Research on the Mechanism and Treatment Technique of Invert Floor Heave After the Penetration of Large Cross-section Tunnel in Slight Inclined Stratum

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Abstract. The newly built Chengdu-Guiyang Railway Gaopo Tunnel crosses through an interstratified rock mass of slightly inclined mud-sand. Sometime between the completion of the main structure and the construction of the monolithic ballast-less track, some sections began to bulge or even crack with a maximum deformation of up to 46 mm. At the same time, the vault concrete began to peel, causing significant risks in and challenges to the construction of the ballast-less track and the succeeding operations. In this study, the mechanism of and treatment techniques for invert floor heave following penetration of a large-cross-section tunnel into a slightly inclined stratum were investigated. In the study, the in-situ stress values and physical and mechanical parameters of the surrounding rock were obtained through field testing, and the extracted parameters were used in theoretical analysis based on discrete element numerical calculation to determine the applicable mechanism of invert floor heave. Finally, the implementation effects of different treatment techniques were compared and analyzed using the control variable method to identify final treatment measures. The results of this study reveal the following. (1) The heaving section of the invert floor in the tunnel area primarily comprised thin to medium-thick or interbedded mudstone and sandstone with argillaceous cement and was, in general, gently inclined. The joints of the rock mass were relatively well developed and broken. The effects of tectonic stress were not strong, and vertical stress was dominant. The swelling of the surrounding rock was weak. (2) The rock layer cracked as a result of the bending deformation of the layered surrounding rock in the unfavorable force direction. The argillaceous cement was lost or diluted in the presence of water. The weakening of the interlayer connection up to and including failure was an important contributory factor to the invert floor heave. (3) In treating the invert floor heave problems induced by joint surface weakening, the control effect of deepening the inversion was determined to be superior to those obtained by increasing the bolt length or strengthening the support stiffness. Based on these conclusions, a comprehensive management method involving the construction of a circular cavern using double linings and a 12-m long bolt was selected as a replacement treatment to effectively control the deformation. These results provide a reference for the design and construction of similar projects in the future.
1. Introduction

Geological settings with slightly inclined soft and hard rock interlayers are widely distributed in the northwest and southwest regions of China. With the development of the high-speed transportation network in the western part of the country, the construction of large-cross-section tunnels through slightly inclined strata has become an inevitable technical problem.

Soft rock is subject to large degrees of deformation at fast deformation speeds occurring over long time intervals[1]. As such, weak strata are not only prone to large degrees of deformation during construction but can also produce continuous deformation and invert floor heave that endanger the safety of the tunnel once it has been opened to traffic [2]. According to survey data, many opened railway tunnels have undergone various degrees of cracking, damage, invert floor heave, and other degeneration after varying periods of operation [3]. Lee et al. [4] enumerated primary examples of invert conditions worldwide and analyzed their causes, concluding that earthquakes, landslides, groundwater, high ground stress extrusion, and other factors were the primary sources of the invert condition. In tunnels, invert floor heave can cause track irregularities that can directly endanger driving safety and cause significant structural loss. The Guanjiao Railway Tunnel is cut through schist at a maximum burial depth of approximately 500 m. Shortly after opening to traffic, the bottom of the tunnel rose by 30 cm, causing transport to be interrupted [5]. The Muzhailing Tunnel on the Lanzhou–Chongqing Railway has suffered serious invert floor heave and upturn damage, with vertical cracks in the invert of up to 1–5 cm and hoop cracks generated in some local areas forming a through crack with the sidewall of the tunnel. The invert floor heave in the center of the tunnel was found to be 33 mm [6]. Similarly afflicted structures include the Wushaoling Tunnel of the Lanzhou–Wulumuqi High-speed Railway, the Jiazhuqing Tunnel of the Nanning–Kunming Railway, and the Baoziliang Tunnel of the Baoji–Zhongwei Railway. Hu et al. [7] analyzed the causes of tunnel invert floor heave and concluded that the main cause is poor lithology and low strength in the surrounding rock. In addition, surrounding rock tunnels that are not well drained can become seriously muddled and expand under the action of groundwater, further intensifying invert floor heave. Du et al. [8] used a combination of indoor model tests and extended finite element numerical simulation to study the three basic failure modes of invert floor heave under different loading modes. Shi et al. [9-10] studied the stresses on invert structures through on-site monitoring and theoretical analysis, analyzed the mechanisms of invert floor heave disease, and proposed reinforcement and rectification measures.

To effectively prevent and control invert floor heave, researchers in China and abroad have carried out a significant amount of research to produce a wide variety of invert floor heave prevention and control measures, which can be summarized into three major types: support reinforcement methods [11-12], pressure relief methods [13-14], and combined support methods. Support reinforcement, which includes the use of floor anchors, floor grouting, closed brackets, and reverse arching, is the most common means of strengthening floors. Pressure relief methods involve the transference of stress to deep surrounding rock through the application of technical approaches. It is usually carried out via slitting, drilling, and loose blasting. Combined support involves the application of a combination of support reinforcement and pressure when neither can achieve a desired effect separately.

In this study, theoretical analysis and numerical simulation based on on-site geological conditions were used to analyze the mechanism underlying invert floor heave. To remedy the condition in the Gaopo...
Tunnel, applications of the support and reinforcement method were explored and the application effect of each treatment measure was quantitatively calculated and compared to obtain an on-site change plan.

2. Project Overview

The Gaopo Tunnel is located at the junction of Heishu Town, Zhenxiong County, Yunnan Province and Heguantun Town, Bijie City, Guizhou Province; it runs across the provincial boundary and the watershed of the Chishui and Wujiang Rivers. The buried depth of the tunnel is 100–200 m, and the surrounding rock lithology is primarily sandstone, mudstone, carbonaceous shale, and bauxite interbedded with coal seams. The tunnel area is located in the fold belt of the Yangtze quasi-platform in the north of the Yunnan-Guizhou plateau, and its geological structure is complex. There are well-developed fractures and folds, broken-stratum rock, and a dominant east-west structure in which the line crosses the tectonic line at a large angle. At the beginning of construction, the Va design was applied; following serious deformation along with cracking and dropping of the initial support and distortion of the steel frame, measures such as deepening the arch and adding anchors at the bottom of the tunnel were adopted to ensure the safety of the coal-bearing stratum structure by primarily solving the problem of bulging and settlement at the top and bottom of the slightly inclined rock layer. Following deformation monitoring, the measures were judged to be effective and construction continued. Following additional tunnel penetration, it was found that invert floor heave and cracking occurred along part of its course and that the side wall was cracked, the arch concrete had peeled off of the block, and the steel bar was bent. The maximum invert floor heave was 46 mm.

2.1 On-site geological conditions

Excavation at D3K342+750–D3K342+810 revealed that the surrounding rock is argillaceous sandstone and mudstone intercalated with shale in thin to medium-thick layers even with the coal line. The rock layer is slightly inclined, the tunnel face is dry, and the rock mass is relatively soft. A photograph of the tunnel face is shown in Figure 1.

Excavation in sections D3K342+810–D3K343+160 revealed that the surrounding rock is mudstone interposed with shale and coal line in thin to medium-thick layers and that the rock layer trends toward the end of a larger mileage. Under the effect of the core of the Gaopo 1# anticline, the rock mass is severely squeezed, with well-developed joints and fissures, a relatively broken rock mass, a disordered local level, and poor stability. The mass is easily deformed, the excavation of the tunnel face is free of water, and the arch is easily collapsed. A photograph of the tunnel face is shown in Figure 2.

Excavation of D3K343+160–D3K343+169 revealed that at a dark grey sandstone is located approximately 3 m below the vault. This sandstone has medium-thick layers and is relatively complete and of sufficient stability. Beneath it is an interlayer development of mudstone, shale, and sandy mudstone with medium-thin layers in a nearly horizontal orientation. The rock is soft, and the tunnel face is free of water. A photograph of the tunnel face is shown in Figure 3.

![Figure 1. D3K342+790 Tunnel face](image1)
![Figure 2. D3K342+830 Tunnel face](image2)
![Figure 3. D3K343+160 Tunnel face](image3)
The rock surrounding the invert floor heave is distributed in soft and hard layers of primarily sandstone with interlayers of argillaceous cement, which have a dip angle of approximately 10° and are slightly inclined.

2.2 Ground stress test

In-situ stress measurement of six sections of hydraulic fracturing was successfully carried out in a hole at a depth range of 278.0–450.0 m. The fracturing curve recorded in each section was complete and standard in shape, each pressure parameter was relatively clear, and the repeatability was high. The maximum horizontal principal stress (SH) value within the test depth range was 7.0–15.0 MPa, while the minimum horizontal principal stress (SH) value range was 5.0–10.0 MPa. The estimated vertical stress (SV) value at each measuring section was in the range of 7.0–12.0 MPa. In general, within the test depth of the hole the minimum horizontal principal stress was equal to the minimum overall principal stress, the vertical principal stress (SV) did not differ significantly from the maximum horizontal principal stress, and the structural stress was relatively weak. Linear regressions of the stress-to-depth relationships in the hole are shown in Figure 4. The correlation coefficients of the maximum and minimum horizontal principal stresses are 0.868 and 0.897, respectively, indicating that the stress values are discrete with depth. As shown in Figure 5, the degree of slope is small and changes more regularly with depth. The direction of maximum horizontal principal stress (SH) changes from N50°W to N42°W to N54°W with increasing depth, indicating that the direction of the maximum principal stress near the measuring point is N49°W, while the line direction is approximately N51°W and approximately parallel to the direction of maximum principal stress. At the buried depth of the tunnel, the vertical and horizontal stresses are approximately 9 and 7.9 Mpa, respectively, and the lateral pressure coefficient is approximately 0.9.

![Linear regression curve of horizontal principal stress](image1)

(a) Linear regression curve of horizontal principal stress
![Variation of principal stress value with depth](image2)

(b) Variation of principal stress value with depth

Figure 4. Change in principal stress with depth

2.3 Expansion test

The primary special rock and soil sections of the tunnel passing through the coal stratum comprise mudstones and bauxite rocks. The maximum thickness per single layer is approximately 4 m, indicating expansive rock. A total of 32 groups of swelling rock samples were sampled from the main tunnels of the coal measure strata and the flat-guided surrounding rock for laboratory testing for swelling rock characteristics. The sampled lithologies included mudstone, carbonaceous shale, sandy mudstone,
argillaceous sandstone, and bauxite. A total of 16 groups were determined to be expansive rocks; most were weakly expanded, while a few were moderately expansive. Among the samples, the mudstone, carbonaceous shale, and bauxite had slightly higher proportions of expansive rocks and accounted for approximately 54% of the expansive samples, while the remaining lithologies accounted for a smaller proportion.

3. Analysis of floor heave of inverted arch

The field testing and analysis described in the preceding section revealed that the rock mass of the Gaopo Tunnel is gently inclined and relatively broken in its soft-quality components, which comprise the material foundation of the floor heave. The mechanical conditions of the post-excavation floor heave include high ground stress, tectonic influence, and stress redistribution; other important reasons for deformation include the softening effect of groundwater, weak expansibility of the surrounding rock, and construction activity.

Because rock mass comprises rock blocks and structural surfaces, the post-tunnel-excavation deformation can be divided into material and structural deformation. The slightly inclined interstratified rock mass has obvious structural characteristics, while the tunnel deformation follows a specific directional trend. In this section, we discuss how argillaceous cementation loss after the deformation of the tunnel structure led to a decrease in the contact stiffness between rock strata, which, in turn, led to the uplift of the tunnel invert.

3.1 Mechanism analysis of floor heave of invert arch

Figure 5 shows a geological sketch of the tunnel shown in Figures 1 to 3. It is seen from the picture that the layered distribution of sandstone and interbedded mudstone—with the mudstone serving as a contact cementation material—is relatively obvious and that there is an occasional coal seam.

Before analyzing the deformation of the tunnel structure, we examine the secondary stress state following tunnel excavation. To conduct this analysis, we adopt the elastic stress solution and obtain, via in-situ stress testing, a tunnel lateral pressure coefficient of approximately 0.9; for the convenience of analysis, the lateral pressure coefficient is set to 1. The stress value is the same at all angles, the shear stress is zero, and the radial and tangential stresses follow the distributions shown in Figure 6.

![Figure 5. Geological sketch of Gaopo Tunnel](image)

![Figure 6. Stress distribution in elastic rock surrounding circular tunnel excavation](image)

It is seen from Figure 6 that the stress state at or close to the boundary following tunnel excavation corresponds to a state of large confining pressure and small vertical pressure in which the tangential stress has a significant degree of influence on the surrounding rock. As a result of the divergence in stress directions, the structural plane in the figure can be divided into opening and closing deformations. It is seen from the schematic of structural plane deformation in Figure 7 that, in a post-excavation tunnel, the opening deformation occurs along the direction perpendicular to the tunnel and the structural plane, while the closing deformation occurs parallel to the structural plane.
Figure 7. Deformation diagram of joint surface

Figure 8. Deformation diagram of surrounding rock

An image of the deformation of the sandstone and interbedded mudstone rock mass as a result of the opening deformation of the structural plane is shown in Figure 8. The layered surrounding rock is cemented with mud, and the structural plane at the vertical plane has opened as a result of the post-excitation stress state, resulting in a decrease in the cementing force of the sandstone at both ends. Under the influence of groundwater, the mud will be further diluted or lost, leading to weakening or even failure of the interlayer connection.

The floor heave of an invert is caused by the weakening or failure of interlayer contact, which results in an enhancement of the anisotropy in the layered surrounding rock and, consequently, deformation of the structure in an unfavorable direction. This process does not occur instantaneously but gradually with the action of excavation and water. As such, the floor heave phenomenon can also occur in weakly expansive or non-expansive stratum following tunnel completion.

3.2 Numerical simulation analysis of floor heave of invert arch

In this section, we describe the use of the UDEC discrete element numerical calculation method to verify that decreasing the argillaceous cementation will lead to the formation of the upper drum of the inverted arch as a result of weakening of the structural surface stiffness. We also determine reasonable physical and mechanical indices for the rock layer based on a comparison of simulation results with on-site deformation data. Using these results, further analysis is carried out.

Because coal seams are not common in the surrounding rocks, the rock mass can be considered a pure mud-sand interbedded rock mass for simplicity. The rock strata parameters are shown in Tables 1 and 2. The vertical and horizontal ground stresses are assumed to be 9 and 7.9 MPa, respectively. The support form follows the original design, with a lining thickness of 30 cm, anchor length of 4 m, and support parameters as shown in Tables 3 and 4. The size of the model is 100 m (length) × 90 m (height). The rock layering is based on the Mohr–Coulomb constitutive model. The rock layer dip angle is 10° and the layer thickness is 1 m. The contact of the structural plane adopts the Coulomb slip criterion. The calculation model is shown in Figure 9.
Table 1. Rock mass calculation parameters

| Parameter                  | Value  |
|----------------------------|--------|
| Density (kg/m³)            | 2600   |
| Bulk Modulus (GPa)         | 13.7   |
| Shear Modulus (GPa)        | 5.74   |
| Cohesion (MPa)             | 5.5    |
| Friction (°)               | 35     |
| Tensile Strength (MPa)     | 2      |

Table 2. Structural plane calculation parameters

| Parameter                  | Value  |
|----------------------------|--------|
| Normal Stiffness (GPa/m)   | 5      |
| Tangential Stiffness (GPa/m)| 2      |
| Cohesion (MPa)             | 1.7    |
| Friction (°)               | 20     |
| Tensile Strength (kPa)     | 10     |

Table 3. Support calculation parameters

| Parameter                  | Value  |
|----------------------------|--------|
| Density (kg/m³)            | 2500   |
| ratio                      | 0.15   |
| Elastic Modulus (GPa)      | 23     |
| Compressive Strength (MPa) | 6      |
| Tensile Strength (MPa)     | 3      |
| Residual strength (MPa)    | 1.5    |

Table 4. Bolt calculation parameters

| Parameter                  | Value  |
|----------------------------|--------|
| Density (kg/m³)            | 7500   |
| Compressive Strength (MPa) | 630    |
| Tensile Strength (MPa)     | 5      |
| Elastic Modulus (GPa)      | 500    |
| Bond stiffness (GPa/m)     | 1.6    |
| Bond strength (Mpa)        | 2      |

Based on the determined formation parameters and in-situ stress, the appropriate boundary conditions could be determined by comparing the calculated and measured deformation data. Under the boundary conditions defined above, the structural surface parameters on the unfavorable side of the tunnel were weakened after the model calculation was stabilized and the deformation was further analyzed to obtain the deformation law following a period of excavation.

Application of the reverse analysis method revealed that the structural surface parameters of the balanced model were weakened to 1/10 of their pre-balanced levels, which served to simulate the argillaceous cement weakening caused by the opening and deformation of the structural surface. As
shown in Figure 10, the inverted arch uplift and deformation characteristics obtained under conditions equivalent to the equilibrium condition were close to the field-measured values.

**Figure 10.** Comparison of calculated and actual deformations

Figures 10 (a) and (c) show, respectively, the on-site failure and measured deformations. Figure 10 (b) shows a deformation cloud diagram, while Figure 10 (d) shows a vertical deformation diagram of the inverted arch monitoring point, in which region I corresponds to the displacement caused by excavation unloading and region II corresponds to the displacement caused by weakening of the structural surface. It is seen from Figure 10 (b) that, under such boundary conditions, the maximum arch settlement of the tunnel and maximum invert uplift are located, respectively, to the left and right of the center. The calculated deformation is consistent with the actual failure shown in Figure 10 (a). As seen in Figures 10 (b), (c), and (d), the calculated deformation is close to the measured deformation. As the inverted arch is applied later in the modeling, the calculated displacement of the inverted arch is equivalent to the full deformation displacement; Figure 11 shows a comparison of the latter half of Figure 10 (d) with the measured inverted arch uplift. Following completion of tunnel construction, the uplift of the tunnel is close to that of the deformation after the structural surface parameters have been weakened. These results demonstrate that the boundary conditions and calculation parameters applied in this modeling process credibly reflect the real deformation of the Gaopo Tunnel and that the weakening of the structural surface can explain the uplift phenomenon that occurred after the tunnel was completed.
4. Treatment measures for floor heave of inverted arch

In this section, we describe the use of numerical simulation to determine effective treatment measures for the floor heave of the inverted arch and propose a reasonable construction scheme for the Gaopo Tunnel based on the results. The models, parameters, and boundary conditions described in this section are the same as those used in Section 3.

4.1 Numerical simulation of treatment measures

The control variable method was applied to quantitatively calculate the treatment effects of the following three methods: 1) increasing the arch-span ratio of the invert, 2) increasing the support stiffness and thickness, and 3) increasing the length of the invert bolt.

1) Analysis of arch-span ratio

As altering the arch-span ratio will change the bolt setting angle, the bolt was omitted in this modeling process, and only the effect of changing the arch-span ratio on the treatment of the inverted arch uplift was analyzed. To model this treatment, the original designed tunnel invert was dug further into a circular tunnel under four working conditions, as shown in Figure 12.

2) Analysis of support stiffness and thickness
The five working conditions in terms of initial support stiffness and thickness are listed in Table 5.

| Number | Support thickness(cm) | Steel frame spacing(cm) |
|--------|-----------------------|-------------------------|
| Test 2-1 | 30                    | 80                      |
| Test 2-2 | 50                    | 80                      |
| Test 2-3 | 30                    | 60                      |
| Test 2-4 | 50                    | 60                      |

3) Analysis of the length of the invert bolt

The lengths of the bolt at the tunnel bottom was varied from 4 to 12 m under the five working conditions listed in Table 6. The bolt layout diagram is shown in Figure 13.

| Number | Length of invert bolt(m) |
|--------|--------------------------|
| Test 3-1 | 4                        |
| Test 3-2 | 6                        |
| Test 3-3 | 8                        |
| Test 3-4 | 10                       |
| Test 3-5 | 12                       |

4.2 Effect comparison of treatment measures

The primary focus of this paper is the secondary uplift of the inverted arch following tunnel completion, i.e., the floor heave caused by weakening of the structural surface. The inverted arch uplift of this section is equivalent to the displacement in region II in Figure 10 (d). Using the working conditions described in the preceding sections, the heave results were calculated at different arch-span ratios (Figure 14); different support stiffnesses and thicknesses (Figure 15); and different bolt lengths (Figure 16).
It is seen from Figure 14 that, as the invert deepens, the invert uplift of the tunnel decreases until the inverted arch uplift of the circular tunnel is only 64.36% of the original design value.

It is seen from Figure 15 that increasing the support thickness has an obvious effect in terms of inhibition of the invert uplift; decreasing the steel frame spacing and increasing the support stiffness, by contrast, have no obvious effect. When the support thickness is increased to 50 cm, the inverted arch uplift is reduced to 67.8% of its original value.

It is seen from Figure 16 that, when the bolt length is less than 8 m, increasing the length has no obvious effect on the suppression of inverted arch heave and that, while the effect at a length of 10 m is more significant than at 4 m, it is still relatively insignificant.

4.3 The scheme adopted at the construction site

Based on the conclusions described in Section 4.2, and in reference to the actual situation on site and in consideration of certain safety factors, a construction approach involving the excavation of a circular tunnel, 12-m-long anchor bolt, and application of a double-layer primary support excavation and support method was selected to rework the section of the Gaopo Tunnel with a high degree of deformation and invert uplift, as shown in Figure 17. The same boundary conditions and weakening parameters applied in Section 3 were applied to calculate the deformation under this working condition, as shown in Figure 18.
Relative to the original design outlined in Section 3, the vertical deformation is reduced from 15 to 7 cm, or 46.6% of the original design; the horizontal convergence (on both sides) is reduced from 18 to 10 cm, or 55.6% of the original design; and the secondary uplift is decreased from 4.5 to 1.8 cm, or 40% of the original design.

5. Conclusion

1) The Gaopo Tunnel area floor heave section was found to primarily comprise thin to medium-thick mudstone layers interbedded with sandstone with argillaceous cement, along with the occasional coal seams. The dip angle of the rock strata was found to be approximately 10–30°, corresponding to a slightly inclined condition. The joints of the rock masses are relatively developed and broken. The tectonic stress is relatively weak and primarily in the form of vertical stress. The expansibility of the surrounding rock is weak.

2) In tunnel areas with little tectonic stress and weak surrounding rock expansibility, the deformation of the layered surrounding rock in the unfavorable direction of stress causes the strata to open, and the resulting loss or dilution of argillite cement owing to water contact and the weakening or even failure of interlayer bonding are all important factors leading to floor heave.

3) For the relevant treatment measures for the floor heave caused by weakening of the joint surface, the following control effects were determined. Deepening the invert was found to have the best effect, while strengthening the supporting thickness was found to be superior to increasing the bolt length. Strengthening the supporting stiffness was found to have little effect on restraining the inverted arch uplift. The bolt has a significant effect on inhibiting the inverted arch uplift only when its length exceeds 10 m, and the inhibition of uplift is insensitive to bolt length at lengths below 10 m.

4) Through numerical calculation and analysis, it was finally decided that the large deformation of the high-slope tunnel and the upward arch uplift should be reworked by adopting a circular tunnel cross-sectional shape, using 12-m-long bolts, and applying a double-layer primary support excavation support method. After a trial implementation of this scheme, the monitored tunnel displacement was found to be sufficiently small.

5) For tunnel projects found to have similar geological and hydrological conditions as those occurring at the Gaopo Tunnel, it is suggested that the invert depth should be lengthened, the support thickness of the invert should be increased, and a long anchor or cable should be used to prevent the floor heave of the invert. By taking these measures, costs in terms of lost time and later reworking expense can be avoided.

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