**CIVIL & ENVIRONMENTAL ENGINEERING | RESEARCH ARTICLE**

**Strength response characteristics and coupling support of deep roadway in soft rock masses**

Huang Jun-jie, Qin Ya-guang*, Zhao Shuang, Wang Wei and Wen Lei

**Abstract:** Monitoring analysis was performed for the deformation characteristics of a deep roadway in soft rock masses in a certain mine by the convergence instrument. It was indicated that shotcrete and bolt support for the surrounding rock deformation under forces produced a poor effect so that a shotcrete rockbolt mesh coupling support pattern was put forward and four coupling support schemes were designed to carry out field monitoring for surrounding rock deformations on the basis of above schemes. According to the research, we could summarize as follows. Firstly, when load in floor strata reached a certain value, shear failure took place on the edge of floor; then, with the continuous increase of such a load, shear zones were subsequently expanded and connected together into one big region; at that time, as the load on floor strata had been up to its limit, the floor could be settled sharply as long as this load went up slightly so that the uplift of rock mass gave rise to floor heave. Secondly, bearing capacity of floor rocks was dependent on the average binding force, internal friction angle and unit weight of floor rocks as well as the concentrated stress action width of lateral wall and the average pressure value borne by the roadway floor. Thirdly, after the shotcrete rockbolt mesh coupling support was adopted, two-sided displacement dropped 80% and above, roof subsidence decreased more than 70% and the capacity of floor heave falls by over 90%; as a result, stability of the roadway could be significantly improved.

**Subjects:** Engineering & Technology; Civil, Environmental and Geotechnical Engineering; Tunnelling & Underground Engineering

**Keywords:** roadway in soft rock masses; surrounding rock deformation; floor heave; numerical analysis; coupling support

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**PUBLIC INTEREST STATEMENT**

- Monitoring analysis was performed for the deformation characteristics of a deep roadway in soft rock masses by the convergence instrument.
- A shotcrete rockbolt mesh coupling support pattern was put forward and four coupling support schemes were designed.
- Optimization selection is conducted for different schemes through a comparative analysis of actual engineering monitoring and numerical simulation.
- Reference for roadway support under similar conditions.

**ABOUT THE AUTHORS**

Our team is mainly engaged in mine safety technology related research, the main research contents include: mining safety, roadway support, cavity laser detection, mine information construction, etc. Has participated in the national “11th Five-Year” and “12th Five-Year” technology support plan. Deep soft rock roadway has been a major problem of mine production, through research to find out the soft rock roadway and supporting methods, accumulation under different conditions of soft rock roadway support experience, has high directive value in engineering practice.
1. Introduction
Surrounding rocks of tunnels are found to be in a complex mechanical environment after roadway excavation. Existing theories of rock mass mechanics are only able to approximately solve some mechanical problems and corresponding calculation results may be greatly different from the physical truth sometimes. According to traditional support theories, roof and loose surrounding rocks on both side walls are treated as external loads of support; while the issue of tunnel support is studied by modern elastic theory under ideal assumed conditions. Currently, internationally popular New Austrian Tunneling Method (Rabcewicz, 1975) remains as an approach and the problem of what supporting object is still cannot be answered by it definitely. With regard to the Support Theory of Broken Rock Zone (Tang & Wang, 2006; Zhang, Zhang, Hou, Wu, & Zhou, 2015; Zhou & Li, 2011), the broken rock zone is considered to the final product of surrounding rock deformation and damage regardless of various sophisticated situations and influencing factors that are generated in soft rocks (Corthésy, Leite, Gill, & Gaudin, 2003; Giambastiani, 2014; Milton, 2014). Therefore, it can be said that such a theory is not well-conceived enough. Seen from the research level of quantitative support theory, it is very difficult for it to become a practical and operational theory at present. Considering corresponding reasons, they can be described from the following two aspects. On one hand, as the constitutive relation of various factors taken into account is too complex and there are too many parameters, the calculation become extremely sophisticated and hard; moreover, the determination for the size of supporting force requires the Theory of Strength or the Stability Criterion, but it is difficult to apply the mechanical model into the design of such a force due to insufficient researches at present. On the other hand, it is also difficult for currently established model to think over both the support and the surrounding rock deformation processes. With the increasingly deeper development of mining, multiple problems have exposed to shotcrete and bolt support of soft rock roadway (Kang, 2014; Ma, 2010; Zhu, Wang, Chen, Chen, & Wang, 2011). In case that the stress is not very large, soft rocks are equipped with characteristics of brittle ones; in comparison, when the stress is high, obvious soft rock features can be exhibited due to brittleness reduction and plasticity enhancement.

In this paper, soft rock roadway of No. 600 level (depth of 600 m) in an underground mine is taken as an example to study dynamic evolution laws of soft rock roadway stress as well as the surrounding rock stress and displacement features in floor roadway during their exploitation by adopting a method of field monitoring combining numerical simulation. On this basis, not only is a shotcrete rockbolt mesh coupling support pattern presented, but four support schemes are designed direct at the engineering practice. Then, through a comparative analysis of actual engineering monitoring and numerical simulation, optimization selection is conducted for different schemes.

2. Engineering background
Target mine is an underground mine. Concerning its construction scale, its annual production capacity is 1.3 million tons. As the exploitation extends in depth and breadth and the increase of roadway service life, most of such roadways have deformed and damaged, such as wall caving, roof falling, slumping and floor heave, etc. In addition, surrounding rock stress gradually goes up in roadways owing the increase in their burial depths; hence, problems exposed in roadway support become ever more prominent. The fault zone of this mine is fractured seriously, and the surrounding rock is mainly shale with poor bearing capacity. The maximum principal stress measured is 8.94 MPa. Hardness of the rock in No. 600 level in such a mine ranges from 3f to 5f and its lithology has given priority to shale. According to the experimental results of laboratory, it has been measured that uniaxial compressive strength of shale is 30.71 MPa with a critical load of softening from 9.2 to 15.3 MPa. Net fault surface and excavated section of the roadway are 13.43 and 15.63 m² respectively; and the total length of this project is 2,500 m. Reinforced mortar cement-and-grouted steel bolt and shotcrete for the arch is employed as its support pattern. Moreover, main reinforcement of support material is No. 6 reinforcing steel bar and the bolting mesh size is 2 × 2 m with a 1.5 m long bolt, a 15 mm × 15 mm net dimension, a 1 m array pitch between bolts, a distance of 800–1,000 mm
and a jet thickness of 120 mm. After multiple years of mining, cracking spalling and roof caving phenomena have occurred to both sidewalls of the roadway.

In order to master the damage degree of wall caving, roof falling and floor heave in the roadway, an SWJ-IV convergence instrument is adopted to monitor deformation and displacement of tunnel cross-sections. According to the measured value of this instrument, the semi-parametric regression analysis method (AghaKouchak & Nasrollahi, 2010; Lorino, Sanaa, Robin, & Daudin, 2004) is utilized to eliminate errors. Monitoring results for the obtained two-sided displacement in tested roadway as well as roof subsidence and floor heave capacity in No. 600 level main roadway are shown in Figures 1 and 2.

In accordance with measured results, it can be seen that due to impacts of excavation and in situ stress field, high-stress soft rock roadways are significantly deformed with a two-sided displacement of 155 mm, a roof subsidence of 93 mm and a floor heave capacity of being up to 232 mm which is the greatest one among roadway deformation and displacement followed by two-sided displacement and the smallest roof subsidence. Those monitoring results indicate that current support pattern is unable to effectively control the deformation of surrounding rocks.

Figure 1. Relation between surrounding rock deformation and time.

Figure 2. Relation between floor heave capacity and time.
3. Analysis of soft rock roadway floor heave mechanism

3.1. Stress distribution and mechanic model for both sides of roadway

Due to the internal stress of rock mass that is required to be distributed again after roadway excavation, dead weight of overlying rock mass in the excavation space is shifted to each side of the roadway so as to form $P_a'$, a side abutment pressure higher than stress of in situ rock stress $P_0$. Corresponding stress distribution is given in Figure 3; in which, $P_a$ refers to the peak value of abutment pressure, $N$; $P_0$ to the stress of in situ rock stress, $N$; $B$ to the abutment pressure zone width, $m$ and $r_a$ to the average radius of roadway, $m$.

Here, the entire roadway can be seen as architecture or a structure constituted by three parts of a superstructure formed by the roof and the overlying rock mass, a structure foundation by two-sided rock mass and a foundation bed by floor rocks. Load on the superstructure can be transferred to the foundation bed through the structure foundation and the width of structure foundation is $B$ which is also the width of abutment area (as shown in Figure 4).

$$P_a = nP_0$$
(1)

where, $n$ is the stress concentration factor and $P_a$ is the peak value of abutment pressure, $N$.

The width $B$ of abutment area can be determined by following equation.

$$P_a' = P_u$$
(2)

where, $P_a'$ represents the side abutment pressure, $N$ and $P_u$ represents the ultimate bearing capacity of floor rock mass, $N$.

When the load $P$ in floor strata reaches a certain value, shear failure firstly takes place on the edge of floor; then, with the continuous increase of it, shear fracture zones are correspondingly expanded. Moreover, if the load keeps going up, those zones can be connected together into one big region subsequently; at that time, it is indicated that the load on floor strata has reached its ultimate
bearing capacity $P_u$ and hence it is on the verge of being damaged. Then, with a slight increase in such a load, the floor could be settled sharply so that general shear failure happens to the floor strata and floor heave is also generated by the uplift of floor rock mass.

### 3.2. Ultimate equilibrium of roadway floor

Concerning the floor rock mass under the action of concentrated abutment pressure (upper load), stress state of the rock mass is transferred from elasticity to plasticity provided that its stress has reached or exceeded corresponding ultimate bearing capacity. Consequently, continuous shear-sliding surface is formed in such a rock mass. At the same time, the active status area AOC, the passive status area ADB and their transition zone ACD take their shapes in the floor rock mass, as shown in Figure 4. According to rock mechanics properties (Mei & Chen, 2013), because the rock mass has a certain number of primary and secondary structural surface, combined with the impact of excavation of the construction, so the surrounding rock can not be a complete and continuous as a whole, so the use of Terzaghi’s theory to calculate the surrounding rock pressure effect is also better. The side average concentrated limit stress (or the ultimate bearing capacity of floor) on width $B$ corresponding to the floor rock mass which is in a plasticity state of ultimate equilibrium can be achieved within the range of OCDB as follows.

$$P_u = \frac{1}{2} rBN_r + qN_q + CN_c$$  \hspace{1cm} (3)

In this equation, $N_r$, $N_q$ and $N_c$ are all bearing capacity coefficients serving as functions for the internal friction angle of floor rock mass.

To be specific,

$$N_q = e^{\tan \phi} g^2 (45^\circ + \phi/2)$$  \hspace{1cm} (4)

$$N_c = (N_q - 1) \cot \phi$$  \hspace{1cm} (5)

$$N_r \approx 2(N_q + 1) \tan \phi$$  \hspace{1cm} (6)

where $C$—the average binding force of floor rock mass; $\phi$—the internal friction angle of floor rock mass; $r$—the unit weight of floor rock mass and $q$—the counterforce of floor support.

Within the scope of width $B$, the average value of side concentrated pressure is approximate to,

$$P_n = \frac{P_0}{2} \left( n + \frac{L}{B} \right)$$  \hspace{1cm} (7)

where $L$—width of elastic pressure area in the pressure zone; $B$—width of side abutment pressure; $P_u$—ultimate bearing capacity of floor rock mass and $q$—counterforce of floor support, $\alpha = 450 - \phi/2$.

1. If $P_u > P_n$, floor rock mass in the roadway is steady or locally damaged without the phenomenon of floor heave.

2. If $P_u \leq P_n$, or, when the side concentrated pressure of roadway exceeds the ultimate bearing capacity of floor rock mass, general plastic shear failure can be generated by the rock mass; then, the shear failure object extrudes out of the floor into the interior of roadway along the continuous sliding surface OCDB so that floor heave comes into being.
4. Numerical simulation for stresses of surrounding rocks before and after roadway support

4.1. Numerical model
As a qualitative or quasi quantitative method of rock mass stability evaluation, numerical simulation is required to make some necessary assumptions about rock mass medium properties and computational simulation, etc. Large- or larger structural planes which control the stability of roadway surrounding rocks need to be thought about at the time of analog computation. The Drucker-Prager Yield Criterion (D-P Criterion) (Alejano & Bobet, 2012; Shin, Kim, Kim, & Rhee, 2015) adopted by numerical simulation has been extensively applied because it is able to reflect compression-shear failure nature for geotechnical materials.

In this model, the simulation range can be put as length is 52 m and height is 40 m. While vertical compressive stress 6.354 MPa is exerted uniformly on the upper surface of this model that takes advantage of s stress displacement boundary condition, a horizontal compressive stress 1.197 MPa that varies together with changes in depth is put on both sides of it. Moreover, for the lower surface of such a model, vertical displacement is fixed with lateral displacement implemented for its left and right boundary constraints.

4.2. Simulation parameters
Considering differences in the test pieces used by rock mechanics experiment and the rock mass of the project, reduction of experimental data is carried out. Mechanical parameters of rock mass after reduction can be seen from Table 1.

4.3. Analysis on spatial and temporal distribution effects of mechanical response for roadway surrounding rocks

4.3.1. Spatial and temporal distribution effects of surrounding rock stress
As a surrounding rock distress distribution diagram for No. 600 level roadway before and after excavating and timbering of a tunnel, Figure 5 shows that after roadway excavation, the stress of surrounding rocks is the compressive one. Under the circumstance of roadway excavation without support, stress concentration happens to two arch angles of the three-centered arch tunnel and base angle of two sides. The maximum stress with a peak value of 14.8 MPa appears in above two arch angles; the stress on other parts of such a back is distributed uniformly with a small range of variation. Regarding the base angle, its stress peak is 12.4 MPa and the maximum stress on two sides is 9.46 MPa. However, after shotcrete and bolt support is adopted, stress state of surrounding rocks is changed by bolt so that stress on both sides of the roadway significantly falls followed by two arch angles whose stress peak value is 9.8 MPa. In addition, the stress on base angle is still very large and it has a peak value of 11.2 MPa. In case that existing shotcrete and bolt support is employed in this mine, stress of the roadway surrounding rock decreases as a whole which can be seen

| Table 1. Mechanical parameters of rock mass applied in simulation |
|--------------------------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Category | Density (10^3 kg/m³) | Elasticity modulus (GPa) | Poisson ratio | Tensile strength (MPa) | Internal friction angle (°) | Compression strength (MPa) | Cohesion (MPa) |
|-----------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| Roof Dolo-mite | 2.66 | 5.63 | 0.28 | 0.79 | 32.27 | 18.82 | 6.05 |
| Ores Phosphate ore | 2.76 | 3.68 | 0.32 | 1.19 | 42.18 | 8.68 | 4.32 |
| Floor Sandstone | 2.52 | 6.63 | 0.23 | 0.46 | 42.56 | 20.10 | 5.51 |
| Shale | 2.75 | 2.37 | 0.22 | 0.40 | 38.87 | 5.47 | 3.04 |
from Figure 5. That means the phenomenon of stress concentration has not been solved fundamentally. Especially the stress on base angle, it is oversized with a peak value of 12.2 MPa. Furthermore, obvious stress concentration here is generated by floor heave that is given rise to in the roadway. This is in consistency with the field measured result and the conclusion of theoretical analysis.

4.3.2. Spatial and temporal distribution effects of surrounding rock displacement

Before and after shotcrete and bolt support (Unit/m); (b) Vertical displacement distribution before and after shotcrete and bolt support (Unit/m)

Figure 6(a) and (b) are surrounding rock displacement distribution diagrams of the No. 600 level roadway before and after excavating and timbering of a tunnel. In accordance with those diagrams, it can be seen that after roadway excavation, two-sided surrounding rocks extrude and move towards the roadway with a relatively large two-sided displacement. However, thanks to the adoption of shotcrete and bolt support, such a displacement modestly decreases to a certain degree. Such a phenomenon indicates that currently utilized support pattern has a poor control effect over surrounding rocks due to a principal reason of wall caving which is formed in the roadway. The fact that vertical displacement is distributed symmetrically and the floor heave capacity is greater than roof subsidence is basically consistent with field measured results.

4.3.3. Distribution of plastic zone in surrounding rocks

The plastic zone distribution in surrounding rocks of the No. 600 level main haulage roadway before and after excavating and timbering of a tunnel is shown in Figure 7. It can be seen from such a diagram that, after roadway excavation, the plastic zone in floor is the largest with a width of about 5 m due to soft surrounding rocks; in addition, widths for two-sided plastic zone and roof are around 4 and 3 m respectively. After shotcrete and bolt support, reduction in the plastic zone is obvious the plastic zone in floor is the largest with a width of about 2 m, widths for two-sided plastic zone and roof are around 2 and 1 m respectively, indicating that support resistance is the main factor impacting surrounding rock plastic zone. Therefore, with the increase in such a resistance, the plastic zone is gradually degraded. In case that support is not provided timely, the plastic zone may tend to keep expanding and the surrounding rock in original plastic zone enters into a broken state followed by its deformation in a manner which is much more serious than rocks in the plastic zone. Since roadway support, as the plastic zone has a tendency of gradual diminishing together with the increase of support resistance, the phenomenon of stress concentration can be improved and surrounding rock deformation is also relatively decreased.
5. Scheme design and selection

5.1. Coupling support principle for shotcrete rockbolt mesh and surrounding rock

Coupling of the bolting mesh and the surrounding rock is crucial to the stability of roadway. Excessively strong or weak shotcrete and bolt support is able to lead to local stress concentration which further gives rise to roadway damages. Only if both the strength and the rigidity of them reach the standard of coupling, their deformations can be mutually coordinated. When the stress concentration area in surrounding rock transfers and disperses to the low stress zone during compatible deformation, it signifies that they have been up to the coupling. As a result, an optimal support effect is achieved.

In line with researches on numerical simulation, at the early stage of roadway construction, stress at the top of roadway surrounding rock is concentrated rapidly making it a hazardous area of roadway collapse. After the implementation of bolting net coupling support, as the stress concentration area on roof falls in a rapid manner and the stress state for sided low stress area is enhanced promptly, stress states of different parts in the entire surrounding rock are inclined to be homogenized. Therefore, after the bolting net coupling support is put into practice, surrounding rock support status can be changed from an open environment to a closed mechanical environment and the stress concentration area in surrounding rock also transfers and disperses towards the low stress area; i.e. the whole process of stress dispersion homogenization is automatically realized by bolting-net coupling design. Together with the transformation of surrounding rock stress from the concentration area to a low stress area, the bolt stress tends to be homogenized, so are stress and strain fields in the surrounding rock.

5.2. Optimization selection of support scheme

According to physical conditions in this mine, combined with theoretical and numerical analysis, a pattern of shotcrete rockbolt mesh coupling support has been employed. For specific parameters put forward in all support schemes, please refer to Table 2 and Figure 8. Rockbolts with the diameter of
20 mm were selected. The main mechanical parameters are: the density $\gamma = 7.8 \text{ t/m}^3$, the elastic modulus $E = 214 \text{ GPa}$, the yield force $P_s = 114 \text{ KN}$, the average breaking force $P_b = 171 \text{ KN}$, and the elongation rate of 21.3%. Spray layer thickness is 100 mm.

Based on the Flac$^{\text{TM}}$ numerical analysis method, all support schemes are simulated and corresponding simulation results can be seen in Figures 9–11. Figure 9 is distribution of maximum

### Table 2. Shotcrete rockbolt mesh coupling support scheme

| Schemes | Support parameters |
|---------|--------------------|
| Scheme 1| Spray thickness, 120 mm; Length of grouting rockbolt, 3 m; Inter-row spacing, 900 mm × 900 mm; Length of slit wedge tubing bolt, $L = 1,500$ mm; Interval × Pitch = 2,000 mm × 1,000 mm; Bar-mat reinforcement selected for protecting wire net, A3 carbon steel as material, 6 mm as bar diameter; Net dimension, 150 mm × 150 mm; Meshes, 2,000 mm × 2,000 mm |
| Scheme 2| Spray thickness, 100 mm; Length of grouting rockbolt, 2.5 m; Inter-row spacing, 900 mm × 900 mm; Length of floor anchor, 3 m; Interval, 1,000 mm; Bar-mat reinforcement selected for protecting wire net, A3 carbon steel as material, 6 mm as bar diameter; Net dimension, 150 mm × 150 mm; Meshes, 2,000 mm × 2,000 mm; Protecting wire net is fixed by ground anchor or short bolt with a length of 40 cm, arranged as a quincunx |
| Scheme 3| Spray thickness, 100 mm; Length of grouting rockbolt, 2.5 m; Inter-row spacing, 900 mm × 900 mm; Length of floor anchor, 3 m; Interval, 800 mm; Bar-mat reinforcement selected for protecting wire net, A3 carbon steel as material, 6 mm as bar diameter; Net dimension, 150 mm × 150 mm; Meshes, 2,000 mm × 2,000 mm; Protecting wire net is fixed by ground anchor or short bolt with a length of 40 cm, arranged as a quincunx |
| Scheme 4| Spray thickness, 100 mm; Length of grouting rockbolt, 2.5 m; Inter-row spacing, 700 mm × 700 mm; Length of floor anchor, 3 m; Interval, 800 mm; Protecting wire net is fixed by ground anchor or short bolt with a length of 40 cm, arranged as a quincunx |
Seen from above results, while no control over the floor is presented in Scheme 1 which is able to lead to relatively large floor heave capacity, Schemes 2–4 adopt the floor anchor to get command of roadway floor heave and have generated significant effects. Nevertheless, the selection of bolt parameters must be performed reasonably based on a theoretical calculation by relying upon physical field conditions and comprehensively taking the relation between factors of the bolt into consideration.

5.3. Economic benefit analysis
As for primary differences between the new and the original support schemes, they can be described from the following aspects. Regarding the former, the adoption of floor anchor control over floor heave is able to increase support density and reduce the shotcrete thickness; moreover, the net is

### Table 3. Surrounding rock stress displacement comparison for each supporting scheme

| Schemes | Maximum stress (MPa) | Minimum stress (MPa) | Maximum vertical displacement (mm) | Maximum horizontal displacement (mm) | Ranges of plastic zones (mm) |
|---------|----------------------|----------------------|-----------------------------------|-------------------------------------|-----------------------------|
| Scheme 1 | 9.78                 | 7.25                 | 94                                | 38                                  | 4,370                       |
| Scheme 2 | 8.31                 | 6.27                 | 75                                | 28                                  | 3,659                       |
| Scheme 3 | 6.75                 | 4.86                 | 32                                | 17                                  | 3,273                       |
| Scheme 4 | 4.23                 | 2.94                 | 19                                | 11                                  | 2,758                       |
suspended by short anchor instead of slit wedge tubing rockbolt. As a result, the economic comparison between such schemes is carried out in allusion to their differences. Main expenses of the old and new support schemes can be seen in Tables 4 and 5.

In comparison with the original support scheme, the new one can be adopted to save a direct support cost of 46.70 dollars/m which is 79.3% of that of the original scheme. According to current
maintenance situation in such a mine, no anticipated renovation is required within 10 years if the new support scheme is employed; however, if the original one is used, it needs to be renovated for 3 times at least. Considering that labor charges for one renovation are 239.88 dollars/m, renovation expenses saved in 10 years by means of the new scheme can be calculated as follows provided that an area of 100 m is repaired each time.

\[
46.70 \times 100 + (225.56 + 239.88) \times 100 \times 3 = 144302 \text{ dollars}
\]

On this basis, the new support scheme has the capability to create huge economic benefits. Within the No. 600 level roadway of this mine, a 46 m long testing roadway which has the same geological condition with it is chosen to perform industrial tests for those four schemes, industrial test results of the four schemes are shown in Figure 12, deformation of Scheme 1 is the most obvious, then is Scheme 2. Top plate of Scheme 4 is the most smooth, and the support effect is the most ideal.

![Figure 12. Roadway support effect sketch of each scheme.](image-url)
Moreover, surrounding rock deformation is monitored and relevant monitoring results have been shown in Figures 13 and 14.

(1) According to monitoring results of surrounding rock deformation in the roadway, it can be seen that for those four support design schemes, the two-sided displacement of them is greater than their roof subsidence; among which, as no effective control over floor is put forward in Scheme 1, its deformation is the greatest with a two-sided displacement of 54 mm, a roof subsidence of 25.4 mm and a floor heave capacity of 86 mm. In other words, the two-sided displacement takes a proportion of 1/3 in the whole deformation and the floor heave capacity occupies about half of it. Regarding Scheme 4, it has the minimum roadway surrounding rock deformation with a two-sided displacement of 29.2 mm, a roof subsidence of 21.2 mm and a floor heave capacity of 18.1. Comparing with another three schemes, surrounding rock deformation in the roadway presented in Scheme 4 has enormously decreased; especially when it is compared with the original support design, it is obvious that the two-sided displacement, the roof subsidence and the floor heave capacity have all reduced 80, 70 and 90% and above separately.

(2) Monitoring results show that deformation rate of surrounding rock in the roadway reaches the maximum value after 3–7 days when mortar bolt is installed. In detail, the rate obtained based on Scheme 1 is the largest while the smallest for Scheme 4. On the 40th day or around, such a deformation tends to be relatively steady. Hence, it can be determined that the second shotcrete should be implemented on about 40th day after the installation of mortar bolt.
(3) Based on the comparative analysis of surrounding rock deformation monitoring results acquired from four support schemes, the fourth one is able to generate an ideal control over the deformation. Therefore, it is selected as the optimal scheme.

6. Conclusions
In this paper, a No. 600 level soft rock roadway in an underground mine is taken as an instance to conduct researches on strength response characteristics of surrounding rocks in the soft roadway both after and before shotcrete and bolt support based on the finite difference method. Then, force-deformation is analyzed for surrounding rocks after roadway excavation so as to put forward a shotcrete rockbolt mesh coupling support pattern and design four coupling support schemes. Through the comparative analysis between actual engineering monitoring and numerical simulation, main conclusions can be drawn as follows.

(1) When the load in floor strata reaches a certain value, shear failure firstly takes place on the edge of floor; then, with the continuous increase of it, shear fracture zones are correspondingly expanded. Moreover, if the load keeps going up, those zones can be connected together into one big region subsequently; at that time, it is indicated that the load on floor strata has reached its ultimate bearing capacity and hence it is on the verge of being damaged. Then, with a slight increase in such a load, the floor could be settled sharply so that general shear failure happens to the floor strata and floor heave is also generated by the uplift of floor rock mass.

(2) Based on field monitoring on surrounding rock deformation achieved by four support schemes, it is found that the control over surrounding rock deformation carried out in line with Scheme 4 is the most ideal one. By contrast to the original support design, reductions in two-sided displacement, roof subsidence and floor heave capacity respectively are all above 80, 70 and 90% so that the stability of roadway is substantially improved. As a result, the field monitoring results are in conformity with numerical analysis summary.

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Author details
Huang Jun-jie
E-mail: 2051321767@qq.com
Qin Ya-guang
E-mail: csuqyg@163.com
ORCID ID: http://orcid.org/0000-0002-0733-9600
Zhao Shuang
E-mail: 1446695356@qq.com
Wang Wei
E-mail: 305334943@qq.com
Wen Lei
E-mail: 1446695356@qq.com

1 School of Resources and Safety Engineering, Central South University, Changsha 410083, PR China.

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