Numerical analysis on the bearing mechanism of branch tunnel in upper softer and lower hard ground

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Abstract. It is of great interest to investigate the excavation disturbance mechanism of branch tunnel in the upper soft and lower hard ground. In this paper, a fine 3D numerical simulation, considering the longitudinal gradient of tunnel, was performed to investigate the bearing mechanism of branch tunnel. By comparison of the characterises of stress redistribution, two different bearing mechanisms were obtained: the beam bearing mechanism dominates the capacity of super-large section, and the beam bearing zone and stress concentration zone are identified in the arch zone: In the small interval section, the arch bearing mechanism works, and the stress loosening zone and arch bearing zone appears above the arch. The attributed reasons are related to the thickness of hard layer above the arch and the span of tunnel. Correspondingly, the arch settlement in the super-large section is much larger than that of the small interval section, while the deformation at arch foot of both sections moves outside. Furthermore, through the analysis of the maximum principal stress direction, the determination method of the outer boundary of the pressure arch was discussed.

1. Introduction
In recent years, many expressway tunnels have been built in cities. These tunnels often meet several challenges, such as the branch tunnel, large span, shallow buried and upper soft and lower hard layer. Thus, the analysis of excavation disturbance induced by tunneling plays a key role during the design and construction.

In the study of tunnel excavation disturbance, analysing the characteristics of stress redistribution have been one of the most effective means. The analytical solution of stress distribution induced by tunneling in the uniform layer had been studied based on the plane strain hypothesis, such as Fenner Formula, Caquot Formula and Bray Formula et al (Brown et al., 1983). Incorporating the Strength theory, the disturbed state of the surrounding rock can be determined (Cai et al., 2014; Chen et al., 2018). With the in-depth understanding of the secondary stress field, the pressure arch concept was widely accepted and applied. For this concept, reasonable arch axis is important and was solved under different condition (e.g., Li, 2005). For example, it is considered to be an ellipse in a super-large tunnel (Zhang et al., 2021). Moreover, all above studies were based on the stress state of the surrounding rock after excavation.
There are frequent stress conversions under the step-by-step excavation disturbance. Numerical simulation, considered to be a mean of reproducing the construction of tunnel, is widely used (Cai., 2008; Weng et al., 2010). Considering the stress redistribution, the inner boundary of the pressure arch was defined as the point where the maximum principal stress reaches its maximum value near the tunnel. The area between the inner boundary and tunnel wall was considered as plastic zone (Liang et al., 2005, Yu and Wang, 2008). However, there are many ways to determine the outer boundary of pressure arch. For example, Huang et al. (2002) and Zheng et al. (2015) considered the outer boundary of the pressure arch as the location where the maximum principle stress rotation occurs; Wang et al (2016) and Jiu et al (2021) determined the outer boundary by calculating the arching coefficient. Huang et al. (2013) regarded the region where incremental of tangential stress is greater than 0 and the incremental radial stress is less than zero as stress bearing zone; the region between the bearing zone and tunnel wall as stress is loosening zone. In summary, there is still no conclusion on the determination method of pressure arch boundary. In addition, most of the previous studies only considered the uniform layer and few reported the research on super-large tunnels.

In this paper, a FLAC3D numerical model of super-large branch tunnel in the upper soft and lower hard layer ground is established, considering the longitudinal gradient of tunnel. The bearing mechanism of surrounding rock is revealed by comparing the characteristics of stress redistribution.

2. Numerical model for branch tunnel

2.1. Engineering background
The north extension of Qiaocheng East Road, an urban expressway with many bifurcation tunnels, located in the south of Shenzhen. Especially, near the K3+695, there are five kinds of layer overlying the tunnel (see figure 1), the thickness of hard layer changes from 1.7m to 8m in the super-large span, which may affect load distribution. Besides, the maximum span of the excavation is about 32 m, which is the largest excavation span in China. The tunnel interval gradually changes from 5m in the small interval section without middle wall. Therefore, this section may become the control project of the whole tunnel.

2.2. Numerical modelling
To understand the surrounding rock mass behavior during the whole excavation process, a fine 3D numerical model with simulation area K3+545~K3+845 is established by FLAC3D (see figure 2), considering the longitudinal gradient of tunnel. The ground in the numerical model is simplified as the

Figure 1. Geological longitudinal section. Figure 2. Numerical model of branch tunnel: (a) small distance section; (b) super-large section.
same height. To eliminate the boundary effect, the size of the model is 300m(X-axis) × 300m(Y-axis) × 200m(Z-axis). The cross of the tunnel is three-center arch. The super-large span is about 32m, the mainline tunnel span is about 15m and the branched tunnel is about 9m in the small interval section. In the model, the displacement boundary conditions are applied to the lateral boundaries and the bottom boundary, the displacements along with the normal direction are restrained. The top boundary is free. This model has a total of 487343 units and 84580 nodes.

| Grounds                      | Unit Weight (kNm⁻³) | Young Modulus (MPa) | Poisson's ratio | Cohesion (kPa) | Frictional angle(°) |
|------------------------------|---------------------|---------------------|-----------------|----------------|---------------------|
| Plain fill                   | 18.70               | 7.50                | 0.38            | 20.00          | 15.00               |
| Fully weathering medium-grained granite | 19.20               | 80.00               | 0.35            | 32.00          | 28.00               |
| Massive strong weathering medium-grained granite | 23.00               | 300.00              | 0.32            | 39.00          | 35.00               |
| Earthy strong weathering medium-grained granite | 19.50               | 180.00              | 0.33            | 27.00          | 32.00               |
| Moderately weathering medium-grained granite | 26.00               | 7500.00             | 0.30            | 3400.00        | 36.00               |
| Weak weathering medium-grained granite | 26.20               | 30000.00            | 0.23            | 8000.00        | 40.00               |

The elasto-plastic model of Mohr-Coulomb criterion is used in the model. The adopted parameters for different grounds are shown in Table 1. The tunnel construction process is modelled using a three-bench three-step approach. Each excavation step corresponds to an advancement of the tunnel face of 2m. The interval between the mainline tunnel face and the branched tunnel face is 2m. Two monitoring sections, four measuring lines are set in the numerical model (see figure 2). The monitoring section 1 located in the small interval section (Y=225m) (measuring line 1 and measuring line 2), and the monitoring section 2 located in the super-large section (Y=75m) (measuring line 3 and measuring line 4). As the construction of the project has not been started, this study is only an advance study to provide reference for the construction of the project. In order to illustrate the correctness of the numerical model, the numerical modeling method used in this paper is referred to reference [9]. The ground parameters are selected strictly according to the preliminary investigation report provided by the investigation institute. Once the project starts, the verification of numerical simulation will be carried out immediately.

3. Disturbance analysis of tunnel excavation

3.1. The initial ground stress

The initial ground stress in the vertical is caused by the self-weight of the rock mass. The horizontal stress is adopted as \(\nu/(1-\nu)\) times the vertical effective stress. The model is initially consolidated to reach equilibrium and produce the in-situ stress field before excavation. The results of initial in-situ stress and theoretical calculation can be mutually verified (see figure 3).
3.2. Tunnel deformation analysis

Figure 4 presents the displacement vector after simulation completed. The length of the arrow indicates the magnitude of the displacement. In the small interval section (see figure 4a), thick hard layer and small span result in a smaller deformation. In the super-large section (see figure 4b), the thin hard layer and large tunnel span lead to a larger settlement, which has the maximum value 5.6 mm at the arch. However, the deformation trend is opposites in the arch foot.

![Figure 4. The displacement vector: (a) small interval section; (b) super-large section.](image)

3.3. The result in the small interval section

In general, tunnel excavation changes the initial stress field. As shown in figure 5, the maximum vertical stress occurs in the waist. The maximum principal stress is 5.28MPa. According to the Mohr-Coulomb criterion the uniaxial compressive strength of weak weathering medium grained granite is 34 MPa, much larger than 5.28MPa, which indicates the surrounding rock mass is in the elastic state. Horizontal stress concentration appears in the arch foot. In order to better analyze the disturbance induced by excavation, stress increment caused by excavation along the radial direction of the tunnel are described by the radial stress increment (Δσ_r) and tangential stress increment (Δσ_θ). In the arch, horizontal stress is treated as tangential stress and vertical stress is treated as radial stress. It is opposite in the waist.

![Figure 5. The stress distribution in monitoring section 1: (a) horizontal stress; (b) vertical stress](image)

In the arch (measuring line 1), as the distance to tunnel increases, Δσ_θ is less than 0 first near the tunnel and then Δσ_θ increases gradually, until to the boundary of hard layer, Δσ_θ starts to decrease abruptly. Due to the effect of excavation unloading, Δσ_r is always less than 0. According to the results of variation trend of stresses, surrounding rock mass can be divided into three parts namely a stress loosening zone (Δσ_θ<0, Δσ_r<0), an arch bearing zone (Δσ_θ>0), and an in-situ zone (Δσ_θ=0, Δσ_r=0), which are presented in figure 6a.

In the waist (measuring line 2), the measuring line located in weak weathering medium-grained grained. Δσ_θ reaches its maximum value on the tunnel wall owing to the stress concentration, and then it decreases rapidly with the increase of distance. Δσ_r is also greater than 0, which is different with the
arch. Surrounding rock mass can be divided into two parts, namely an arch bearing zone ($\Delta \sigma_\theta > 0$) and an in-situ zone ($\Delta \sigma_\theta = 0$, $\Delta \sigma_r = 0$), which are presented in figure 6b.

To sum up, stress loosening zone and the phenomenon that $\Delta \sigma_\theta$ is less than 0 have not been observed in the waist. Increased $\Delta \sigma_r$ in the waist and arch foot can be authenticated with the result of deformation.

3.4. The result in the super-large section

The results of stress redistribution in super-large section are presented in figure 7. It is clear that vertical stress concentrates in the waist and the maximum principal stress is 7.48MPa, much less than 34MPa, so the surrounding rock mass is in elastic state. As figure 7 shows, there is a significant difference (horizontal stress) between the small interval section and super-large section. In the super-large section, horizontal stress concentration is observed in the arch, which is the same as a circular tunnel (Brown et al., 1983).

Figure 6. The stress distribution in the measuring lines: (a) measuring line 1; (b) measuring line 2

Figure 7. The stress distribution in monitoring section 2: (a) horizontal stress; (b) vertical stress.

Figure 8. The stress distribution in the measuring lines: (a) measuring line 2; (b) measuring line 4.
The results obtained from the measuring line 3 and measuring line 4 in the arch and waist are described in figure 8. In the arch $\Delta \sigma_\theta$ increased first and then decreased gradually, $\Delta \sigma_r$ is always less than 0. Compared with figure 6a, $\Delta \sigma_\theta$ is always greater than 0 in figure 8a and the peak point of $\Delta \sigma_\theta$ is higher. Interestingly, the peak point located in the boundary between the upper soft layer and the lower hard layer. Surrounding rock mass can be divided into three parts, namely a beam bearing zone ($\Delta \sigma_\theta > 0$ until to $\Delta \sigma_\theta_{\text{max}}$), a stress concentration zone ($\Delta \sigma_\theta_{\text{max}}$ until to $\Delta \sigma_\theta = 0$) and an in-situ zone ($\Delta \sigma_\theta = 0$, $\Delta \sigma_r = 0$), which are presented in figure 8a.

In the waist, the distribution rules are the same as figure 6b, except the incremental of tangential stress is higher in figure 8b.

Comparing to the results in the small interval tunnel section, the large span tunnel section has the following characteristics: (1) a loosening zone happens in small interval section while it does not appear in large span tunnels; (2) the peak point of tangential is coincide with the boundary between the upper soft layer and lower hard layer.

The stress distribution results of two additional monitoring sections (Y=25m, super-large span section and Y=175m, small interval section) are presented in figure 9. For the incremental stress, in the small interval section, it can be seen that the result in Y=175 is close to Y=225. However, in the super-large section, the decrease in the thickness of hard layer results in the obvious increase of $\Delta \sigma_\theta_{\text{max}}$. Based on the surrounding rock conditions of the tunnel, the thickness of hard layer can be considered as an important factor affecting the result of stress redistribution.

![Figure 9. The stress distribution of arch: (a) small interval section; (b) super-large section.](image)

4. Bearing mechanism of surrounding rock

4.1. Arch bearing mechanism
As shown in figure 10a, pressure arch can be used to explain the characteristics of stress distribution in the small interval section. The reasonable arch axis is oval owing the vertical stress in much larger than horizontal stress. The height of arch(H) is about 21m in the mainline tunnel of small interval section. Inside the pressure arch, $\Delta \sigma_\theta$ is greater than 0. Due to the upper softer and lower hard ground, the peak point of $\Delta \sigma_\theta$ is at the ground boundary of upper soft layer and lower hard layer. Therefore, arch bearing zone can be defined as the region of increased tangential stress. Under the arch bearing zone, surrounding rock mass bear a small load which can lead to a loosening zone (see figure 10b). Outside of the arch bearing zone is in-situ zone.

4.2. Beam bearing mechanism
In the super-large section, a beam bearing mechanism is applicable in the upper softer and lower hard layer (see figure 11a). The load of upper softer layer can be simplified as uniform load applying to the beam. The thickness depends on the hard layer thickness. As the thickness decreases, the tangential stress in the top boundary of the beam increases rapidly. The tangential stress decreases with the depth. The mechanics phenomenon is same as figure 8a. The existence of bearing beam divided the arch bearing zone into two parts: beam bearing zone and stress concentration zone. Outside of the stress concentration zone is the in-suit zone (see figure 11b).
Figure 10. The bearing mechanism in small interval section: (a) force diagram; (b) zone diagram.

Figure 11. The bearing mechanism in super-large section: (a) force diagram; (b) zone diagram.

4.3. The rotation of principle stress
The direction of maximum principle stress relates to the degree of stress concentration. In a sense, the outer boundary of the pressure arch (Yu and Wang, 2008), the outer boundary where the maximum principal stress rotation angle is 90 degrees, is the point where tangential stress is small.

In the arch of small interval section, arch bearing mechanism can lead to a loosening zone formed (see figure 11a). Inside the loosening zone, the maximum principle stress direction goes from horizontal to vertical with the distance from tunnel increases (see figure 12a). Inside the arch bearing zone, the tangential stress increases and the radial decreases. Nevertheless, the maximum principal stress rotation angle is small.

In summary, the maximum principle stress direction depends on the magnitude of the tangential stress and radial stress. Therefore, the outer boundary of pressure arch should be defined as the point where $\Delta\sigma_{\theta}$ tending to zero.

Figure 12. The distribution of principal stress: (a) small interval section; (b) super-large section.

Figure 12b shows the direction variation of the principle stress at different positions of super-large section. The maximum principle stress direction is the vertical before excavation in the xz plane. The direction of maximum principal stress changes few in the waist. On the contrary, the rotation angle of
the maximum principal stress axis reaches 90° at the top of the arch. The outer boundary where the maximum principal stress rotates to 90° and the boundary between the upper softer layer and lower ground layer is coincidence, which is related to the beam bearing mechanism. The rotation angle of the maximum principal stress decreases to 0 gradually from the waist to the arch.

In the arch of super-large section, beam bearing mechanism can lead to the maximum principle direction is horizontal in the beam bearing zone. In the outer boundary of beam bearing zone, the vertical stress becomes the maximum principle stress.

5. Conclusion
In this paper, a fine 3D numerical simulation is carried out to study the bearing mechanism of surrounding rock mass in upper soft and lower hard layer. The stress redistribution characteristics of different sections are discussed. Based on these investigations, the following conclusions can be drawn:

(1) In the small interval section, arch bearing mechanism can be used to explain the characteristic stress redistribution. The stress loosening zone, arch bearing zone and in-situ zone can be identified in the surrounding rock mass, based on the characteristic of stress redistribution.

(2) In the super-large section, the beam bearing mechanism is obtained, which may be related to the thin thickness of hard layer. Therefore, beam bearing zone, stress concentration zone and in-situ zone can be identified.

(3) Based on the discussion of principal stress direction, the point where the incremental of tangential stress tending to zero can be define as the outer boundary of pressure arch.

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6. References
[1] Brown E T, Bray J W, Ladanyi B and Hoek E 1983 J. Geotech. Eng.-ASCE 109 15-39.
[2] Cai M 2008 Tunn. Undergr. Space Technol. 23 618-28.
[3] Cai M, Kaiser P K, Tasaka Y, Maejima T, Morioka H and Minami M 2004 Int. J. Rock Mech. Min. Sci. 41 833-47.
[4] Chen H H, Zhu H H, Zhang L Y 2018 Rock Mech. Rock Eng. 54 1391-410.
[5] Huang Z P, Broch E and Lu M 2002 Tunn. Undergr. Space Technol. 17 249–261.
[6] Huang F, Zhu H H, Xu Q W, Cai Y C and Zhuang X Y 2013 Tunn. Undergr. Space Technol. 35 207-218.
[7] Jiu W B, Lai J X, Qiu J L, Cao X Y, Feng Z H and Song F T 2021 Chin. J. Geotech. Eng. (Preprint).
[8] Kong X X, Liu Q S, Zhang Q B, Wu Y X and Zhao J. Tunn. Undergr. Space Technol. 71 382-90.
[9] Liu X S and Wang Y 2010 Tunn. Undergr. Space Technol. 28 447-55
[10] Liang X D Liu Gang and Zhao J 2005 J. Hohai Univ. (Nat Sci.) 03 317-7
[11] Li L K 2015 structural mechanics vol 5, ed Ge X (Beijing: Higher education press) chapter 4 pp 56-65.
[12] Weng M C, Tsai L S, Liao C Y and Jeng F S 2010 Tunn. Undergr. Space Technol. 25 397-406.
[13] Wang S R, Chun L L, Wang Y G and Zou Z S 2016 Tehnicki Vjesnik 23 181-9.
[14] Wu X, Jiang Y, Guan Z and Gong B 2019 Tunn. Undergr. Space Technol. 83 135-144.
[15] Yu B and Wang H J 2008 Theory of pressure arch and division of buried depth of tunnel vol 1, ed Shi B (Beijing: China railway publishing house) chapter 3 pp 27.
[16] Zheng K C, Ding W Q, Jin W and Luo Y 2015 Chin. J. Geotech. Eng.,2015,3772-77(in Chinese).
[17] Zhang M Q, Lu G, Liu J Y, Luo D H and Sun G Q 2021 Construction technology of Badaling underground station Beijing Zhangjiakou high speed railway vol 1, ed Li N (Beijing: China communications press) chapter 4 pp 47-50.