A kind of control technology for squeezing failure in deep roadways: a case study

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ABSTRACT

Under high horizontal ground stress, the squeezing failure has been a common failure pattern for deep roadway. And the large deformation of surrounding rock mass around the roadway has also been a major challenge for deep mining practices. This paper describes a case study of the failure mechanisms and stability control technology of deep roadway under high horizontal ground stress in Jinchuan mine in Gansu Province, China. And this paper also aims to propose a kind of support strategy to prevent the plastic slippage around the floor of roadway. This study started with an in situ broken zone measurement programme to determine the depth of broken zone at each part of the roadway. Combined with long bolt and floor beam, an improved support strategy has been proposed. The field test results show that the improved support strategy can successfully solve the problems of side wall collapse and floor heave. It can significantly restricts rock mass large deformation and avoid frequent repair. Compare to the original support, the improved support strategy can greatly save investment of mines and also has good application value.

KEYWORDS

Deep roadway; high stress; severe deformation; floor heave; improved support

1. Introduction

With rapid economic developments, the global demand for mineral resources is increasing. After many years of exploitation, shallow mineral materials have dried up in many areas of the world, thus deep mining engineering is the inevitable trend of the future (Diering 1997; Gurtunca and Keynote 1998; Li et al. 2001). Currently, underground coal and metal mining has become deeper and deeper in order to keep pace with the increasing demand for mineral materials worldwide (Jiao et al. 2013). For instance, the depth of the Anglo gold mine in the Republic of Ghana has reached 3700 m, and there are three gold mines whose depths are over 2400 m in Kolar, India. In addition, the depths of nonferrous metal mines are over 1000 m in Canada, Australia, the United States and many other countries.

With increasing mining depth, the deformation of rock masses becomes more and more serious (Vogel and Andrast 2000; Li 2010), especially for excavation activities under high in situ stress, where the failure associated with squeezing conditions often results in significant problems for roadway design and support. In the case of deep mining, the surrounding rock mass exhibits the characteristics of high in situ stress, as well as large and long-duration deformation (Xie et al. 2015). Many roadways may experience a number of engineering disasters such as roof caving, floor heave and side wall collapse, which increase the security risks of deep roadways. As roadway maintenance costs grow, they significantly restrict the operations of the mines. Therefore, the instability of deep...
roadways has been a focus in the fields of mining and rock engineering (Hoek et al. 1995; Wang and Wang 2000; Jiao et al. 2013; Shen 2014).

Previous research primarily used field experiments and model testing. In one field experiment, Shen et al. (2016) proposed a method to improve stress conditions and control the deformation of surrounding rock masses. Wang and Wang (2000) presented a case study to demonstrate the application of the outlined principles for stability control of roadways excavated in soft rocks using a multiple-support system with joints, and demonstrated that this system can maintain the stability of a roadway well. Combing with backfilling chemical grouting material and yieldable U-shaped steel sets, Jiao et al. (2013) provided a practical strategy to support the roadways in a loose, thick coal seam, and on-site monitoring indicates that the strategy can accelerate the stability of roadways. Lin et al. (2015) used model tests to investigate the failure behaviour of the large tunnel in the Jinping II hydropower station. Based on theoretical analysis and physical simulation tests at the Xinzhanhai No. 1 coal mine, Meng et al. (2013) analysed the failure mechanism of a large section of roadway in soft rock and proposed support technology to prevent the shear slip in surrounding rock mass.

Although many kinds of support methods and technology have been proposed and achieved good effects, existing support strategies are not completely suitable to other types of engineering, which have their own geological conditions. The failure mechanism and support technology for squeezing failure in deep roadways has rarely been studied, especially for the squeezing failure in the floors and lower ribs of roadways. In regions of high horizontal stress, squeezing failure is quite serious. Floor heave is obvious, and severe cases can lead to instability and even failure of a whole roadway. However, support types like yieldable U-shaped steel, shotcrete lining and bolts do not create better control of squeezing deformation in the floors and lower ribs of roadways. Currently, in many mines, floor grouting is used to control the deformation of the floor heave, and it can improve the rock mechanical properties of roadway floors. However, numerous engineering practices have proved its poor effect on squeezing failure (Wang and Feng 2005). Moreover, in ultrabasic rock, where water invading rocks result in strength deterioration and expansion of volume (Ma et al. 2016, 2017; Zhou et al. 2016, 2017), the application of grouting support is not recommended (Wu et al. 2003; Fu et al. 2011). Therefore, it is very necessary to study the failure mechanisms and support technology of surrounding rock masses around floors and lower ribs. Based on analysis and field experiments, this paper reports a case study on deep roadway stability at the Jinchuan mine in Gansu Province, China, where the roadways are suffering from severe floor heave and side wall collapse. First, the depth of the broken zone of the roadway has been measured through acoustic tests. Based on the test results, the bolt parameters, including length and layout angle, have been improved. To prevent plastic slippage around the floor of the roadway, a floor beam with a new kind of layout has been used to prevent squeezing failure. The field test results show that the new support technology controls the surrounding rock convergence well and saves mining investment capital.

2. Engineering background and the description of problems

2.1. Engineering geology

Jinchuan Group Co. Ltd., the largest nickel production base in China, occupies an important position in China’s economy. The Jinchuan mine is located in Gansu Province, China (Figure 1), where the terrain, at an altitude of 1500–1800 m, is relatively flat, and there is a temperate continental climate. The Jinchuan mine can be divided into four sections, mining areas I, II, III and IV.

In mining area II, the uniaxial compression strength of most rocks is larger than 130 Mpa. However, there are many joints and discontinuities in the rock mass, which is in a broken state after
cutting by various structured surfaces (Figure 2). Under this broken state, the mechanic properties of the rock mass degrade severely. In addition, mining area II contains many ultrabasic rocks, such as Lherzolite, which means that grouting cannot be applied to reinforce the surrounding rock.

Moreover, as mining and underground constructions migrate to deeper grounds, the \textit{in situ} stress also escalates, especially in the form of horizontal ground stress. To understand the distribution of stresses in a mine, \textit{in situ} stress measurement is carried out by a hollow inclusion gauge method. The results from successful stress measurements (mining area II) at different depths are shown in Figure 3. $\sigma_v$ represents the vertical stress, and $\sigma_{h\text{max}}$ and $\sigma_{h\text{min}}$ represent the maximum and minimum value of horizontal \textit{in situ} stress, respectively. As can be seen from Figure 3, the \textit{in situ} stress exhibits a good linear relationship with depth, and it can be confirmed that horizontal stress is the maximum principle stress in this mine.

Based on the data in Figure 3, the calculated stress–depth relationships are expressed as follows:

\begin{align}
\sigma_{h\text{max}} &= 0.03595x + 4.13994 \\
\sigma_{h\text{min}} &= 0.01783x + 2.7661 \\
\sigma_v &= 0.01484x + 1.72985
\end{align}

where $x$ represents the mining depth in metres. The calibration of pre-mining stress in this study is performed on the basis of stresses calculated from Equations (1)–(3). $\sigma_{h\text{max}}$ is more than 40 Mpa when depth reaches 1000 m, and $\sigma_v$ stays around 17 MPa at a 1000 m depth. Under high \textit{in situ} stress, the side wall, roof and floor suffered severe damage, and the roadway needed to be repaired every quarter.
2.2. Patterns of roadway failure under high in situ stress

The mechanical properties of rock masses degrade severely; under high ground stress, the deformation of rock masses becomes more and more serious, and the surrounding rock masses exhibit obvious squeezing failure characteristics. In Figure 4 are the examples of squeezing failure in the Jinchuan mine. Some of the roadways have wall-to-wall convergence in excess of 30%, and floor heave convergence up to 1.2 m has been recorded. In Figure 4(a), the original support method of the roadway comprises U-shaped steel sets, fibre-reinforced concrete and a bolt. Under high ground stress, the lower rib of the roadway convergence is obvious. Fibre-reinforced concrete cracking and shedding, as well as bending deformation and tensile failure, occurs at the bottom half of the U-shaped steel. Along with the squeezing failure at the lower rib of the roadway, the floor heave is also very serious.

In Figure 4(b), the roadway is also supported by U-shaped steel sets, fibre-reinforced concrete and a bolt. There is no shedding in the fibre-reinforced concrete. However, a large block has been squeezed out from the lower rib of the roadway by horizontal stress. With the convergence of the side wall, the U-shaped steel has been snapped by the tension.

For the Figure 4(c), along with the U-shaped steel sets, fibre-reinforced concrete and bolt, the floor of the roadway has been reinforced by concrete. After failure, the large deformation mainly concentrates on the side walls and floor. As shown in Figure 4(c), the side walls move to the free face, and the wall-to-wall convergence almost reaches 50% of the spacing of the roadway. At the same time, the concrete on the floor up-warps and a tension fracture appears around the median line of the floor. Finally, the broken stones squeeze out from the floor of the roadway and result in a failure pattern in Figure 4(c).
Figure 4. Examples of squeezing failure of deep roadway in Jinchuan mine.
Overall, the repairing cycle of deep roadways in the Jinchuan mine is 3–6 months. To provide a safe environment for workers and maintain high productive efficiency for the mines, special measures to prevent the occurrence of plastic sliding and excessive floor heave are needed.

3. New roadway support design

3.1. Bolt parameter optimization

The field investigation found that the deep roadway of Jinchuan has been severely damaged. Based on field observation, some of the bolts at the lower rib have not been yielded (Figure 5). They have been pulled out and moved to the free face with rock blocks, especially for the spandrel and lower ribs of the roadway. The reason for this phenomenon is two-fold: first, under high horizontal ground stress, the large blocks at the roadway rib move to the free face. Second, the length of the bolt at the corner of the roadway was not anchored into the stable rocks; some of them remained in the broken zone and did not form an effective reinforcement.

To guarantee the effectiveness of the bolt in supporting rock deformation, the depth of the broken zone should be determined before a reasonable length can be decided. These measurements were conducted as a part of this study. The broken zone measurements took place in the transportation roadway numbered 958 at two locations (M1 and M2) (Figure 6(a)). The acoustic test method was used, and measurements were taken at each location in the roof, side wall and floor. The testing and data analysis process is shown in Figure 6(b). The acoustic wave test is based on the theory of elastic wave propagation in solid media (Li et al. 2017). There is one launch probe and two receiver sensors on the test bar; a sound wave is emitted from the probe and received through the receiver sensors. The wave velocity can then be calculated through the time gap between the first and second receiver sensors. Before test, there is some preparatory work that should be completed. First, holes should be drilled along the boundaries of the roadway as shown in Figure 6(b) (1-1 to 1-8). Then, sound waves should be launched at a distance with a manual shaking method (the test footage is 150–200 mm) and the modulated sound waves are received through two receiver sensors (Figure 6(b)). During the testing process, using water as a coupling agent, the depth of the broken zone can be confirmed after analysis of the wave speed along the test boreholes.

Generally, wave velocity decreases along with fracture development, and it increases along with rock mass stress and density (Barton 2007). The acoustic wave curves of the surrounding rock mass of a deep roadway can be classified into three categories (Li et al. 2017). As shown in Figure 7(a), for region A, because the rock mass near the boundaries of the roadway is heavily damaged, the wave decreases along with fracture development, and it increases along with rock mass stress and density. Therefore, the acoustic wave curves in this region are classified as type A.
velocity is lowest. Region C represents the deeper zone where the rock mass is disturbed but not
damaged. Because most of the original joints are closed, the wave velocity in this region is highest
and presents only minor fluctuation. Region B is between A and C, and the wave velocity in this
region is higher than in region A but with obvious fluctuation. Based on the acoustic wave test, the
depth of the broken zone is the length of regions A and B (Deng et al. 2001).

Figure 7(b,c) shows the mean wave velocity at different depths of the surrounding rock mass in
section M1 and M2, respectively. It can be seen from the figures that the wave velocity exhibits an
increasing trend along with test depth, and the items lower velocity and rise with fluctuation are eas-
ily distinguishable. Point c in Figure 7(b,c) represents the wave velocity catastrophe point, and indi-
cates that the wave velocity reaches the region C of Figure 7(a). As mentioned above, the depth of
the broken zone is the length of regions A and B. Because the length of the test probe is 200 mm, so
the depth of the broken zone is the absissa values of the wave velocity catastrophe point c (Figure 7
(b,c) minus 200 mm). Based on the measurement results, it was confirmed that, at the periphery of
the roadway, the depth of the broken zone was close to the length of a conventional bolt (2250 mm)
or deeper. At section M1 (Figure 8(a)), the depth of the broken zone at the roadway spandrel and
corner already reaches 2400 and 2450 mm, respectively. It is more than the length of a conventional
bolt, and under high ground stress, the bolt will move to the free face with the broken rock mass.
Section M2 (Figure 8(b)) exhibits similar characteristics with M1, and the depths of the broken zone
at the spandrel and the corner are also higher than other parts of the roadway.

According to the measurement results and the actual situation of the project construction, the
bolt length of the spandrel and lower ribs was adjusted to 3000 mm, and the bolt could anchor into
the stable rocks and form an effective reinforcement. But for the roof and side wall, the depth of the
broken zone is around 1800 mm, and the original bolt is long enough to anchor into the stable rock
mass. At the same time, as mentioned in Section 2.2, the failure of the roadway is mainly concen-
trated in the floor and lower rib. Based on consideration of maintenance costs, the bolt on the roof
and sidewall remain unchanged. Moreover, the bolt at the corner of the roadway is at an angle of
approximately 45 degrees with the horizontal plane. The bolt would prevent plastic slippage and
restrict the floor heave to some extent (Jiang et al. 2009; Wu et al. 2011; Guo et al. 2012; Liu et al.
2012). The proposed design includes six original bolts and two long bolts in the roof, as well as three
original bolts and one long bolt in each side wall. The long bolt has a length of 3000 mm and the
original bolt has a length of 2250 mm; both are 25 mm in diameter, as shown in Figure 9.

3.2. Controlling for large deformations near roadway floor

As mentioned above, high ground stress resulted from a lack of effective support to prevent the
squeezing failure around the floor of the roadway, which means that overall failure was unavoidable
as a consequence of side wall collapse and floor heave. In the improved method, to prevent plastic slippage around the floor of the roadway, a floor beam with a new kind of layout has been used to limit squeezing failure. Figure 10 shows the schematic diagram and the details of the layout of the floor beam in the improved support method. Steel beams made of ordinary carbon round steels...
(16 mm thick, 219 mm outer diameter, 4500 mm long) were placed in the middle of the groove. The compression stability coefficient was 0.88, the yield strength was 238 Mpa and the axial bearing capacity was 2200 KN.

As can be seen from Figure 10(a), the floor beam has been laid out in a groove, and both ends of the beam contact with surrounding rock via a sub-plate. The maximum groove depth is 600 mm, and the gap between the floor beam and groove has been filled up by gravel. Even if the floor experiences a degree of uplift, because of the fluidity of the gravel, the floor beam will not come in contact with the rock mass directly (Figure 10(b)). The load force from the bottom of the floor beam has been eliminated, and it also contributes to the stability of the floor beam under high ground stress.

The comparison between original and improved support methods is shown in Figure 11. As shown in Figure 11(a), the roadway was supported by a bolt, fibre-reinforced concrete and U-shaped steel sets. Seventeen $\Phi 25 \text{ mm} \times 2250 \text{ mm}$ bolts were installed in the roof and side wall with row spacing of 800 mm. The fibre-reinforced concrete was double-deck and 200 mm thick. The row spacing of the U-shaped steel was 600 mm, and the floor of the roadway was also reinforced by pouring concrete.

In the improved support strategy, 12 conventional bolts ($\Phi 25 \text{ mm} \times 2250 \text{ mm}$) were installed in the roof and side walls with row spacing of 800 mm. Two long bolts ($\Phi 25 \text{ mm} \times 3000 \text{ mm}$) were installed in the spandrel and corner of the roadway. As the bolt at the spandrel and lower ribs has

![Figure 8. Broken zone measurement results.](image)

![Figure 9. Improved bolt layout diagram.](image)
been replaced by a long bolt with a length of 3000 mm, the rock bolt can anchor into the stable rocks and form an effective reinforcement. For the U-shaped steel sets, the row spacing was adjusted to 800 mm. As can be seen from Figure 11(b), the roadway floor has been supported by a floor beam, and the row spacing is 1600 mm. Moreover, in the improved support strategy, the double-deck, fibre-reinforced concrete has been replaced by a single layer with a thickness of 150 mm. The component consumption of the two support methods is listed in Table 1.

4. On-site experimentation

To test the effectiveness of the improved support method and compare it with the original one, a field industrial experiment and monitoring were carried out in the transportation roadway of the 958 working face in the Jinchuan mine. Considering the differences of the lithology for the surrounding rocks of the roadway, a 50-metre-long section of roadway was chosen as a trial of the improved support, while the remaining part of the roadway was supported by the original method. The improved support method was tested as shown in Table 1.
The improved support method was implemented according to the design drawings (Figure 11(b)) and supporting parameters (Table 1), and the layout of the steel beam is presented in Figure 12. As can be seen from Figure 12, the plate of the steel beam was in close contact with both ends of the side walls. To fix the steel beam, both ends of the beam would be filled with concrete, along with the gap between the plate and the side wall.

### 4.1. Performance of the new support system

The roadway stability status of different support methods and different periods is demonstrated in Figure 13(a,b), which shows the roadway stability of the original and new support methods, respectively. The experimental method was observed to produce a much better roadway profile than the original method. The floor heave of the roadway was obvious, as shown in Figure 13(a), photographed four months after the installation of the original support.

The roadway floor was lifted, which was clearly visible and accompanied by cracking. The investigation showed the floor heave reaching 900 mm. In addition, there were many cracks in both walls, especially in the lower rib of roadway. Serious convergences occurred and resulted in cracks produced in the fibre-reinforced concrete.

In contrast, the roadway maintained in a good condition is shown in Figure 13(b), photographed 12 months after installation of the improved support. The deformation of the surrounding rock was

| Support method  | Bolt Length  | Bolt Spacing | Thickness of fibre-reinforced concrete | Steel arch spacing | Steel beam spacing |
|-----------------|--------------|--------------|---------------------------------------|-------------------|-------------------|
| Original support| 2250 mm (17) | 800 mm       | 200 mm (double metal meshes)          | 600 mm            | –                 |
| Improved support| 2250 mm (12) | 800 mm       | 150 mm (single metal meshes)          | 800 mm            | 1600 mm           |

The improved support method was implemented according to the design drawings (Figure 11(b)) and supporting parameters (Table 1), and the layout of the steel beam is presented in Figure 12. As can be seen from Figure 12, the plate of the steel beam was in close contact with both ends of the side walls. To fix the steel beam, both ends of the beam would be filled with concrete, along with the gap between the plate and the side wall.

![Figure 12. The layout of floor beam in field experiment.](image)
well-controlled by the new support, the floor showed no signs of the lifting phenomenon. Furthermore, the lower rib did not show obvious deformation or large-scale convergence. It is clear that the new support is more suitable for roadways excavated in such high in situ stress areas.

4.2. Measurement of surrounding rock deformation

The four monitoring sections of the roadway supported by the experimental section were separated by gaps of 12 m (Figure 14). This arrangement aimed to provide more reliable monitoring data, and the deformation of the roadway was measured within 1 mm of accuracy. At that time, the roadway working face was 30 m past the fourth monitoring section, and the average advance rate of the roadway was about 3 m per day. The monitoring content mainly comprised the convergence monitoring of the side walls and roof-to-floor monitoring. The monitoring point of the side wall was located at the middle of the two side walls, and the roof-to-floor monitoring point was located at the centre lines of the roof and floor.

Measurement for the field results of the improved programme lasted for nearly 10 months. The relationship between cross-section deformation and time after rehabilitation and installation of improved supporting system is shown in Figure 15(a,b). The curves of the deformation rate of the roadway over time are shown in Figure 16(a,b). As shown in Figure 15(a,b), the deformation–time curve of wall-to-wall and roof-to-floor can be divided into three stages. The first stage is labelled as
a rapid deformation stage, and the duration of this initial deformation stage was usually about 30 days. As can be seen from Figure 15(a,b), a rapid increase of the deformation took place in this stage. The deformation–time curve was similar to a straight line, and the results showed that the maximum convergence rate of roof-to-floor and wall-to-wall were 14.5 (Figure 16(a)) and 12 mm per day (Figure 16(b)), respectively, in this stage.

The second stage is the deceleration stage, and this stage usually lasted three to four months. In this stage, the overall deformation rate was on the decline, although the deformation rate of the surrounding rock showed obvious fluctuation (Figure 16(a,b)). Although there was an obvious time-dependent component to the surrounding rock deformation, the magnitude of side wall convergence and roof-to-floor deformation of the roadway at this stage was much lower than that at the previous stage.

In the last stage, the deformation of surrounding rock was very slow. After 280 days, the roof-to-floor convergences for sections #2, #3 and #4 were 174, 153 and 171 mm, respectively; on the other hand, the convergences between the two side walls for sections #1, #2, #3 and #4 were 169, 185, 177 and 181 mm, respectively. The rheological deformation could be perpetual and last as long as the roadway existed, but because effective measures were taken to support the roadway, the deformation rate for the side wall and roof-to-floor convergence was maintained at a lower value.

The long-term field monitoring programme revealed that the roof-to-floor rheological convergence rate and the side wall rheological convergence rate were steadily controlled at about 0.04 and 0.03 mm per day, respectively (Figure 16(a,b)). This indicates that the improved support method

![Figure 15](image1.png)

**Figure 15.** Surrounding rocks convergence versus time.

![Figure 16](image2.png)

**Figure 16.** Surrounding rocks convergence rate with time.
can contribute to the long-term stability of the transportation roadway. Although a certain degree of convergence in both the side walls and the roof-to-floor section existed, the convergence was within the permissible range since the roadway’s width ranges from 5000 mm to 5500 mm.

A comparison of material consumption per metre between the original support method and the improved strategy is provided in Table 2. As shown in Table 2, because of the adjustments of the support parameters, although there is a floor beam in the improved support strategy, there was a net cost reduction compared with the original method. The roadway supported by the improved support strategy retained stability after 12 months. However, the roadway with the original support method needed to be repaired almost every four months. Compared to the original support method, the improved support strategy can greatly save mine investments.

5. Conclusions

This paper described a case study on the failure mechanisms and support strategies of deep roadways under high horizontal ground stress. The transportation roadway of the Jinchuan mine in China was a typical deep roadway that experienced engineering disasters like side wall collapse and floor heave. The support components on the side wall moved with rock masses to free face, and there is obvious rock mass plastic slippage around the roadway floor.

An improved support strategy has been proposed. The bolt parameters have been improved according to the test results of the depth of the broken zone; the floor beam laid out via grooving has been used to restrict floor heave. For implementation and verification, a field experiment was conducted in the transportation roadway of the 958 mining level of the Jinchuan mine. The test results show that the improved support strategy can successfully solve the problems of side wall collapse and floor heave. It can significantly restrict rock mass large deformation and prevent the need for frequent repairs. Compared to the original support method, the improved support strategy can greatly save mine investments and has good application value.

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