Study on Mechanical Properties of Surrounding Rock in TBM Tunnel Construction Considering Excavation Effect

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Abstract

The mechanical properties of surrounding rock of TBM tunnel are gradually changed during excavation. The stress of surrounding rock is divided into three stages: primary stress, secondary stress and third stress. The analytical method is used to analyze the stress state of each stage. The evolution law of spatial effect of excavation surface is obtained. Then the displacement of surrounding rock considering the spatial effect of excavation surface is studied. The stress and strain development process of surrounding rock is analyzed with an engineering example. The research results provide theoretical guidance for the construction of TBM tunnel in China.

Keywords

TBM Tunnel; Mechanical Properties; Excavation Effect; Surrounding Rock.

1. Introduction

At present, with the rapid development of tunnel construction, the development law of surrounding rock deformation caused by excavation unloading effect has become a research hotspot. In Ref. [1], based on the unified strength theory, the elastoplastic theory of the surrounding rock of TBM inclined shaft is solved, and the analytical solution of the plastic zone radius, surrounding rock stress and displacement of the surrounding rock in the two-way non-isostatic stress field is derived. The stress field and seepage field coupling of the surrounding rock of TBM inclined shaft are analyzed, and the elastoplastic mechanical analysis model of the inclined section is established. In Ref. [2], the numerical simulation is used to analyze the deformation of the surrounding rock of the TBM construction card machine accident, and the tunnel is presented as a whole. The Ref. [3] relies on the introduction of the TBM construction process in the 1415A roadway of Zhangji Mine. The geological conditions of the surrounding rock of the roadway are analyzed and monitored, and the support conditions are numerically simulated to obtain the internal force of various support schemes. In Ref. [4], through digital photographic measurement technology and transparent rock experiment technology, the deformation law of tunnel cross section and longitudinal section surrounding rock is obtained, and the time-space evolution law of TBM tunnel deformation in deep composite stratum is revealed. However, the effect of the spatial effect of the excavation face is not considered. In Ref. [5], through the numerical simulation of the tunnel excavation and support process, the influence of the spatial effect of the excavation surface on the initial stress release of the surrounding rock during the construction process and the variation law of the surrounding rock-support interaction are studied.

2. Engineering Geological Conditions

The main and auxiliary inclined shafts of Well No. 1 in the Taigemiao mining area of Xinjie Mine are all inclined from the surface at an angle of s, with the length of 6314m, the excavation diameter of Tunnel D is 7.6m, and the relative elevation difference between the inclined wellhead and the bottom is 660m. Open excavation method shall be adopted within 205m of the inclined shaft head, and the length of TBM construction section is 6109m. In order to study the relationship between stress performance and buried depth of surrounding rock, four types of buried depths of 300m, 400m, 500m and 600m below the surface are taken in this paper. See Table 1 for specific data of main parameters of four geology along the line [6-9].

Tab 1. The selected values of geological parameters

Rock formation	name	Bottom depth / m	Elastic E/GPa	Poisson's μ	Cohesion c /MPa	Internal / degree
2	K_{1zh}	386	7.3	0.21	2.6	27
3	J_{2a}	506	9.6	0.2	2.9	27
4	J_{2z}	649	10.8	0.2	3.2	27
5	J_{1-2y}	699	8.5	0.17	3.3	27

The single-shield tunneling machine is mostly used in soft rock and broken formation, and the TBM(shield)construction section of the Project is excavated by the tunneling machine. Diameter of down-hole is 7.m, length of shield main machine is L=10m. A section of inclined shaft with a longitudinal length of 59.5m is taken as the scope of study. AA plane and FF plane are the left and right boundaries of the research scope. The shield machine is advanced from GG position, with a 1.5m footage to the left, and it is advanced to the AA surface at the left end through 33 excavation steps. The observation surface CC of displacement and stress of national rock is 30m away from the left end boundary, and the 13th step is excavated to this surface.

The rock mass is in the initial stress state (i.e. the original rock stress state)before the excavation of the court course, which is called the primary stress state. In general, the initial stress state of solid rock is balanced. After TBM excavation, the initial stress balance state of national rock stress is broken, and the stress size and direction will be readjusted until the balance is formed again,the stress state at this time is called the secondary stress state. The secondary stress state is greatly influenced by the excavation mode, excavation speed and other factors. If the secondary stress state satisfies the stability requirements, no supporting measures are required to maintain self-stability. On the contrary, proper supporting measures must be taken for control to stabilize the surrounding rock of the tunnel. After the construction of the supporting structure, the surrounding rock and the support form the whole of common deformation. If the two interact, the secondary stress state is changed again, which requires the second time to reach the new stress balance state, which is called the tertiary stress state. If the three stress states meet the stability requirements, a stable suspicious track system will be formed and put into production. Otherwise, support shall be continued or the support type or supporting parameters shall be changed to achieve stability.

3. Initial Stress State of Surrounding Rock of TBM Construction Inclined Shaft

The force model of the surrounding rock of the inclined shaft is shown in Fig.1.

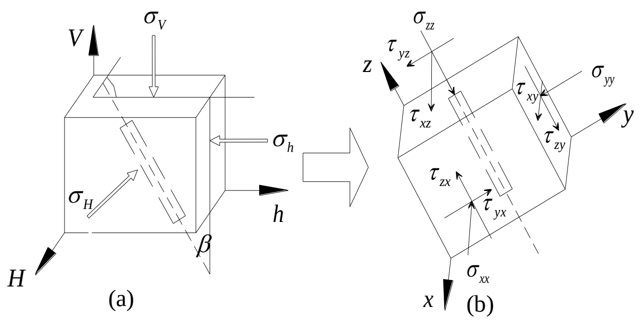

Fig 1. Coordinate conversion of surrounding rock stress ofinclined shaft

In Fig. 1(a), the angle between the axis of the inclined well and the vertical direction is β, the angle between the azimuth of the well and the horizontal maximum stress is α, $\sigma_V = p_0$ is the vertical stress(p_0 is the original rock stress), σ_H is the maximum horizontal stress and σ_h is the minimum horizontal principal stress.

Because the principal stress direction and the well axis direction are inconsistent, the stress distribution model of the surrounding rock is more complicated. In order to simplify the stress state of the surrounding rock, based on the coordinate transformation theory of elastic mechanics, the z-axis of the coordinate system is parallel to the well axis. As shown in Fig. 1(b), the stress expression is:

$$\begin{cases}\sigma_{xx} = \left(\sigma_H \cos^2\alpha + \sigma_h \sin^2\alpha\right)\cos^2\beta + \sigma_V \sin^2\beta \\ \sigma_{yy} = \sigma_H \sin^2\alpha + \sigma_h \cos^2\alpha \\ \sigma_{zz} = \left(\sigma_H \cos^2\alpha + \sigma_h \sin^2\alpha\right)\sin^2\beta + \sigma_V \cos^2\beta \\ \tau_{xy} = \left(-\sigma_H + \sigma_h\right)\cos\beta\cos\alpha\sin\alpha \\ \tau_{xz} = \left(\sigma_H \cos^2\alpha + \sigma_h \sin^2\alpha - \sigma_V\right)\cos\beta\sin\beta \\ \tau_{yz} = \left(-\sigma_H + \sigma_h\right)\sin\beta\cos\alpha\sin\alpha\end{cases} \tag{1}$$

The six stress parameters in equation (1) are shown in Fig. 1(b). Since hypothesis $\sigma_H = \sigma_h \neq \sigma_V$,equation (1) can be simplified to：

$$\begin{cases}\sigma_{xx} = \sigma_H \cos^2\beta + \sigma_V \sin^2\beta \\ \sigma_{yy} = \sigma_H \\ \sigma_{zz} = \sigma_H \sin^2\beta + \sigma_V \cos^2\beta \\ \tau_{xy} = 0 \\ \tau_{xz} = \left(\sigma_H - \sigma_V\right)\cos\beta\sin\beta \\ \tau_{yz} = 0\end{cases} \tag{2}$$

At this time, the roadway can be regarded as a thick-walled cylindrical structure under the action of three-way equal stress. Then, under the condition of uniform stress field, the formula (2) can be simplified as:

$$\begin{cases}\sigma_{xx}=\sigma_{yy}=\sigma_{zz}=p_0\\ \tau_{xy}=\tau_{xz}=\tau_{yz}=0\end{cases} \tag{3}$$

4. Study on Elastoplastic Secondary Stress State of Surrounding Rock of TBM Construction Inclined Shaft

4.1 Stress State in the Plastic Zone

When the secondary stress of the cave wall exceeds the yield stress of the rock mass, the rock mass of the cave wall will produce a plastic zone. In terms of the mechanical properties of rock, most rocks are brittle materials and the magnitude of the yield stress is not easy to find. Therefore, the Mohr-Coulomb strength criterion can be approximated as a criterion for entering the plastic state.

$$\sigma_1=\xi\sigma_r+\sigma_c \tag{4}$$

Where σ_1 -maximum principal stress; σ_3 -minimum principal stress; σ_c -theoretical uniaxial compressive strength value, which can be obtained by $2c\cdot\cos\varphi(1+\sin\varphi)$; ξ -the slope of the intensity line can be obtained by $(1+\sin\varphi)(1-\sin\varphi)$. When $\lambda=1$, it can be considered that the tangential stress σ_θ is the maximum principal stress, and the radial stress σ_r is the minimum principal stress. Therefore, the condition that the rock mass enters the plastic state can be rewritten as follows:

$$\sigma_1=\xi\sigma_r+\sigma_c \tag{5}$$

According to the principle of elastic mechanics, the static equilibrium equation and the geometric equation can be kept unchanged except that the stress in the plastic zone satisfies the plasticity criterion. The static equilibrium equation is

$$\sigma_{\theta p}=\frac{\mathrm{d}(r\sigma_{rp})}{\mathrm{d}r} \tag{6}$$

In order to distinguish from the stress in the plastic zone, the above formula expresses the stress symbol on the angle mark by adding p , which is expressed as the stress of the plastic zone. Substituting the plasticity criterion of the rock mass into the above formula,

$$\sigma_{\theta p}=\frac{\sigma_{\theta p}-\sigma_c}{\xi}+\frac{r\,\mathrm{d}\sigma_{\theta p}}{\xi\,\mathrm{d}r} \tag{7}$$

$$\frac{\mathrm{d}\sigma_{\theta p}}{\mathrm{d}r}-\frac{\xi-1}{r}\sigma_{\theta p}=\frac{\sigma_c}{r} \tag{8}$$

Solving the differential equation

$$\left.\begin{aligned}\sigma_{\theta p}&=e^{\int\frac{\xi-1}{r}\mathrm{d}r}\left[\int e^{-\int\frac{\xi-1}{r}dr}\frac{\sigma_c}{r}\mathrm{d}r+C\right]=\frac{\sigma_c}{1-\xi}+Cr^{\xi-1}\\ \sigma_{rp}&=\frac{1}{\xi}\left[\frac{\xi\sigma_c}{1-\xi}+Cr^{\xi-1}\right]\end{aligned}\right\} \tag{9}$$

From the boundary condition $r=a,\sigma_{rp}=0$, the integral constant $C=\frac{\xi\sigma_c}{\xi-1}\left(\frac{1}{a}\right)^{\xi-1}$ is obtained. Substituting C into equation (9) yields the stress in the plastic zone:

$$\left.\begin{aligned}\sigma_{\theta p} &= \frac{\sigma_c}{1-\xi}\left[\xi\left(\frac{r}{a}\right)^{\xi-1}\right]-1 \\ \sigma_{rp} &= \frac{\sigma_c}{\xi-1}\left[\left(\frac{r}{a}\right)^{\xi-1}-1\right]\end{aligned}\right\} \tag{10}$$

4.2 Plastic Zone Radius

According to the condition $r = R_p$, $\sigma_{\theta p} = \sigma_{\theta e}$, $\sigma_{rp} = \sigma_{re}$, the stress in the plastic zone at $\lambda = 1$ is substituted into the condition that the elastic stress is satisfied when $\lambda = 1$: $\sigma_{\theta p} + \sigma_{rp} = 2p_0$, after a series of simplified calculations

$$R_p = a\left[\frac{2p_0(\xi-1)+2\sigma_c}{\sigma_c(1+\xi)}\right]^{\frac{1}{\xi-1}} \tag{11}$$

It can be seen from equation (11) that the radius of the plastic zone is not only related to the strength of the rock mass itself, but also affected by the initial stress p_0 and the cavity radius a.

By substituting the formula (11) into the calculation formula (10) of the stress in the plastic zone, the stress on the boundary of the plastic ring can be obtained.

$$\left.\begin{aligned}\sigma_{rp} &= \frac{1}{\xi+1}(2p_0-\sigma_c) \\ \sigma_{\theta p} &= \frac{1}{\xi+1}(2\xi p_0+\sigma_c)\end{aligned}\right\} \tag{12}$$

5. Study on Elastoplastic Stress State of Surrounding Rock in TBM Construction Inclined Shaft - Science Definition

After the trench chamber is excavated, the support structure is applied in time, and the rock and support around the cave will form a common deformation. The deformation of the surrounding rock to the inside is resisted by the supporting structure, thereby generating elastic resistance. At this time, the surrounding rock enters a state of three stresses.

Assume that the elastic resistance at any point on the surrounding rock-support contact surface is

$$\left.\begin{aligned}\sigma_{ra} &= S_0 + S_n\cos 2\theta \\ \sigma_{r\theta a} &= 0\end{aligned}\right\} \tag{13}$$

Where S_0—uniform resistance, constant; S_n—The maximum amplitude of a change in resistance, constant.

Since the excavation effect is not infinite, the stress extreme condition is

$$\left.\begin{aligned}\sigma_r\big|_{r\to\infty} &= 0 \\ \sigma_\theta\big|_{r\to\infty} &= 0 \\ \sigma_{r\theta}\big|_{r\to\infty} &= 0\end{aligned}\right\} \tag{14}$$

The stress boundary condition of the hole is

$$\left.\begin{aligned}\sigma_r\big|_{r=a} &= \sigma_{ra} \\ \sigma_{r\theta}\big|_{r=a} &= \tau_{r\theta a}\end{aligned}\right\} \tag{15}$$

The general expression of the excavation disturbance stress component is

$$\left.\begin{aligned}\sigma_r &= \frac{A}{r^2} + 2B + (-2C - 6Er^{-4} - 4Fr^{-2})\cos 2\theta \\ \sigma_\theta &= -\frac{A}{r^2} + 2B + (2C + 12Dr^2 + 6Er^{-4})\cos 2\theta \\ \tau_{r\theta} &= (2C + 6Dr^2 - 6Er^{-4} - 2Fr^{-2})\sin 2\theta\end{aligned}\right\} \tag{16}$$

Substituting the formula (16), (13) into (15), you can get

$$A = a^2 S_0,\ F = -\frac{a^2}{2} S_n,\ E = \frac{a^2}{6} S_n \tag{17}$$

From equations (17) and (16), the additional stress field of the surrounding rock generated by the elastic resistance can be obtained as

$$\left.\begin{aligned}\sigma_r &= S_0(\frac{a}{r})^2 - S_n[(\frac{a}{r})^4 - 2(\frac{a}{r})^2]\cos 2\theta \\ \sigma_\theta &= -S_0(\frac{a}{r})^2 + S_n(\frac{a}{r})^4 \cos 2\theta \\ \sigma_{r\theta} &= -S_n[(\frac{a}{r})^4 - (\frac{a}{r})^2]\sin 2\theta\end{aligned}\right\} \tag{18}$$

Similarly, the displacement of surrounding rock caused by elastic resistance can be expressed as

$$\left.\begin{aligned}u_r &= \frac{(1+\mu)a}{E}\left\{-S_0(\frac{a}{r}) + \frac{1}{3}S_n\left[(\frac{a}{r})^3 - 6(1-\mu)(\frac{a}{r})\right]\cos 2\theta\right\} \\ u_\theta &= \frac{(1+\mu)a}{E}S_n\left[\frac{1}{3}(\frac{a}{r})^3 + (1-2\mu)(\frac{a}{r})\right]\sin 2\theta\end{aligned}\right\} \tag{19}$$

Displacement caused by the elastic resistance around the circular cavity is

$$\left.\begin{aligned}u_r^a &= -\frac{(1+\mu)a}{E}\left\{S_0 + \frac{1}{3}(5-6\mu)S_n \cos 2\theta\right\} \\ u_\theta &= \frac{2}{3}\frac{(1+\mu)(2-3\mu)a}{E}S_n \sin 2\theta\end{aligned}\right\} \tag{20}$$

By superimposing the stress component formula (18) generated by the elastic resistance obtained in this section and the previously calculated secondary stress state, the cubic stress state of the surrounding rock can be obtained.

$$\left.\begin{aligned}
&\sigma_r(r,\theta)=\frac{1}{2}(\sigma_z+\sigma_x)(1-\frac{a^2}{r^2})+S_0(\frac{a}{r})^2-\left[\frac{1}{2}(\sigma_z-\sigma_x)\left(1+3\frac{a^4}{r^4}-4\frac{a^2}{r^2}\right)-S_n\left(\frac{a^4}{r^4}-2\frac{a^2}{r^2}\right)\right]\cos 2\theta\\
&\sigma_\theta(r,\theta)=\frac{1}{2}(\sigma_z+\sigma_x)(1-\frac{a^2}{r^2})+S_0(\frac{a}{r})^2+\left[\frac{1}{2}(\sigma_z-\sigma_x)\left(1+3\frac{a^4}{r^4}\right)-S_n\left(\frac{a^4}{r^4}\right)\right]\cos 2\theta\\
&\sigma_{r\theta}(r,\theta)=\left[\frac{1}{2}(\sigma_z-\sigma_x)\left(1+2\frac{a^2}{r^2}-3\frac{a^4}{r^4}\right)-S_n\left(\frac{a^4}{r^4}-\frac{a^2}{r^2}\right)\right]\sin 2\theta
\end{aligned}\right\} \tag{21}$$

Superimposing equation (19) and equation (20) to obtain the three displacements of the surrounding rock.

$$\left.\begin{aligned}
&u_r(r,\theta)=\frac{(1+\mu)a}{E}\left\{\begin{aligned}&\left[\frac{1}{2}(\sigma_z+\sigma_x)-S_0\right]\frac{a}{r}-\frac{1}{2}(\sigma_z-\sigma_x)\left[(1-\mu)\frac{4a}{r}-\frac{a^3}{r^3}\right]\cos 2\theta+\\&\frac{1}{3}S_n\left[\frac{a^3}{r^3}-6(1-\mu)(\frac{a}{r})\right]\cos 2\theta\end{aligned}\right\}\\
&u_\theta(r,\theta)=\frac{(1+\mu)a}{E}\left\{\frac{1}{2}(\sigma_z-\sigma_x)\left[(1-\mu)\frac{2a}{r}-\frac{a^3}{r^3}\right]+S_n\left[\frac{a^3}{3r^3}+(1-2\mu)(\frac{a}{r})\right]\right\}\sin 2\theta
\end{aligned}\right\} \tag{22}$$

The displacement coordination condition expresses the relationship between the deformation of the surrounding rock and the deformation of the supporting structure during the entire construction process of the tunnel. The radial displacement and tangential displacement corresponding to the release load in the initial ground stress are

$$\left.\begin{aligned}
u_a&=\frac{(1+\mu)a}{E}(3-4\mu)\frac{\sigma_z-\sigma_x}{2}\cos 2\theta\\
v_a&=-\frac{(1+\mu)a}{E}(3-4\mu)\frac{\sigma_z-\sigma_x}{2}\sin 2\theta
\end{aligned}\right\} \tag{23}$$

Assuming that the radial force between the support and the surrounding rock is $S_n\cos 2\theta$, and the tangential interaction force is $S_t\sin 2\theta$, the radial interaction force and the tangential interaction force can be obtained by the structural mechanics method. The formula for calculating the deformation and internal force of each section of the supporting structure under the joint action is

$$\left.\begin{aligned}
u_1&=\frac{a^4}{18E_1I}(2S_n+S_t)\cos 2\theta\\
v_1&=-\frac{a^4}{18E_1I}(2S_n+S_t)\sin 2\theta\\
N&=-\frac{a}{3}(S_n+2S_t)\cos 2\theta\\
M&=\frac{a^2}{6}(2S_n+S_t)\cos 2\theta
\end{aligned}\right\} \tag{24}$$

In the formula, the radial displacement, tangential displacement, elastic modulus and moment of inertia of the u_1, v_1, E_1, I --support structure. If the tangential resistance is not counted, that is $S_t = 0$, according to the continuous deformation condition between the surrounding rock and the supporting structure, and the equations (23) and (24) are reused, the elastic resistance coefficient is obtained.

$$S_n = \frac{3(3-4\mu)(\sigma_z - \sigma_x)}{2(5-6\mu+4Q_2)} \tag{25}$$

In the middle,there is $Q_2 = \dfrac{a^3 E}{12(1+\mu)E_1 I}$.

6. Conclusion

In this chapter, the displacement and stress changes of tunnel surrounding rock during TBM excavation are theoretically analyzed. The initial stress state, secondary stress state and cubic stress state of surrounding rock are studied. The radial displacement and stress variation of surrounding rock were studied and the following conclusions were obtained:

(1) Due to the spatial effect of the excavation surface, although the surrounding rock in front of the face of the face is not excavated, it has a certain advance deformation, and the influence range is about double the diameter of the hole. The stress of the surrounding rock at the position of the face is drastically reduced.

(2) The degree of deformation first is significantly affected by the strength of the surrounding rock, and the lower the strength of the surrounding rock, the first deformation develops,the more obvious.

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References

[1] Zhao Zhenyan, Wang Weili. Rapid construction technology and practice of large section inclined shaft [J]. Coal Mine Safety, 2010, (6): 60-65.

[2] Soviet army. Theory and Technology of surrounding Rock Stability Control in double Shield TBM Construction of Deep buried soft Rock Tunnel [D]. Wuhan: Wuhan University, 2010.

[3] Yang Yue. Study on Mechanical characteristics of inclined Shaft Segment and Anchor and shotcrete lining structure in TBM (Shield Machine) Construction [D]. Beijing: China University of Mining and Technology (Beijing), 2015.

[4] Yukinori Koyama. Present status and technology of shield tunneling method in Japan[J].Tunneling Underground Space Technology,2003,18(2):145-159.

[5] Zheng Yutian. The theoretical basis of elastic-plastic viscosity in rock mechanics [M]. Beijing: coal Industry Press, 1988.

[6] Deng Jian, Zhu Hehua. Finite element Monte-Carlo method using neural networks for geotechnical reliability analysis[J]. Journal of TONGJI University, 2002, 30(3):269-273.

[7] Zuo Yulong, Zhu Hehua, LI Xiaojun. An ANN-based four order moments method for geotechnical engineering reliability analysis[J]. Rock and Soil Mechanics, 2013,34(2):513-519.

[8] ZhouU Kaili, Kang Yaohong. Neural network model and progress design of MATLAB simulation[M].Beijing: Qinghua University Press,2005.

[9] Zhang Dongqing, Zhao Dongxu, Sun Qingxu. Program calculation method of reliability index of tunnel lining based on ANSYS[J]. Journal of Shijiazhuang hang Yingren.2003,22(3):395-399.