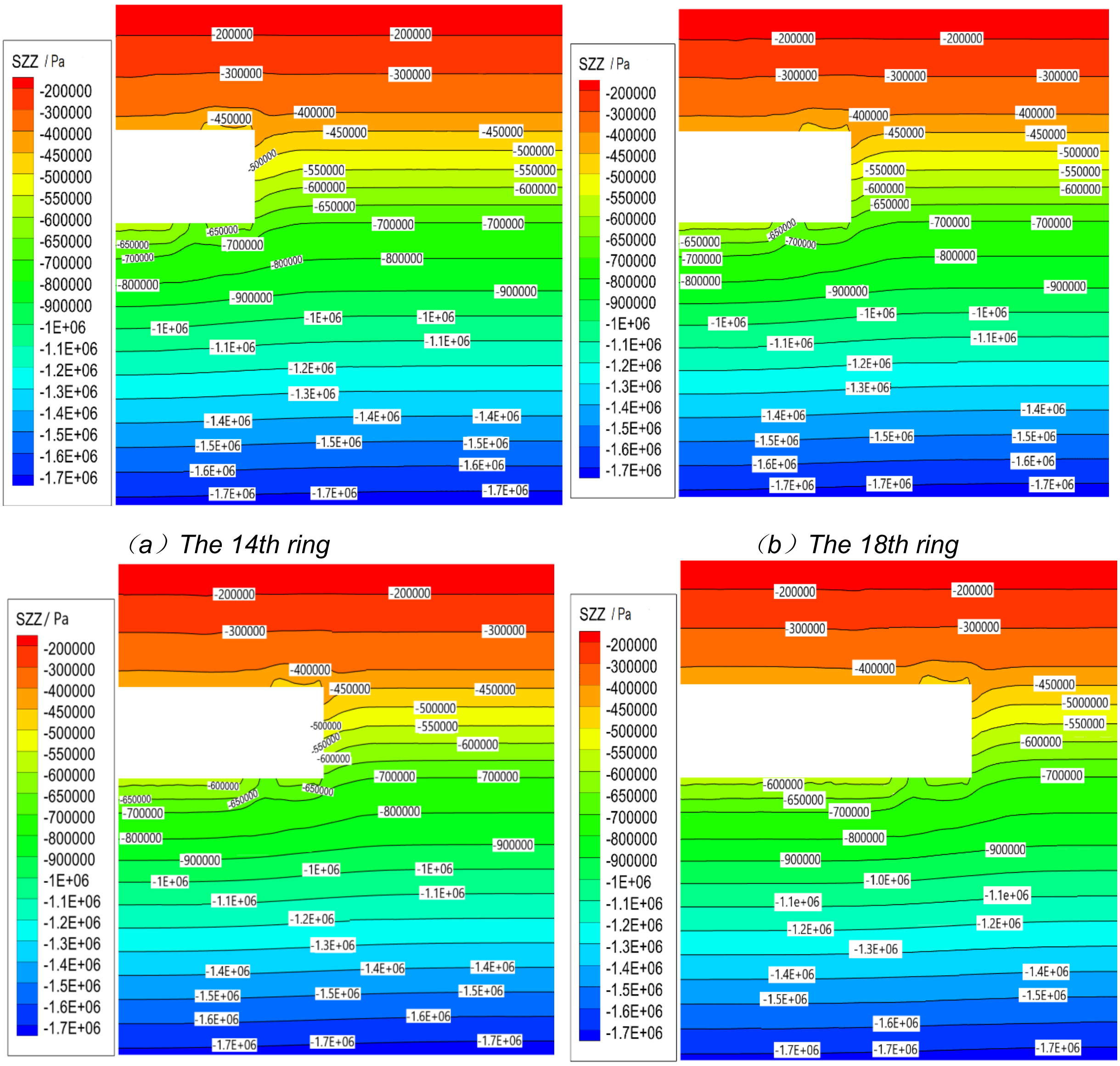
（c） The 21st ring *（d） The 30th ring*

Fig 5 – Transverse stress distribution during shield construction

According to Figure 5 (a), when the shield approaches the tunnel face (the 14th ring), the vertical stress of the stratum decreases slightly compared with the initial stress, and the shield has begun to affect the soil in front of the tunnel face. As can be seen from Figure 5 (b), when the shield passes through the tunnel face, the vertical stress of the upper soil layer increases due to the extrusion of the floating effect and presents an upward protrusion, while the bottom decreases due to the release of the stress. It can be seen from Figure 5 (c),and Figure 5(d) that when the shield tail moves away, there is a stratum loss between the strata. The existence of the gap leads decrease of the stress due to the release of the strata stress around the tunnel, and the disturbance also gradually decreases from the tunnel position to both sides, and the surrounding vertical stress presents funnel shape. Also it is seen from Figure 5 (d) that after the grouting body solidifying and hardening, the vertical stress of the formation is basically the same and tends to be stable.

Distribution of axial stress field

Figure 6 is the vertical stress on the longitudinal section where the tunnel axis is located during shield construction, and the vertical stress of the stratum in the shield construction process is analyzed.



（a）The 14th ring

（b）The 18th ring

（d）The 21st ring

（d）The 30th ring

Fig. 6 – Axial stress distribution during shield construction

The Figure 6 (a) shows as the tunnelling approaches the tunnel face, the degree of disturbance increases, and the stress release of the formation in front of the tunnel face increases, and the formation stress distribution presents a funnel shape. In Figure6 (b), the stress in the range around 8m in front of the shield face decreases due to the stress release, and the undisturbed area is still in the initial stress state. The formation stress at the shield tail gradually tends to be stable with the gradual advancing of the shield. When the shield passes through the strata, the stress of the upper soil above the tunnel increases due to the floating effect, and the stress of the bottom soil under the tunnel decreases due to the stress release. The distribution of vertical stress in Figure 6 (d) is almost the same as that in Figure 6 (c), and the formation stress tends to be stable as the shield tail gradually moves away.

CONCLUSIONS

(1) During the shield tunnelling, as the shield gradually approaching to the monitoring section, the strata will be disturbed and vertical subsidence will occur. When the shield tunnelling passes through the monitoring section, the surrounding rock soil in a certain range above tunnel uplifts, but the settlement of the strata under the riverbed will continue to increase. Shield tail grouting can effectively control the subsidence of the strata, the increasing range of the strata subsidence gradually decreases, and the strata subsidence tends to a stable state with the consolidating and hardening of the grouting layer.

(2) As the shield approaching to the tunnel monitoring section, the pore water pressure of the formation decreases gradually due to the shield disturbance. When tunnel shield passes through the monitoring section, excess pore water pressure will be formed in the stratum. With the shield advancing, pore water pressure will also increase. The pore water pressure of the soil will rise sharply when the shield tail grouting. With the shield moving away from the monitoring section, the pore pressure in the stratum above the tunnel gradually decreases, then increases and finally tends to be stable.

(3) With the shield advancing the monitoring section, the vertical stress of the stratum around the tunnel decreases due to disturbance. When the shield passes through the monitoring section, the stress in the strata above the tunnel increases due to uplift extrusion, and the soil stress under the tunnel arch bottom decreases due to stress releasing. When the shield is away from the monitoring section, the formation loss occurs around the tunnel, and the stress decreases in the surrounding stratum due to the stress release and shows a funnel-shaped distribution.

(4) For the shield tunnelling in soft soil area, it is suggested that the support force of tunnel face and grouting pressure should be set as 1.1 times the original lateral and vertical soil and water pressure respectively, which can minimize the disturbance of the stratum.

ACKNOWLEDGEMENTS

This paper is supported in part by the National Natural Science Foundation of China (Grant number: 51968045), and the Science and Technology Planning Project of Zhejiang Traffic Quality Supervision Industry(Grant number: ZJ201906).

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