Research Article

Analysis of Crustal Stress and Its Influence on the Stability of the Deep Tunnel in the Huanaote Mining Area

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Received 7 December 2021; Revised 8 January 2022; Accepted 10 January 2022; Published 1 March 2022

Academic Editor: Lijie Guo

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Crustal stress is a critical parameter utilized to analyze the stability of the tunnel in underground hard-rock mining sites. In order to analyze the distribution law of crustal stress and its influence on the stability of the deep tunnel in the Huanaote mining area, the in-situ stress field model of this area was constructed firstly by using the borehole stress relief method. Moreover, the numerical model is established based on three real engineering conditions: the excavation direction is parallel, vertical, and intersection to the measured maximum horizontal principal stress direction, respectively. Results show that, compared with the other two layout schemes, the stress of two side walls reaches peak at 27 MPa, when the excavation direction is intersected to the measured maximum horizontal principal stress direction, which indicates that the support and maintenance of two side walls should be strengthened to ensure the stability of the tunnel and the safety of personnel and equipment in real projects.

1. Introduction

The Huanaote mining area is located in the middle of the Inner Mongolian Plateau. The lowest mining depth is more than 800 meters from the surface, which is the deepest underground hard-rock mining site in Inner Mongolia. The production began at 2020, and varying degrees of rock burst occurred during the development stage between +100 m and +80 m levels. In order to provide scientific reference for the layout and design of stopes and tunnels, in-situ stress measurement must be carried out to obtain the three-dimensional stress state of the mining area [1, 2].

At present, the borehole stress relief method is the most widely used and the most mature technology in the world. The basic data such as initial strain of stress relief are collected on-site, and it will be combined with indoor temperature compensation test, confining pressure calibration, and related rock mechanics test, and the overall distribution of in-situ stress in the mining area will be determined [3–9]. However, the acquisition circuit of the conventional hollow

inclusion strain gauge is a conductor, which is easily influenced by temperature change. The Australian ES&S company has launched the HID type of hollow inclusion probe to realize in-situ digitization of data, but there is no temperature calibration algorithm to correct the temperature inaccuracy. Due to the function of the cooling water channel of the drilling rig and the influence of the friction torque in the drill pipe, there is a risk of being twisted when the measuring wire is removed [10–13].

“Double temperature compensation and the in-situ stress relief measurement considering the nonlinearity of rock mass” [14, 15] are carried out firstly to accurately analyze the distribution law of crustal stress in the Huanaote mining area and its influence on the stability of deep tunnel. Based on the field condition, a numerical model is established to simulate the variation law of stress and displacement of the surrounding rock of tunnel under three working conditions: the excavation direction is parallel, vertical, and intersection to the maximum horizontal principal stress, respectively [16–18].

2. Measurement and Analysis of Crustal Stress in the Mining Area

2.1. Instrument for Measuring Crustal Stress. Instruments utilized for in-situ stress measurement are the most significant part in the field test. The measurement equipment mainly includes a stress meter, stress meter mounting rod, digital dynamic signal test system, confining pressure calibrator, high- and low-temperature test chambers, binder with combining drill bit, and core barrels. Envelope stress meter is the first principle to in-situ stress measurement, and its quality determines the accuracy of measurement results. The improved hollow inclusion strain gauge probe (Figure 1) adopts high-strength nonmagnetic aluminum alloy material, and the instrument cavity behind the device is packaged with thermally conductive silica gel, which has good waterproof, heat dissipation, and shock absorption performance.

From the head to the end of the strain gauge, the sequence of strain gauge following the clockwise order is A-B-C. The strain gauge parallel to the borehole direction is 0# strain gauge, which is 45°, 90°, and 135° clockwise. Taking group A strain gauge as an example, 1# strain gauge as circumferential strain gauge is recorded as A90, 2# strain gauge axial strain gauge recorded as A0, 3# (A45) and 4# (A135) strain gauges are $\pm 45^\circ$ with borehole axis, respectively. Similarly, 5#~12# strain gauges are recorded as B90, B0, B45, B135, C90, C0, C45, and C135, respectively.

2.2. Steps of Ground Stress Measurement and the Stress Relief Method. A horizontal borehole with a 130 mm diameter is drilled on the side wall first (Figure 2-①), and it reached 3~5 times the depth of the tunnel span [19, 20]. The bottom of the hole is flattened with a flat bit, and the horn mouth is punched with a cone bit. Then, a concentric hole with a diameter of 36 mm and a depth of 35~40 cm is punched from the bottom of the hole (Figure 2-②). The stress gauge is sent to the predetermined position in the hole by the installation rod with the directional device (Figure 2-③). After the cementing agent is solidified, the thin-walled drill with a diameter of 130 mm continues to deepen the large pores, so that the rock around the strain gauge is gradually separated from the surrounding rock, which results in stress relief of the core (Figure 2-④). In the process of stress relief, the strain values measured by each strain gauge in the strain gauge are automatically recorded by the bridge conversion device and the data collector. According to instructions, strain data are recorded every 2 cm footage of advance.

After the stress relief is completed, the strain data stored in the data collector will be printed out by the computer, and the stress relief curve is drawn, which represents the curve of the strain value of each strain gauge with the stress relief depth. Then, according to the temperature change of the acquisition circuit measured by the synchronous dual temperature channel, the temperature calibration test is carried out to reduce the temperature influence in the circuit, and the release curve corrected by dual temperature compensation is obtained. The final stable value of the curve is used as the original data for calculating the in-situ stress.

FIGURE 1: Improved hollow inclusion technique [19].

FIGURE 2: Trepanning stress relief method [19].

The confining pressure calibration test is carried out with the original core obtained from the on-site casing release, and the strain value caused by the confining pressure during the measurement process of the hollow inclusion strain gauge is corrected. The elastic modulus, Poisson's ratio, and K coefficient of rock are obtained from the temperature-calibrated stress-relief strain and confining pressure calibration test results. The stress value of original rock is calculated by Formula (1)~(4).

$$\begin{aligned} \varepsilon_\theta &= \frac{1}{E} \{ (\sigma_x + \sigma_y) k_1 + 2(1 - \nu^2) \\ &\quad [(\sigma_y - \sigma_x) \cos 2\theta - 2\tau_{xy} \sin 2\theta] k_2 - \nu \sigma_z k_4 \}, \\ \varepsilon_z &= \frac{1}{E} [\sigma_z - \nu(\sigma_x + \sigma_y)], \\ \gamma_{\theta z} &= \frac{4}{E} (1 + \nu) (\tau_{yz} \cos \theta - \tau_{zx} \sin \theta) k_3, \\ \varepsilon_{\pm 45^\circ} &= \frac{1}{2} (\varepsilon_\theta + \varepsilon_z \pm \gamma_{\theta z}), \end{aligned} \quad (1)$$

where $\sigma_x, \sigma_y, \sigma_z, \tau_{xy}, \tau_{yz}, \tau_{zx}$ —the three-dimensional stress component at the measuring point; ε_θ —circumferential strain value of hole wall; ε_z —axial strain value of hole wall; $\varepsilon_{\pm 45^\circ}$ —strain value in the direction of $\pm 45^\circ$ with borehole axis; $\gamma_{\theta z}$ —shear strain value; E —elastic modulus of rock at measuring point; ν —Poisson's ratio of measuring point rock; θ —angle between strain gauge and x axis, where counterclockwise rotation is positive; and k_1, k_2, k_3, k_4 —four correction coefficients given by Worotnicki and Walton [21], collectively known as K coefficients, are calculated by the following formula:

$$\left. \begin{aligned} k_1 &= d_1 (1 - \nu_1 \nu_2) \left(1 - 2\nu_1 + \frac{R_1^2}{\rho^2} \right) + \nu_1 \nu_2 \\ k_2 &= (1 - \nu_1) d_2 \rho^2 + d_3 + \nu_1 \frac{d_4}{\rho^2} + \frac{d_5}{\rho^4} \\ k_3 &= d_6 \left(1 + \frac{R_1^2}{\rho^2} \right) \\ k_4 &= d_1 (\nu_2 - \nu_1) \left(1 - 2\nu_1 + \frac{R_1^2}{\rho^2} \right) \nu_2 + \frac{\nu_1}{\nu_2} \end{aligned} \right\}, \quad (2)$$

where

$$\left. \begin{aligned} d_1 &= \frac{1}{1 - 2\nu_1 + m^2 + n(1 - m^2)} \\ d_2 &= \frac{12(1 - n)m^2(1 - m^2)}{R_1^2 D} \\ d_3 &= \frac{1}{D} [m^4(4m^2 - 3)(1 - n) + x_1 + n] \\ d_4 &= \frac{-4R_1^2}{D} [m^6(1 - n) + x_1 + n] \\ d_5 &= \frac{3R_1^4}{D} [m^4(1 - n) + x_1 + n] \\ d_6 &= \frac{1}{1 + m^2 + n(1 - m^2)} \\ D &= (1 + x_2 n) [x_1 + n + (1 - n)(3m^2 - 6m^4 + 4m^6)] \\ &\quad + (x_1 - x_2 n)m^2 [(1 - n)m^6 + (x_1 + n)] \end{aligned} \right\}. \quad (3)$$

where R_1 —inner radius of hollow inclusion; R_2 —installation hole radius; G_1 —rigid modulus of hollow inclusion material; G_2 —rigid modulus of rock; ν_1 —Poisson's ratio of hollow inclusion material; ν_2 —Poisson's ratio of rock; and ρ —radial distance of resistance strain gauge in hollow inclusion.

$$\begin{aligned} n &= \frac{G_1}{G_2}, \\ m &= \frac{R_1}{R_2}, \end{aligned} \quad (4)$$

$$x_1 = 3 - 4\nu_1,$$

$$x_2 = 3 - 4\nu_2.$$

2.3. Layout of Measurement Point. This site is developed by shafts, and it is divided into main shaft and air shaft. The main shaft is located outside the rock movement range of footwall in No. 55 exploration line ore body. The net section of the shaft is 5.5 m, and the wellhead elevation is +986.0 m, while the bottom elevation is +30 m. The full depth of the shaft is 956 m, which contains 11 levels from +600 m to +100 m, and the height of each level is 50 m. According to the progress of the infrastructure project in the mining area, seven in-situ stress measuring points are arranged in the five infrastructure levels of +350 m, +300 m, +250 m, +100 m, and +80 m, of which one measuring point is arranged in the level of +250 m, +100 m, and +80 m, and one measuring point is arranged in the two wings of +350 m level and +300 m level, respectively. The layout positions and drilling conditions of each measuring point are shown in Table 1.

2.4. Stress Relief Analysis. According to the data recorded by the resistance strain gauge during the stress relief process of seven boreholes, the stress relief curve is drawn. Based on the temperature variation of the acquisition circuit measured by the synchronous double temperature channel, the stress relief curve is compensated and corrected, and the stress relief curves are as shown in Figure 3.

It can be seen from the stress relief curve that the change of recovery strain with footage depth is basically synchronous in the process of core release. The strain values measured by each strain gauge are generally very small before the lifting depth of the sleeve hole reaches the measured section (i.e., the section where the strain gauge is located). Some strain gauges even measure negative strain values, which is the result of stress transfer caused by casing holes, which is equivalent to "excavation effect." Many curves change in opposite directions when the removal depth of the casing is close to the measured section. The maximum strain occurs when the casing bit passes near the measuring section. When the socket depth exceeds a certain distance from the measured section, the strain value gradually stabilizes and the curve tends to be stable. The final stable value will be used as the original data for calculating in-situ stress.

2.5. Confining Pressure Calibration Test. The calculation of crustal stress requires the elastic modulus and Poisson's ratio of rock at each measuring point. However, the state and mechanical properties of underground rock mass are

TABLE 1: Position of each measuring point of ground stress and parameters of test holes.

	Location	Coordinate (x, y, z)	Hole depth (m)	Orientation (°)	Dip angle (°)	Burial depth (m)	RQD (%)
1#	350 m level	(557356.94, 5098624.77, 352.3)	9.1	84	3.5	526	42
2#	350 m level	(556825.46, 5099062.66, 352.3)	6.3	180	4	526	92.3
3#	300 m level	(557315.27, 5098571.06, 301.3)	6.2	305	3	576	52.7
4#	300 m level	(556630.72, 5099129.99, 301.3)	7.0	215	4.3	576	98.9
5#	250 m level	(557255.03, 5098459.76, 252.1)	8	180	4	626	78.4
6#	100 m level	(557367.82, 5098517.70, 102.3)	6	25	4	776	97.5
7#	80 m level	(557171.35, 5098497.59, 81.8)	6.1	172	4.5	796	98

FIGURE 3: Continued.

FIGURE 3: The relief curve of double temperature compensation correction: (a) 350 m level 1# measuring point, (b) 350 m level 2# measuring point, (c) 300 m level 3# measuring point, (d) 300 m level 4# measuring point, (e) 250 m level 5# measuring point, (f) 100 m level 6# measuring point, and (g) 80 m level 9# measuring point.

extremely complex, and the properties of two neighboring rock mass could have a critical variation. In order to ensure the accuracy of in-situ stress calculation results, the rock elastic modulus and Poisson's ratio must be able to truly represent the original rock where the strain gauge is located. It has a great significance for measurement accuracy and crustal stress calculation to conduct confining pressure calibration tests by using the original rock core obtained by the borehole stress relief method.

After the stress relief is completed, the hollow inclusion strain gauge is still bonded in the central hole. By applying confining pressure to the core of the sleeve hole, the strain value caused by confining pressure in the measurement process of the hollow inclusion strain gauge is corrected. From the measured confining pressure-strain curve, the

elastic modulus and Poisson's ratio of the rock can be calculated according to the thick wall tube theory [3, 21].

The elastic modulus and Poisson's ratio of surrounding rock are calculated by using the hollow inclusion strain obtained in the second loading-unloading process as the effective data. The elastic modulus, Poisson's ratio, and K coefficient of surrounding rock at each measuring point are calculated according to formulas (5), (7), and (8). The calculation results are shown in Table 2.

$$E = K_1 \left(\frac{P_0}{\varepsilon_\theta} \right) \frac{R^2}{R^2 - r^2}, \quad (5)$$

$$\nu = \frac{\varepsilon_z}{\varepsilon_\theta}$$

TABLE 2: Elastic modulus and Poisson's ratio at each measuring point.

Point	E (GPa)	ν	K_1	K_2	K_3	K_4
350 m level	36.18	0.141	1.123	1.114	1.015	0.967
350 m level	36.10	0.140	1.124	1.117	1.005	0.997
300 m level	35.010	0.131	1.2641	1.1508	1.0078	0.9885
300 m level	36.14	0.143	1.1557	1.1059	1.0816	0.9776
250 m level	35.14	0.132	1.1616	1.1608	1.1078	0.9971
100 m level	36.22	0.141	1.216	1.108	1.105	0.9885
80 m level	36.18	0.142	1.325	1.210	1.134	0.9842

2.6. The Distribution Law of Crustal Stress. Based on the formula between the strain of the hollow inclusion and the three-dimensional stress component, combined with the measured rock mechanics parameters and the stress relief curve of the sleeve core, utilizing the least square method of curve fitting, the three-dimensional in-situ stress measurement results of each measuring point are calculated as shown in Table 3.

Results show that the distribution of in-situ stress field in the Huanaote mining area has the following rules:

- (1) Each measuring point has two principal stresses closing to the horizontal direction, and the inclination angle varies from 2.73° to 34.87° , while a principal stress is close to the vertical direction, with a 20° to the vertical direction.
- (2) The direction of the maximum principal stress is close to horizontal, and all the maximum horizontal principal stress directions in seven measuring points follow the NE-SW direction, which is basically consistent with the maximum principal stress direction of the regional tectonic stress field. Six of the maximum principal stresses at seven measuring points have an angle less than or equal to 10° with the horizontal plane, which are almost parallel. For the ratio of the maximum horizontal principal stress ($\sigma_{h,max}$) to the vertical principal stress (σ_v) in seven measuring points, they are all more than 1.5 times, with the maximum value of 1.89 times, compared with the minimum value of 1.57 times. The average value is 1.76 times (Table 4). Results show that the stress field in the Huanaote mining area is dominated by horizontal tectonic stress, rather than self-weight stress. The size and direction of the maximum horizontal principal stress are the first principle to the stability of tunnel surrounding rock.
- (3) There is a positive correlation between the depth and maximum horizontal principal stress ($\sigma_{h,max}$), minimum horizontal principal stress ($\sigma_{h,min}$), and vertical principal stress (σ_v), respectively. Moreover, this correlation approximates to a linear growth.

2.7. Crustal Stress Field Model. In order to obtain an accuracy variation of stress field with depth, the linear regression method is used to analyze the stress values in measured points. The regression equations of the maximum horizontal principal stress, the minimum horizontal

principal stress, and the vertical principal stress with depth (formulas (6)–(8)) and the regression curve (Figure 4) are obtained. The regression equation represents the ground stress field model of the mining area, which gives the mechanical boundary conditions of the mining area necessary for numerical simulation, physical simulation research and various quantitative mining design calculation, support reinforcement, and ground pressure control.

Regression equation (ground stress field model):

$$\sigma_{h,max} = 0.046H - 2.46, \quad (6)$$

$$\sigma_{h,min} = 0.021H - 1.55, \quad (7)$$

$$\sigma_v = 0.025H - 1.07, \quad (8)$$

where $\sigma_{h,max}$ —maximum horizontal principal stress, MPa; $\sigma_{h,min}$ —minimum horizontal principal stress, MPa; σ_v —vertical principal stress, MPa; and H —burial depth, m.

3. Stability Analysis of Tunnel Surrounding Rock

According to the mine in-situ stress field model and the field engineering environment, the numerical model is established, and FLAC3D software is used to simulate and analyze the variation law of stress and displacement of roadway surrounding rock under three working conditions: the excavation direction is parallel, vertical, and intersection to the measured maximum horizontal principal stress direction, respectively [22–31]. It provides a theoretical basis for the optimization of underground mining preparation engineering.

3.1. The Axial Direction of Tunnel Is Consistent with the Direction of Maximum Principal Stress. Based on the real condition of the 350 m-level 55-line tunnel, a numerical analysis model was established to study the variation characteristics of surrounding rock stress and deformation of the tunnel with excavation face advancing. The physical and mechanical parameters of numerical model rock are shown in Table 5.

The calculation model is simplified to a quasi-three-dimensional case, and the distance between the surrounding boundary and the center of the tunnel is about 5 times to the width of the tunnel, which reduces the influence of boundary conditions on the simulation results. The establishment model and grid division are shown in Figure 5. The horizontal direction width of the vertical tunnel axis is 40 m, the

TABLE 3: Calculation results of principal stress at each measuring point.

No.	Burial depth (m)	Maximum horizontal principal stress			Minimum horizontal principal stress			Vertical principal stress		
		Value (MPa)	Orientation (°)	Dip (°)	Value (MPa)	Orientation (°)	Dip (°)	Value (MPa)	Orientation (°)	Dip (°)
1 [#]	526	20.74	212.58	-2.73	7.28	122.74	3.32	10.97	-136.72	85.69
2 [#]	526	19.63	267.8	8.38	7.36	177.76	0.25	11.67	86.10	81.61
3 [#]	576	19.91	213.89	-9.10	10.33	306.21	14.10	12.66	92.03	73.11
4 [#]	576	23.0	209.98	-11.75	10.6	123.36	15.83	12.7	-95.09	70.08
5 [#]	626	27.22	250.01	9.99	11.30	332.96	-34.87	15.99	173.69	-53.30
6 [#]	776	35.69	241.39	-0.56	14.69	151.56	16.15	18.82	-30.52	73.84
7 [#]	796	37.97	261.75	9.93	18.57	167.01	25.24	21.25	11.51	62.61

TABLE 4: Ratio of the maximum horizontal principal stress to vertical principal stress.

No.	1	2	3	4	5	6	7
$\sigma_{h,max}/\sigma_v$	1.89	1.68	1.57	1.80	1.70	1.89	1.78

FIGURE 4: The variation law between $\sigma_{h,max}$, $\sigma_{h,min}$, σ_v , and depth.

TABLE 5: Geomechanical parameters of rock mass.

Rock mass density ρ ($\text{kg}\cdot\text{m}^{-3}$)	Elastic model E (GPa)	Friction angle φ (°)	Cohesion c (MPa)	Passion's ratio μ	Tensile strength σ_t (MPa)
2800.0	18.0	36.0	0.7	0.14	0.5

FIGURE 5: Model and grid partition diagram.

FIGURE 7: Stress variation curve of side walls.

vertical direction height is 40 m, and the tunnel axis direction length is 20 m. There are 16,400 grid units and 17,661 nodes. The excavation calculation adopts the displacement boundary condition. According to the distribution characteristics of the measured crustal stress field in the mining area, the vertical load applied on the upper boundary is about 10.0 MPa, the maximum horizontal principal stress is 20.0 MPa (y -axis direction), and the minimum horizontal principal stress is 7.0 MPa (x -axis direction).

3.1.1. Disturbance Stress Analysis. The tunnel excavation produces disturbance stress, and the tunnel model sets stress monitoring points at the midpoint of the two sides and the midpoint of the roof every 2 m; at the same time, monitoring points are set at the depth of 1 m to explore the depth of disturbance. With the gradual advancement of the working face, the stress changes of the monitoring points on the two sides of the tunnel are shown in Figure 6.

It can be seen from Figure 6 that the horizontal stress of the 1 m-depth monitoring point after the excavation is about 0.8 MPa, and it reaches the peak between 4.5 and 8.5 MPa in front of excavation face, with a 3.5 m disturbance depth.

The monitoring points are set every 2 meters at the 1 m depth of side walls. The horizontal stress variation curve of two sides during the excavation is shown in Figure 7. It can

be seen from the figure that the stress change is large in the range of about 3 m from the excavation face, and the stress of two side walls is reduced by about 7 MPa from the excavation of the tunnel.

Monitoring points are set every 2 m at the 1 m depth inside the tunnel roof. Its vertical stress change during the excavation is shown in Figure 8. It demonstrates that the excavation disturbance has a critical influence on the roof, with a significant stress fluctuation. Moreover, a serious stress variation occurs at 4 m away from the excavation face. The vertical stress of the roof caused by tunnel excavation is reduced to about 8 MPa. Figure 8(b) is the vertical stress variation cloud map of the tunnel roof. It can be seen that the roof experiences compressive stress, which is about 0.3 MPa, and the stress is mostly concentrated in the bottom corner and arch shoulder of the tunnel. Meanwhile, the maximum stress inside the side wall reaches 16.5~17.5 MPa.

3.1.2. Analysis of Tunnel Convergence Displacement. The total length of the tunnel model is 20 m, and the monitoring point is set at 10 m. The horizontal convergence curve of the two side walls during the excavation is shown in Figure 9. It indicates that the horizontal displacement of the wall reaches 26 mm after the excavation. When the monitoring point is not reached, the displacement increases slowly. After the

FLAC3D 5.00
 ©2012 Itasca Consulting Group, Inc.
 History
 — 751 ZZ-Stress of zone 163826
 — 752 ZZ-Stress of zone 163836
 — 753 ZZ-Stress of zone 163866
 — 754 ZZ-Stress of zone 163876
 — 761 ZZ-Stress of zone 163896
 — 762 ZZ-Stress of zone 163926
 — 763 ZZ-Stress of zone 163936
 — 764 ZZ-Stress of zone 163966
 — 771 ZZ-Stress of zone 163976
 vs. Step

(a)

FLAC3D 5.00
 ©2012 Itasca Consulting Group, Inc.
 Contour of ZZ-Stress
 Calculated by: Volumetric Averaging

(b)

FIGURE 8: Variation of vertical stress of tunnel roof: (a) stress variation curve of tunnel roof and (b) stress variation cloud map of tunnel roof.

excavation happened at the monitoring point, the wall displacement increases significantly (12 mm), and the horizontal displacement change of the excavation disturbance in

the depth range of 3 m is within 20 mm. With the advancing of working face, the displacement gradually stabilized. Figure 9(c) is the monitoring point in the middle of the side

FLAC3D 5.00
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History
— 31 X-displ of gp 167701
vs. Step

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History
— 321 X-displ of gp 177424
vs. Step

FLAC3D 5.00
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History
— 21 X-displ of gp 167525
— 22 X-displ of gp 167569
— 23 X-displ of gp 167613
— 24 X-displ of gp 167657
— 31 X-displ of gp 167701
— 32 X-displ of gp 167745
— 33 X-displ of gp 167789
— 34 X-displ of gp 167833
— 41 X-displ of gp 167877
vs. Step

(c)

FIGURE 9: Horizontal displacement curve of tunnel sides: (a) displacement variation curve of the left side wall, (b) displacement variation curve of 1 m depth in the left side wall, and (c) displacement variation curve of the monitoring point.

FIGURE 10: Development of vertical displacement of rock mass in the middle of tunnel roof.

wall set every 2 m in the excavation face of the tunnel. It demonstrates that the excavation of the tunnel has a great displacement influence on two side walls. The displacement change within 4 m from the excavation face is large, and the displacement outside 5 m is basically stable. Therefore, the excavation face of the tunnel is within 4~5 m, which indicates a major region for supporting.

Tunnels adopt the full-face excavation method. With the excavation, the vertical displacement development of rock mass in the middle of tunnel roof is shown in Figure 10.

The variation of roof vertical displacement affected by excavation is shown in Figure 11. Monitoring points are set

every 2 m on the roof of the tunnel excavation face. After the excavation, its displacement reaches about 30 mm, compared with 24 mm at depth of 1 m. When the monitoring point is not reached, the displacement increases slowly. After the excavation occurs at the monitoring point, the displacement of the roof increases significantly (9 mm), while the displacement of the wall at 1 m depth is 4 mm. It has the most significant changes in the 2 m range of excavation.

3.2. Vertical Working Condition of Tunnel Axial Direction and Maximum Principal Stress Direction. Based on the

FIGURE 11: Vertical displacement curve of tunnel roof: (a) displacement variation curve of tunnel roof and (b) displacement variation curve of tunnel roof at 1 m depth.

engineering background of the 350 m horizontal 55-line tunnel, a numerical model was established to study the variation characteristics of stress and deformation of surrounding rock when the axial direction of tunnel is perpendicular to the direction of maximum principal stress.

3.2.1. Mining Stress Analysis. The tunnel model sets stress monitoring points every 2 m at the midpoint of the two side walls and the midpoint of the roof. With the gradual advancement of the working face, the horizontal stress changes of the monitoring points of the two sides of the tunnel are shown in Figure 12.

It can be seen from Figure 12 that the horizontal stress of the two side walls after the excavation is stable within 2 MPa; and the stress disturbance at 4 m away from the excavation surface began to stabilize. The stress range of the two side walls is about 5 m, and the stress change value is within 12 MPa.

Monitoring points are set at 1 m depth of tunnel roof with 2 m space, and the change of vertical stress of roof during excavation is shown in Figure 13. It can be seen from Figure 13(a) is that the stress fluctuation of roof is relatively

obvious, and the stress changes greatly within the range of about 4 m from the excavation face. The vertical stress of the roof is reduced by about 7 MPa due to tunnel excavation and finally stabilized at about 3 MPa. Figure 13(b) shows the cloud map of the stress variation of tunnel roof, which the roof stress is small, within 0.6 MPa; the stress is mostly concentrated in the bottom corner of the tunnel and the arch shoulder, and the maximum stress in the side wall is as high as 13.4 MPa. At the depth of 2 m, the maximum principal stress is up to 35 MPa.

3.2.2. Analysis of Tunnel Convergence Displacement. The horizontal convergence curve of the two side walls during the excavation is shown in Figure 14. After the excavation, the horizontal displacement reaches 45 mm; when the monitoring point is not reached, the displacement increases slowly. When the excavation reaches the monitoring point, the displacement increases significantly to 12 mm. The displacement of the monitoring point within 4 m depth from the excavation face changes significantly.

The variation of the vertical displacement at roof influenced by excavation is shown in Figure 15. After the

FIGURE 12: Horizontal stress variation curve of two sides of tunnel.

excavation, the vertical displacement of the roof is about 34 mm, and the displacement at the depth of 1 m of the roof is 11 mm. When the monitoring point is not reached, the displacement increases slowly. When the excavation reaches the monitoring point, the displacement of the roof increases significantly to 15 mm, and the displacement of the wall at 1 m depth is less than 4 mm; the displacement changes in the 3 m range of excavation are obvious.

3.3. The Actual Heading Direction of Tunnel Intersects with the Direction of Maximum Principal Stress. Taking the contact tunnel of 100 m horizontal hoist room as the engineering background, the FLAC3D numerical analysis model is established to study the surrounding rock stress and deformation characteristics when the actual tunneling direction intersects with the maximum principal stress direction. The chamber mouth of the winch room is the 6# in-situ stress measuring point. The angle between the tunneling direction and the direction of the maximum principal stress is 52° . The establishment of the model and the meshing are shown in Figure 16. The x -axis direction is the minimum principal stress direction, with the length of 60 m. The y -axis direction is the maximum principal stress direction, and the

width is 20 m. The height of the z -axis direction is 40 m. The length of the tunnel axis direction is 20 m. The total number of meshing units is 122,930, combined with 81,045 nodes.

3.3.1. Mining Stress Analysis. The tunnel adopts the full-section excavation method, and the maximum principal stress distribution variation rule for each 2 m excavation is shown in Figure 17. The stress concentration area after the excavation is mainly located at 2 m ahead of the excavation section, about 24 MPa, and the stress concentration is up to 27 MPa at about 2 m inside the rock mass of the two side walls. Affected by the model boundary, the stress concentration position at the early stage of excavation is distributed in the right side wall, and at the late stage of excavation, it is distributed in the left side wall. During the excavation process, the stress on the right side of the excavation face is greater than that on the left side, and the difference between the two sides is within 0.5 MPa.

The principal stress variation curve and the principal stress change cloud diagram of the tunnel model roof monitoring point are shown in Figure 18. In Figure 18(a), the initial stress value of excavation is about 18 MPa. With the gradual excavation of the tunnel, the stress redistributes and concentrates in front of the excavation section, which

FIGURE 13: Vertical stress variation of tunnel roof: (a) Stress variation curve of tunnel roof, (b) Stress variation cloud map of tunnel roof.

the maximum value is 24.59 MPa, while the stress value of the roof monitoring point is 21 MPa. When the excavation reaches the monitoring point, the stress decreases rapidly

and finally stabilizes at about 6 MPa, and the stress variation is 15 MPa. In Figure 18(b), the change trend of the minimum principal stress is basically the same as that of the maximum

FIGURE 14: Horizontal displacement curve of the two sides of tunnel.

principal stress, and the stress difference is 3 MPa. The tensile stress appears in the middle of the roof and floor, and the stress value is about 0.23 MPa.

3.3.2. Analysis of Tunnel Convergence Displacement. The length of y axis in the model is 20 m, and the monitoring point is set at 10 m. The convergence curve of the roof direction during the excavation is shown in Figure 19. After the excavation, the maximum displacement of the roof is 32 mm (1# curve), the vertical displacement of the tunnel roof at 1 m depth is 28 mm (3# curve), and the maximum displacement of the tunnel floor is 8 mm (2# curve). According to the displacement curve of the monitoring point, the excavation of the tunnel not only leads to the deformation of the excavated tunnel but also produces the displacement change in the unexcavated area. When the monitoring point is not reached, the displacement slowly increases to 6 mm, which is the displacement change caused by stress release. When it reaches the monitoring point, the displacement of the monitoring point increases significantly to 10 mm; with the continuous advancement of the working face, the displacement growth rate gradually decreases. In order to effectively control the continuous deformation of surrounding rock caused by tunnel section excavation, support measures should be taken in time.

3.4. Stability Analysis of Surrounding Rock under Different Working Conditions. According to the tunnel excavation direction and the measured direction of the maximum horizontal principal stress under parallel, vertical, and intersecting conditions, simulation results of surrounding rock stress and displacement are shown in Table 6:

- (1) When the tunneling direction is parallel to the measured maximum horizontal principal stress direction, the displacement of two sides of the tunnel, the displacement of roof, and the displacement at the depth of 1 m of roof are the least, indicating that the stability of the tunnel is better when the tunneling direction of the tunnel is parallel to the measured maximum horizontal principal stress direction.
- (2) When the tunnel excavation direction is perpendicular to the measured maximum horizontal principal stress direction, the displacement of the two side walls, roof, the displacement variation of the roof at 1 m depth, and the maximum stress of the roof are all greater than the other two conditions, indicating that the stability and safety of the tunnel are worst when the tunnel excavation direction is perpendicular to the measured maximum horizontal principal stress direction.

FLAC3D 5.00
©2012 Itasca Consulting Group, Inc.
History
 11 Z-displ of gp 168736
 111 Z-displ of gp 168648
 112 Z-displ of gp 168670
 113 Z-displ of gp 168692
 114 Z-displ of gp 168714
 1112 Z-displ of gp 168758
 1113 Z-displ of gp 168780
 1114 Z-displ of gp 168802
 1115 Z-displ of gp 168824
 vs. Step

(a)

FLAC3D 5.00
©2012 Itasca Consulting Group, Inc.
History
 12 Z-displ of gp 173823
 311 Z-displ of gp 173735
 312 Z-displ of gp 173757
 313 Z-displ of gp 173779
 314 Z-displ of gp 173801
 372 Z-displ of gp 173845
 373 Z-displ of gp 173867
 374 Z-displ of gp 173889
 375 Z-displ of gp 173911
 vs. Step

(b)

FIGURE 15: Vertical displacement curve of tunnel roof: (a) displacement curve of tunnel roof and (b) displacement curve of tunnel roof at 1 m depth.

FIGURE 16: Model and grid partition diagram.

Contour of Min. Principal Stress
Plane: on
Calculated by: Volumetric Averaging

Contour of Min. Principal Stress
Plane: on
Calculated by: Volumetric Averaging

Contour of Min. Principal Stress
Plane: on
Calculated by: Volumetric Averaging

Contour of Min. Principal Stress
Plane: on
Calculated by: Volumetric Averaging

Contour of Min. Principal Stress
Plane: on
Calculated by: Volumetric Averaging

Contour of Min. Principal Stress
Plane: on
Calculated by: Volumetric Averaging

FIGURE 17: The maximum principal stress distribution cloud chart of tunnel.

FIGURE 18: Variation of tunnel roof stress: (a) maximum principal stress and (b) minimum principal stress.

- (3) When the actual excavation direction intersects with the measured maximum horizontal principal stress direction, the maximum stress of the two side walls

reaches 27 MPa, which is far greater than the other two conditions. In the actual construction process, the support and maintenance of the two sides of the

FIGURE 19: Vertical displacement curve of tunnel roof.

TABLE 6: Mining stress and deformation of surrounding rock under different working conditions.

	Horizontal displacement of two side walls (mm)	Roof displacement (mm)	Roof displacement at 1 m depth (mm)	Maximum stress of two side walls (MPa)	Maximum stress of roof (MPa)
Conditions 1	26	30	24	17	30
Conditions 2	45	34	31	13.4	35
Conditions 3	35	32	28	27	24.5

* Conditions 1: the axial direction of tunnel is consistent with the direction of maximum principal stress; Conditions 2: axial direction of tunnel perpendicular to direction of maximum principal stress; Conditions 3: the actual heading direction of tunnel intersects with the direction of maximum principal stress.

tunnel should be strengthened to ensure the stability of the tunnel and the safety of personnel and equipment.

4. Conclusion

The in-situ stress field model of the Huanaote mining area is constructed by “double temperature compensation and considering the nonlinear measurement technology of in-situ stress relief of rock mass.” On this basis, a numerical model is established according to the field engineering environment to simulate and analyze the variation law of stress and displacement of tunnel surrounding rock under three working conditions: parallel, vertical, and intersecting between the direction of tunnel excavation and the direction of measured maximum horizontal principal stress. Following conclusions are obtained:

- (1) The direction of the maximum horizontal principal stress changes in the range of $212^{\circ}\sim 267^{\circ}$, its value changes in the range of 20~37 MPa, and the lateral pressure coefficient of each measuring point changes in the range of 2.05~2.85. The horizontal tectonic stress in the mining area is large, and the horizontal tectonic stress is dominant. The size and direction of the maximum horizontal principal stress are the key to the stability of tunnel surrounding rock.
- (2) The vertical principal stress is the intermediate principal stress, whose dip angle changes in the range of $53.30^{\circ}\sim 85.69^{\circ}$, and the value changes in the range of 10.97~21.25 MPa, which is close to the self-weight stress of overlying rock mass.
- (3) When the tunneling direction is parallel to the direction of the measured maximum horizontal

principal stress, the displacement of the two side walls, roof, and 1 m depth of the roof are the least, indicating that the stability of the tunnel is better when the tunneling direction of the tunnel is parallel to the direction of the measured maximum horizontal principal stress.

The mine is a newly built mine. During the design and construction of subsequent development and mining preparation engineering, it is recommended to excavate the roadway along the direction of the maximum horizontal principal stress to reduce the impact of original rock stress on roadway stability.

- (4) When the actual heading direction of the tunnel intersects with the measured maximum horizontal principal stress direction, the maximum stress of the two side walls reaches 27 MPa, which is far greater than the parallel and vertical working conditions of the heading direction of the tunnel and the measured maximum horizontal principal stress direction. In the actual construction process, the support and maintenance of the two side walls should be strengthened to ensure the stability of the tunnel and the safety of personnel and equipment.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare no conflicts of interest

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