Hindawi
Advances in Civil Engineering
Volume 2020, Article ID 5686042, 20 pages
https://doi.org/10.1155/2020/5686042

Research Article

New Construction Technology of a Shallow Tunnel in Boulder-Cobble Mixed Grounds

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Received 10 October 2019; Accepted 4 February 2020; Published 21 March 2020

Academic Editor: Sanjay Nimbalkar

As a typical granular bulk medium, problems are common in boulder-cobble mixed grounds, such as easy collapse and instability and difficult effective support for large-section tunnel excavation. Tunnels constructed in BCM grounds are rare still, and there is a big gap between the design and construction of tunnels. Based on the Nianggaicun highway tunnel crossing the BCM grounds, the construction technology of tunnel in BCM grounds is studied by means of literature investigation and field survey. Here are the main conclusions: the overall deformation of surrounding rock is quite small; the pressure distribution of surrounding rock is small and loose pressure is dominant, and the safety reserve of secondary lining is large. The deformation process of surrounding rock concentrates on the construction stage. During the construction process, there are many problems, such as serious overexcavation, difficulty of bolt penetration, and continuous rock fall. In this paper, a three-bench complementary cyclic excavation method is proposed, which replaces the original CD and CRD methods. Meanwhile, the supporting system is optimized. The results show that the disturbance of surrounding rock is reduced, while the safety of construction process and the reliability of structure are increased. The new excavation method and optimized supporting system are expected to fill the gap between design and construction of tunnel in BCM grounds and provide reference for construction of such tunnels in the future.

1. Introduction

With the development of China's infrastructure, highways and railways continue to be built and extended to high altitude areas in the west. Along with this development trend, there are more and more complex new engineering geological problems, which require us to constantly tackle the key problems of scientific research and technology and tackle the construction problems of complex technical geology in order to ensure the feasibility and sustainability of engineering construction. As new complex grounds, BCM formation has puzzled the construction of metro and mountain tunnel in recent years. Like sandy-cobble mixed grounds, it is formed by boulder under the action of flood transportation and long-term river erosion. Therefore, it exists widely in valley area and scouring plain. The skeleton grains are dominated by boulders and cobble, and the total content of their is more than 50%. The grounds consisting mainly of boulders soil and cobble soil are called BCM grounds. At present, most of the pervasive BCM grounds are formed in the Quaternary [1–6] and are widely distributed in some valleys of Beijing, Changsha, Chengdu, Lanzhou, and Tibet (Figure 1). BCM grounds are composed of granules consisting of boulders and cobble, coarse sand, and some clay. It has the characteristics of loose structure, strong permeability, rich water, and poor self-stability, but it does not easily deform. It is essentially different from the more complete continuous medium grounds such as rock mass and soil. The traditional calculation method of continuous medium tunnel lacks guidance for the determination of surrounding rock pressure and structural design of tunnels in boulder-

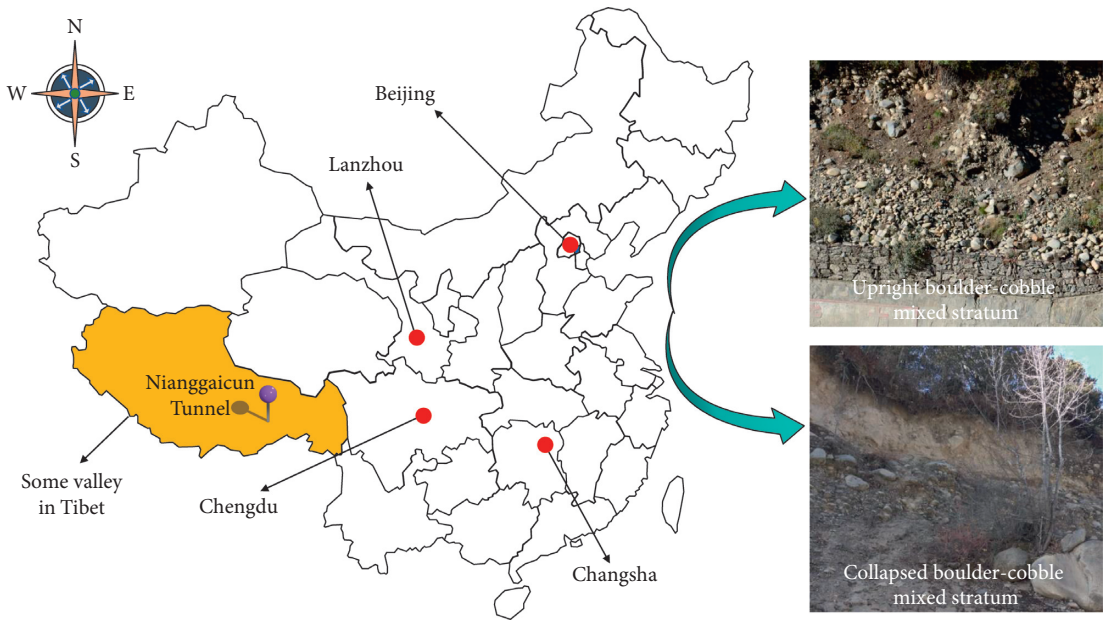


Figure 1: Distribution of boulder-cobble mixed grounds in China.

cobble mixed grounds. Traditional construction methods, such as NATM, are not suitable for the grounds without self-stabilization ability. The shallow excavation method emphasizes advanced support and prereinforcement of surrounding rock. The prereinforcement effect of BCM formation is very poor due to the collapse of rock surface and blockage of boreholes. In addition, when adopting self-propelled bolt reinforcement, it is easy to be hindered by large floated rocks and cobble, resulting in the problem of inaccessibility [7–9]. The failure modes of boulder-cobble mixed grounds include local collapse, arch collapse, and penetration collapse. In addition, there are some problems, such as lag cavity moving up, continuous progressive failure, and so on. The failure of surrounding rock can be summarized as stage, continuity, water sensitivity, and vibration sensitivity. After the excavation of the tunnel, with the redistribution of the surrounding rock stress, the rock on the wall began to fall continuously. Under the influence of seepage and vibration, the phenomenon of continuous rock fall and instability will be intensified and accelerated. If there is a cavity above the rear of the arch crown lining, the cavity will gradually move up with the extension of time and slowly develop to the surface. BCM grounds are similar to SCM grounds in distribution, formation, and material composition, but different in particle size composition, fine material composition, compactness, and cementation degree. At present, there are some tunneling projects successfully constructed in SCM formation, and some technical experience has been obtained. Even though the construction technology of shallow excavation in SCM formation is far from mature, there are a lot of blanks in the design theory and construction technology of tunnel in BCM formation [10–13]. It is an ideal way to solve the engineering problems encountered in the construction of BCM formation by referring to the experience accumulated in the construction of SCM grounds [14–20]. Murata et al. [21] summarized Maiko Tunnel in water-rich SCM formation. They believed that advance reinforcement and the reasonable construction dewatering method were necessary measures for tunnel construction in SCM grounds. Dang [22] tested the stress and surrounding rock pressure of supporting structure under the CD method and bench method in SCM formation of Chengdu. It was found that the surrounding rock pressure of the CD method was much larger than that of the bench method. It was pointed out that less disturbance should be considered first and strong support should be considered secondly in tunnel construction in SCM grounds. Funatsu et al. [23] used PFC2D software to analyze and evaluate the stability of tunnel surrounding rock under the action of advance pipe shed and advance bolt. Jinxing Lai et al. [24] studied the reinforcement test of surface sleeve valve tube for sandy-cobble mixed formation and determined the grouting parameters for the advance curtain grouting of tunnel excavation. As a typical granular bulk medium, BCM formation has problems such as easy collapse and instability and difficult effective support for large-section tunnel excavation. There are very few tunnels constructed in boulder-cobble mixed formation, and there is a big gap in the design and construction of tunnel construction [25–29]. It is necessary to study the difficult problems in the excavation of cobble tunnel and its construction technology. Based on the "Nianggaicun" highway tunnel crossing the boulder-cobble mixed grounds, the construction technology of tunnel in boulder-cobble mixed formation is studied by means of field survey and literature investigation.

Through the study of the tunnel, we summarize the difficulties of tunnel excavation in BCM grounds, optimizing the tunnel excavation and support method, and compare the tunnel construction technology in other granular media in order to sum up the tunnel construction technique which is

suitable for the water-rich BCM grounds. From the long-term effect, the research of this project can make up for the technical gap of tunnel construction in BCM mixed grounds and provide guidance and reference for similar project construction in the future.

2. Engineering Overview

2.1. Introduction to the Project. The Nianggaicun tunnel is a large section of the first-level highway tunnel, with a total length of more than 500 meters, designed to speed 80 km/h, and adopts the independent two-hole, single-hole two-lane standard. The whole tunnel is shallow, the dark excavation section is deep in 7 to 33 m, and there is a small net distance distribution in the hole body section; the minimum net distance is 16 m. The tunnel is located on the east side of the Niyang River on the second-level ground, through the Niyang River three-level terrace and mudslide accumulation area, the terrain is undulating. Boulder and cobble is weak glue or no bond. It is in a medium to dense state and rich in groundwater. The rock mass in the excavation area is in a dense state and surrounding rock is grade V. The geological section of the right line of the tunnel as shown in Figure 2. The tunnel is mainly located in the compacting BCM stratum, the upper part is covered with medium dense BCM stratum, and the top part is covered with cobble. These grounds are mainly composed of boulder and cobble, accounting for more than 20% of cobble, more than 40% of boulder, and overall content between 60% and 80%. According to the current "Geotechnical Engineering Survey" (GB/T 50021-2001, in Chinese) [30] and "Design of Building Foundation" (GB 50007-2002, in Chinese) [31], particles larger than 200 mm are defined as boulder and particles smaller than 20–200 mm are defined as cobble. In "Design of Foundation and Foundation of Highway Bridges and Culverts" (JTG D63-2007, in Chinese) [32] and "Geotechnical Engineering Survey" (GB/T 50021-2001, in Chinese) [30]: grounds with particle size greater than 200 mm and more than 50% of total mass are defined as boulder soil and grounds with particle size greater than 20 mm and more than 50% of total mass are cobble soil. Therefore, the grounds belong to the mixed grounds composed of boulder soil and cobble soil, defined as BCM grounds.

2.2. Surrounding Rock Overview. Geological survey report shows that cobble and boulder are the main types of grounds, and sandy gravel and muddy clay are the main cementing materials in the middle. In the actual excavation, it is found that the content of boulders in grounds is relatively large, and the particle size of floated stone can reach more than 10% above 0.5 m. The surrounding rock in some sections is saturated with water and has obvious local seepage. Fine material has high mud content and can be massaged by hand. Where seepage is obvious, the leaking stones are quickly washed away by seepage water. Figure 3 shows the condition of boulder and cobble on the excavation surface [33].

2.3. Tunnel Excavation and Support Design. Tunnels learn from the experience of general shallow-buried cobble formation tunnel design in excavation and support system design. The design diagram is shown as shown in Figure 4, mainly for the design profile as follows: CD method and CRD are widely used in weak formations, in order to control the deformation of surrounding rock, strengthen the surrounding rock constraints, tunnels in the XS-Va lining section design CD method excavation, in the XS-Vb lining section design CRD method excavation. The reserved deformation amount is taken by 15 cm. The tunnel inlet section is designed in accordance with the regular tunnel, and the large pipe shed is 108 mm in diameter, 30 m in length, and 40 cm in spacing. The empty slurry anchor in all designs of the hole body section is distributed in the upper 3.5 m range of the temporary support in the upper 3.5 m range at the top. The system anchor is a diameter 25 mm self-propelled hollow grouting bolt, and the feet reinforcement bolt is 3 m in length and 51 mm in diameter. The system bolt is 5.0 m in length in the direction of small proximity, 3.5 m in length on the other side, 1.0 m in spacing, and is arranged in plum shape [34].

3. Difficulties in Construction Site of Tunnel in Boulder-Cobble Mixed Grounds

3.1. Difficulties in Pipe Shed Construction in Advance. During the construction of the left tunnel entrance, the drill bit adopts eccentric bit 108 mm in diameter; impactor is 90 mm in diameter and drill pipe is 76 mm in diameter, with a single length of 2-3 m.According to the geological conditions, the pipe design parameters are as follows: the diameter and the thickness of which 108mm and 6 mm; in addition, the length of pipe is 1.6 m. The connection between pipe and pipe is made by thread, the length of thread is 6 cm, the grouting hole is arranged in plum blossom shape, and the diameter of hole is 15 mm. During drilling construction, it is found that the drilling speed is slow and the deviation is serious; after the penetration depth reaches 5 m, the sticking phenomenon occurs and the drilling difficulty occurs. When drilling depth reaches more than 8m, the pipe cannot continue drilling, and it is difficult to pulled out. At this time, it is found the borehole collapse after pulling out The pipe shed cannot be jacked in at all. Taking 13th drilling as an example, the cumulative footage from the first pipe to the sixth pipe is 7.5 m. It is preliminarily estimated that the drill pipe path deviates when the drill bit encounters boulder, thus resulting in conduit fracture. Therefore, we increase the drill pipe by 2 meters, adopt the method of first retreat, and then advance many times to continue drilling. However, it is found that only drill pipe is drilled, and no conduit is penetrated [35]. The drill pipe traces are visible in the front section of the arch protection. After disturbance, the surrounding sand and cobble all show loose state. When the pipe is pulled out, the surrounding loose material will be buried in the hole. The site construction drawings are shown in Figure 5.

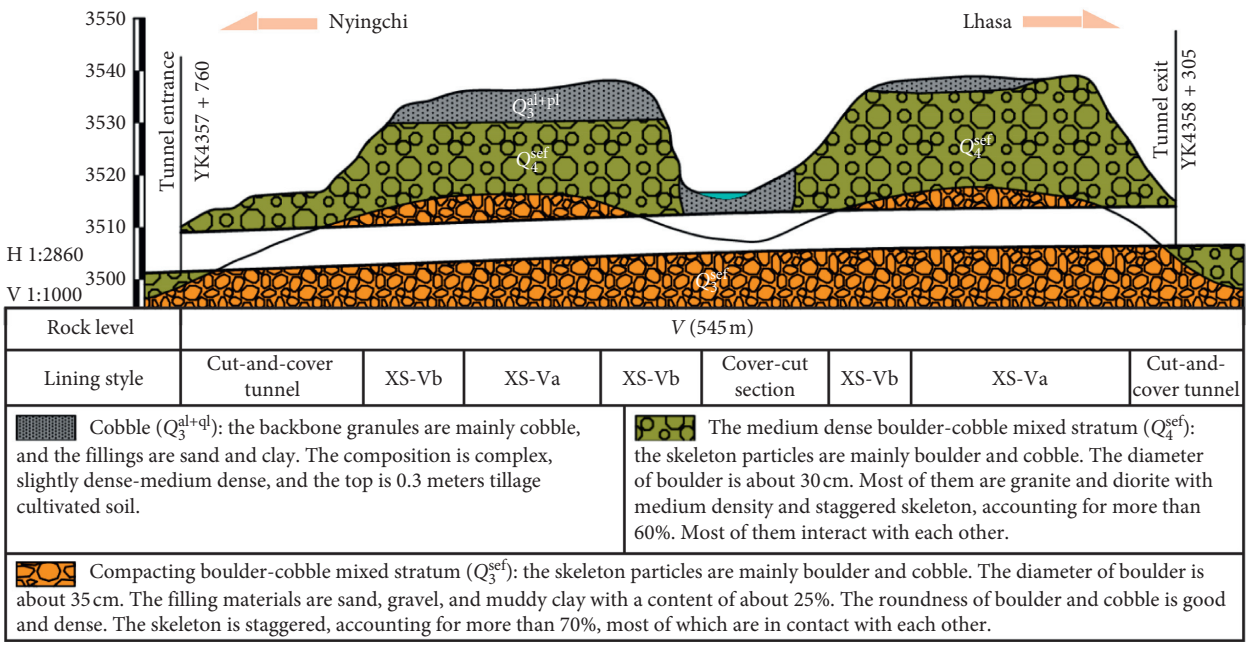


Figure 2: The geological section of the right line of the tunnel.

(a) (b)

Figure 3: Boulder-cobble mixed on excavation face. (a) Details of the compact layers. (b) Boulder removed from excavation face.

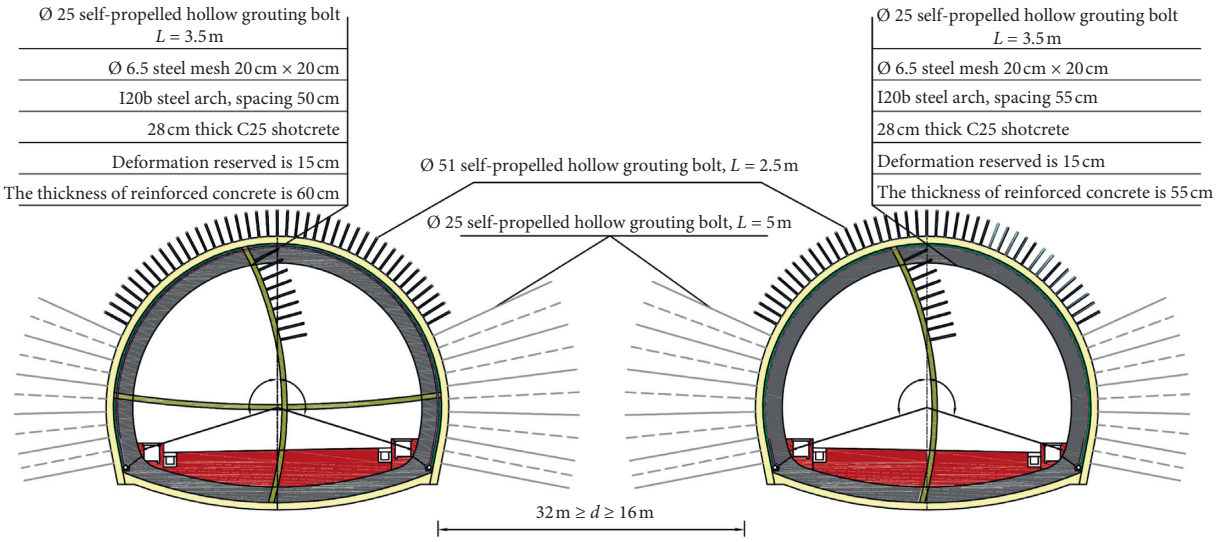


Figure 4: Tunnel section composite lining design.

3.2. Continuous Rock Falling. Taking the side wall as an example, when using the rock bolts which is 25 mm in diameter support for the construction, the average collapse thickness of the surrounding rock wall is 10 cm. However, it will reach 20 cm in depth by using 51 mm-in-diameter rock bolt support; when the rock bolts penetrate the boulder-cobble mixed grounds, large area of gravel falls due to the violent swing at the end of the bolts. When using 51 mm-in-diameter advanced rock bolts, the situation is particularly serious, and the large area of gravel above the face of the excavated surface often collapses when penetrating, which not only causes the insufficient penetration depth but also causes the serious phenomenon of overexcavation. Figure 6 shows a comparison of arch before and after the 51 mm-in-diameter feet reinforcement bolt construction. The cobble stone falls seriously, and the steel mesh is often damaged or seriously deformed.

3.3. Serious Overexcavation. When the left tunnel is excavated to the section ZK4357 + 881, the upper section is measured. The measuring length is 1.0 m, and the measuring standard is the vertical distance from the top excavation surface to the outside of the steel frame. The measured cross section is shown in Figure 7. Overexcavation value from arch top to arch waist basically shows a decreasing trend. The maximum overexcavation value at the top is 65 cm, and the average overexcavation value is 40.4 cm. The average overbreak of the disturbed arch walls on both sides is more than 20 cm. When the water content of surrounding rock is large, the overbreak value will be further expanded. The Code for "Technical Specifications for Highway Tunnel Construction" (JTG F60-2009, in Chinese) [36] stipulates that the average value of overexcavation in broken rock and soil arch of V-grade surrounding rock is 10 cm, the maximum value is 15 cm, and the maximum value of overexcavation in side wall is 10 cm. The excavation of the grounds has obviously exceeded the allowable value of the code. If not properly handled, the quality of the initial support will be adversely affected, which will lead to structural stability reduction, poor stress, and serious risk of collapse.

3.4. Bolt Penetration Difficulty. In the supporting design, the length of system bolts are 3.5m or 5m. But the problem of the system bolt is not only that it cannot reach the unrelaxed area but also that it cannot reach the design length. It is found that the penetration depth of 25 mm-in-diameter bolt is about 1.0–1.5 m and that of 51 mm-in-diameter bolts are about 0.7–1.2 m. The 25 mm-in-diameter bolts are in the limit state of torsion or failure to advance, while the 51 mm-in-diameter bolts are in the final state of failure to advance, and both bolts are prone to penetration angle deviation. The torsional failure mode of the bolt is shown in Figure 8.

3.5. Grouting Difficulty. Grouting plays an important role in self-feeding anchor and tunnel waterproofing. In the dense boulder-cobble mixed grounds, the grouting pressure is designed to be 1.0 MPa. The slurry flows through the drill pipe and diffuses through the grouting hole reserved on the drill bit. But in the actual construction, it is found that the grouting pressure reaches 2.0 MPa, and only very few grouting fluid is injected. According to the relevant research [37], there are two main reasons for this phenomenon. One is that the grouting hole itself is easily blocked by gravel during drilling; the other is that the formation itself is dense and mud content is high, the gravel cannot be discharged during drilling pipe, and fine sand is mixed with mud to form a compact inclusion layer around the rod body. The liquid is difficult to penetrate [38].

4. Construction Method Optimization

The topography of tunnel construction area is steep. The actual length of excavation in the left line is 300 m and that in the right line is 328 m. After using the original design of tunnel construction method, it is found that the little deformation and pressure of surrounding rock are controllable. The problems reflected in the construction site are as follows: the length of bolt drilling is short and torsion break occurs and the process of bolt penetration makes the surrounding rock looser and more difficult to play the role of anchorage. The process of bolt penetration causes large area of gravel falling due to the violent swing of the end of the bolt, which causes damage to the reinforcement mesh of initial support. The cavity of surrounding rock after falling block affects the stability of surrounding rock and needs to be treated The remaining voids affect the stability of surrounding rock and need to be treated. The CD method and CRD method are complicated in working procedure, low in mechanical utilization, high in cost, and long in construction period. Under the condition of serious overexcavation, it is difficult for the top and back of the middle wall to be effectively closed. The whole temporary enclosed structure loses the advantage of uniform compression, and the surrounding rock cannot be effectively stabilized. In view of the above problems in scene, the construction methods should be optimized, including the excavation method and the support system. Among them, the ZK4357 + 857~ZK4357 + 900 and ZK4358 + 200~ZK4358 + 282 construction sections adopt the original design construction method, and ZK4357 + 900~ZK4357 + 982 and ZK4358 + 110~ZK4358 + 200 use the optimized one. The actual excavation length of the right line is 328 m, in which the construction sections YK4357 + 850~YK4357 + 910 and YK4358 + 210~YK4358 + 280 use the original designed construction method and the sections ZK4357 + 910~ZK4358 + 003 and ZK4358 + 105~ZK4358 + 210 use the optimized one.

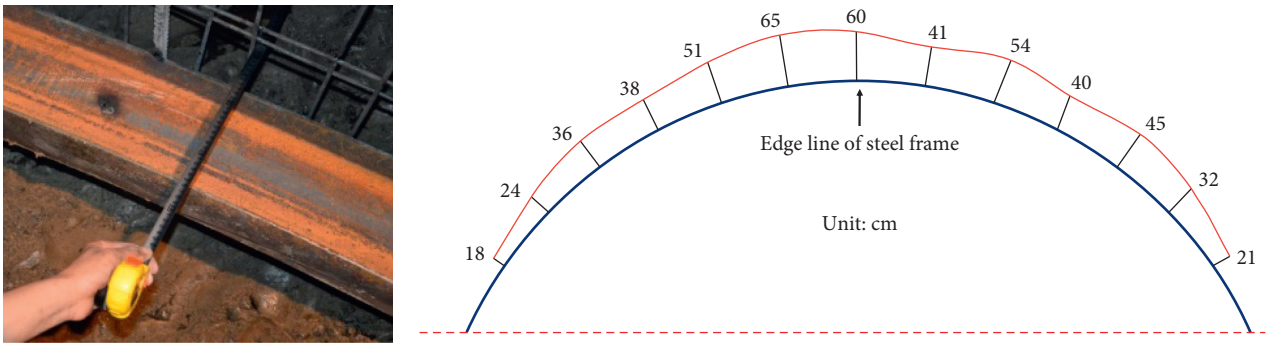
4.1. Reserved Deformation and Overexcavation Treatment. The original design of the tunnel reserved 150 mm, the actual monitoring settlement and convergence deformation are less than 30 mm, and the reserved deformation should be adjusted to 80 mm because the actual problem in construction prefers large deformation to overexcavation; in view of the serious problem of overexcavation in construction, a two-step contour

(a) (b)

Figure 5: Construction drawings of left-line inlet pipe shed. (a) Pipe shed construction drawings. (b) Borehole localization.

(a) (b)

Figure 6: Comparison of (a) before and (b) after insertion of feet reinforcement bolts. (a) Before the feet reinforcement bolts are penetrated. (b) After the feet reinforcement bolts are penetrated.



(a) (b)

Figure 7: Measurements of step overexcavation on section ZK4357 + 801 of left line. (a) Overexcavation measurement of vault edge (65 cm). (b) Overexcavation sketch at the top.

excavation scheme is put forward, that is, underexcavation of 20–30 cm during mechanical excavation and then artificial drilling of the design contour with percussive drill, which effectively alleviates the overexcavation phenomenon.

4.2. Three-Step Complementary Circular Excavation. The step method has many advantages in construction convenience, schedule, and small disturbance. By improving the three-step complementary circular excavation method, it can

(a) (b)

Figure 8: Torsional failure patterns of bolt end.

not only reduce the cost but also accelerate the construction speed and structure closure time. The so-called complementary cyclic excavation method is characterized by the complementary advantages of lock anchor and arch foot backfilling and the alternate cycle of slag backfilling in different excavation steps. The construction steps are as follows.

Upper step arc guide pit with a footage of 0.5 m is constructed with a steel frame, and the middle core soil (2) is reserved for supporting operation with the core soil as the construction platform. The width of core soil is 2 m longitudinally. Workers can stand under it to prevent falling rocks. After removing the core soil (2), the middle part of the middle step (3) is excavated downward, and the width of the groove is 3.5 m so that the excavator can be moved to this position for excavation ahead. At this time, the longest distance from the top of the groove to the upper step is about 5 m, and the arm length of the conventional excavator is close to 6 m. Under the excavation footage of this design, the excavation operation of the arch foot position in front of the arc guide pit can be realized and the basic performance of the field machinery can be brought into full play. Excavation part (4) and corresponding supporting operation: the width of part (4) is 1.5–2 m, which is 3-4 times of the first footage of part (1). After the branch protection is completed, it returns to part (1) of excavation and fills the excavated gravel back into part (4) of arch foot. Before backfilling, it is confirmed that part (4) of the support has certain strength and The backfilling height is about 1.7m and it can stabilize the arch foot Excavation part (5) and corresponding supporting operation: similar to part (4), after completion of supporting operation, the excavation is also transferred to part (2) and (3), and the gravel is backfilled to the same height of arch foot. Then, return to part (1) of the excavation and enter the next cycle. The loop steps are shown in Figure 9.

In the construction of lower steps, left and right are separated, one is constructed before the other, part 6a of excavated soil is backfilled in part 6b and 7a of excavated soil is backfilled in part 7b, and region 8 is reserved for passing passage, which does not affect the construction of upper and middle steps. The distance between upper and lower steps is 15 m~20 m, which has little interference with each other and high construction efficiency. The site construction drawings are shown in Figure 10.

4.3. Supporting System Optimization

4.3.1. Pipe Shed Optimization. According to the construction data of pipe shed, the conventional pipe shed cannot be used as the advance support measure for entering the tunnel in BCM grounds. According to the design proposal, the construction side adjusted the support of pipe shed at the entrance of the tunnel to 76 mm-in-diameter self-propelled hollow grouting pipe shed. The pipe shed has alloy drill bits, an impactor in the front, and grouting holes in the pipe section. Its structure is shown in Figure 11. Field construction shows that the penetration depth of the pipe shed is generally 12 m–15 m; the phenomenon of deviated holes and the problem of nonpenetration have been solved, which greatly solved the problems in the initial stage of construction and ensured the safe penetration of the pipe shed.

4.3.2. Bolt Optimization. The original design of the tunnel was to insert 25 mm-in-diameter self-propelled rock bolt into both side walls, but due to the insufficient penetration depth of the anchor rod and the poor grouting effect, the system rock bolt was cancelled. The original design of the tunnel was to install two 3-meter-long and 51 mm-in-diameter self-propelled anchor rods at the arch foot of each section of the steel frame. According to the penetration test results, the 51 mm-in-diameter rock bolt can penetrate flat without drill bit. The average depth is 1.15 m, only about one third of the design depth. The length of single anchor is set to 1.5 m and the drill bit is cancelled. Combined with the test results of anchor penetration, the diameter of advance anchor rod is adjusted from 51 mm to 25 mm, the length from 2.5 m to 1.8 m, and the spacing from 35 cm to 25 cm, and the overlap length is basically cancelled. In the original design, the advance bolt is overlapped on the steel frame and adjusted to enter through the reserved channel on the steel

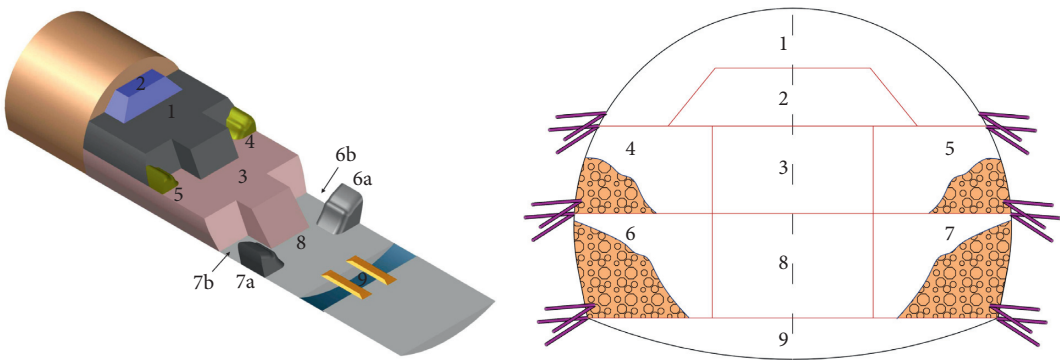


Figure 9: Complementary cyclic three-step excavation method. (a) Three-dimensional sketch of excavation. (b) Excavation of elevation view.

Figure 10: Section distribution of complementary cyclic excavation method. (a) Distribution of excavation surface of upper and middle steps. (b) Construction on one side of lower steps.

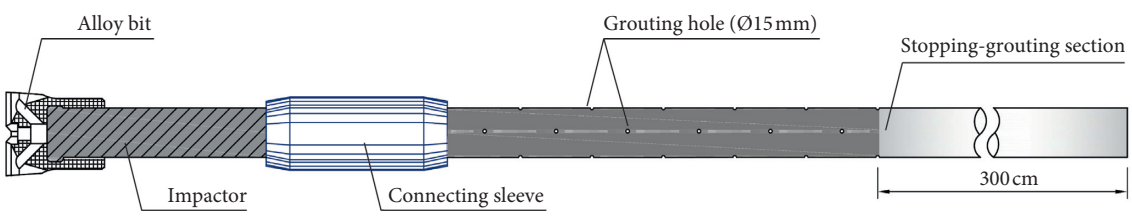


Figure 11: Structure of self-propelled hollow grouting pipe shed.

frame. When penetrating the bolt, the construction face will be disturbed. Therefore, shotcrete reinforcement will be carried out within 30cm of the face in front of the steel frame, and we should pay attention not to cover the reserved holes as well; then, reinforce the advance anchors after drilling them [39].

4.3.3. Backwall Grouting. The common backwall grouting is the synchronous grouting and secondary grouting technology after shield tunnel segment lining. The design unit points out that this technology is used in this cobble grounds. A grouting pipe is reserved every 1.5 m in the ring of the supporting structure and is used after the construction of the lower step. The grouting of a step: the main effect of grouting is filling grouting and seepage grouting. The average grouting pressure in the field test section is only 0.5 MPa, and when the grouting pressure rises, the lining surface has grout leakage. The grouting distribution behind the wall is shown in Figure 12.

4.3.4. Initial Support Optimization. In the early stage of the initial support construction, the main support function is the rock bolt without strength. When the strength is formed, the surrounding rock pressure begins to increase, and the longitudinal restraint of the initial support structure itself plays a supporting role. This kind of support is mainly formed by bending, so the longitudinal connection bar of the top should be strengthened. In order to help concrete

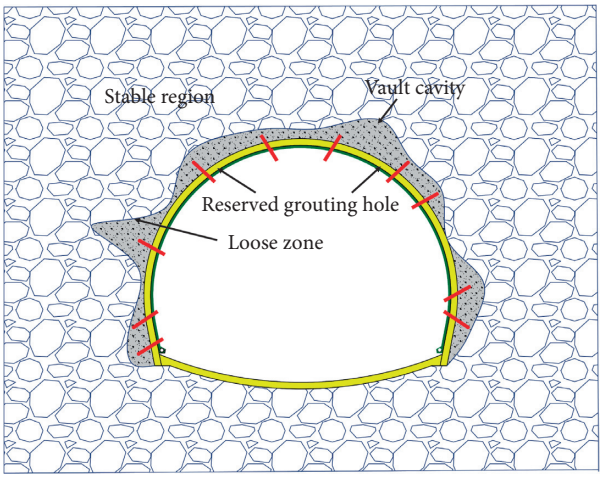


Figure 12: Grouting distribution behind initial support.

structure to be longitudinally tensioned and improve its flexural capacity, the original design of steel mesh is designed as 6.5 cm steel mesh of 20 cm. During construction, there are always some problems such as the steel mesh is often broken, cobbles are squeezed out from the aperture and even injured the constructors. Therefore it is proposed to change the single-layer reinforced mesh into double-layer, and adjust the aperture of steel mesh to 10 cm ×10 cm. Strengthen the solidity of contact welding which can effectively prevent the falling of rocks during grouting and strengthen the bearing capacity of initial support structure.

5. Field Test of Construction and Analysis

After the completion of the construction, there are 22 monitoring sections for surface subsidence and 32 for deformation in the tunnel. Because there are many monitoring sections for deformation, we select typical sections for analysis after data integration. We take the similar geological conditions YK4357 + 880 (section 880) of entrance and YK4358 + 270 (section 270) of exit, YK4357 + 920 (section 920) adjusted by the excavation method and YK4358 + 150 (section 150) as typical sections for analysis (Figure 13). Two groups of sections are representative, the first one is close to the entrance of the tunnel, far from the gully, mainly above the groundwater level, and the formation is wet, but there is no obvious seepage and outflow, and the burial depth is 17.3 m and 16.1 m, respectively. The second group is close to the gully, in which the grounds are highly filled with water so that we can see the obvious seepage from the excavation surface and local femoral effluent; furthermore, water accumulation often occurs at the arch foot, and the burial depth is 24.1 m and 26.4 m, respectively.

5.1. Surrounding Rock Monitoring Settings. The layout of survey points is shown as follows: 1# is used for settlement monitoring after the arch excavations 2# and 3# and 4# and 5# points are arranged after middle and lower steps are excavated to form into two convergence lines, respectively. When excavating by the CD method (Figure 14), the change of line 1 data of horizontal convergence line is accumulated by the change of line 1a and line 1b. It is convenient for comparing the convergence under the two construction methods. Affected by the change of excavation method, there are two convergence lines in the early stage of tunnel excavation. After the change of the excavation method, the arch foot backfilling affects the convergence monitoring of the points 4# and 5# and adopts only one convergence line from the upper part. Surrounding rock pressure test section selected section 920 and section 150 where the surrounding rock conditions are poor and similar to carry out pressure test. The distribution position of the pressure box is shown in Figure 15. The pressure box is placed at the vault, arch waist, arch wall, and near the arch foot between the initial support and surrounding rock. The pressure box corresponds to the former one between the initial support and the second lining. As is shown in the figure, the pressure box is buried selectively under various conditions, but it is not fully buried.

5.2. Analysis of Measurement Results

5.2.1. Deformation Result of Surrounding Rock

(1) Vault Settlement. As shown in Figure 16, after the improvement of the excavation construction method, even with the increase of formation water content and burial depth, the settlement deformation still shows a gradually decreasing trend. It can be found that the settlement values of vaults at different sections are quite different, and the settlement characteristics of two groups of sections are obviously different. The first group of section 880 and section 270 has a great similarity, while the second group of section 920 and section 150 has a great similarity. In the former group of sections, the settlement development is characterized by continuous increase, accelerating development after 4-5 days of excavation, tending to be stable after 18–20 days of excavation, and no further development. The maximum settlement values of the two sections are −24.3 mm and −28.0 mm, the average settlement value is −26.2 mm, and the settlement rates are −1.1 mm/d and −1.3 mm/d, respectively, in the stage of deformation development mm/d. The latter group of sections developed rapidly after 5 to 6 days of excavation and entered the stage of deformation stabilization after 20 to 22 days of excavation. The maximum settlement values of the two sections were −13.0 mm and −14.6 mm, and the average settlement value was −13.8 mm. The deformation rates in the stage of rapid settlement were −0.6 mm/d and −0.7 mm/d, respectively. According to the construction data, the development of settlement is affected by the construction of lower steps. The middle steps are excavated 4-5 days after the upper steps are excavated, the lower steps are excavated 12 days after the middle steps are excavated, and the inverted arch is closed 18 days after the section is excavated. The deformation quickly enters the convergence stage. Regardless of the maximum settlement value or settlement rate, the condition of the second section is better than that

(a) (b)

Figure 13: Comparisons of excavation surface conditions of two groups of sections. (a) Cross section 880. (b) Cross section 150.

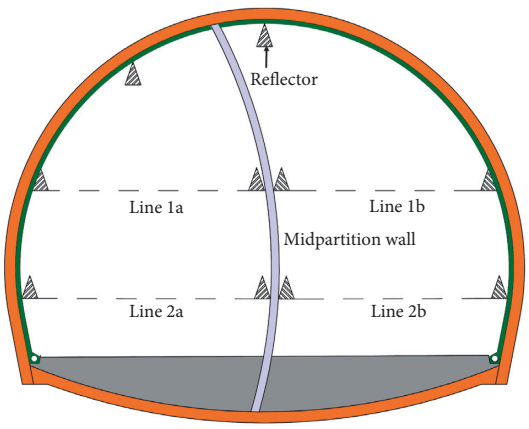


Figure 14: Layout of deformation measurement for the CD method.

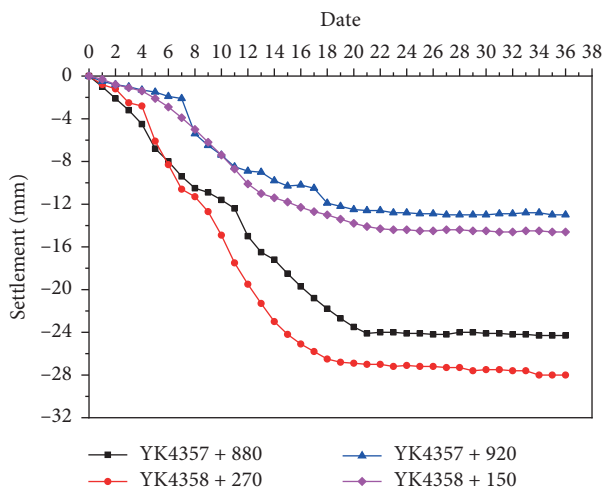


Figure 16: Development curve of vault settlement.

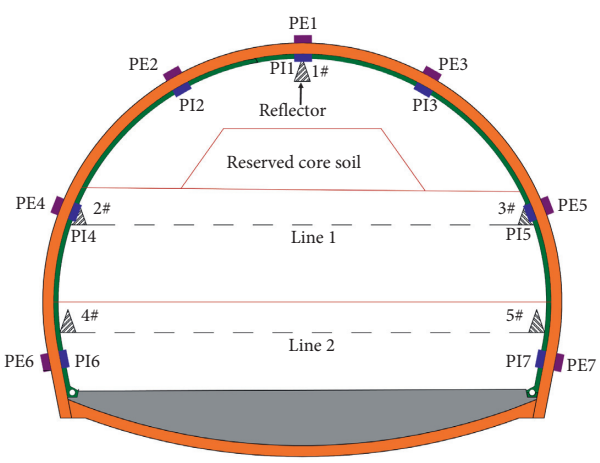


Figure 15: Layout of deformation measurement for stepped tunnel.

of the first section. The maximum settlement and settlement rate of the second section are about 50% of that of the first section.

(2) Horizontal Convergence. The horizontal convergence lines of four typical sections are extracted, and the relationship between convergence deformation and excavation time is obtained as shown in Figure 17. For the convenience of joint analysis with settlement, the horizontal convergence deformation of each section is timed at the beginning of arch excavation. The first group of section 880 and section 270 is two horizontal convergence lines, while the second group of section 920 and section 150 contains a group of horizontal convergence lines. The data of the second group of horizontal convergence are from line 1. It can be seen in the figure that the horizontal convergence deformation of the first group of sections has a high similarity. The deformation of the first group of sections has entered a rapid development after the excavation of the middle step. On the 11th day or so, the horizontal convergence deformation has a short growth stage. On the 18th day, after the inverted arch closes, the horizontal convergence quickly enters a stable stage. The maximum horizontal convergence value of the line 1 of section 880 and section 270 is divided into two stages. The horizontal convergence average is 17.4 mm, the horizontal convergence rate is 0.9 mm/d and 1.1 mm/d in the horizontal

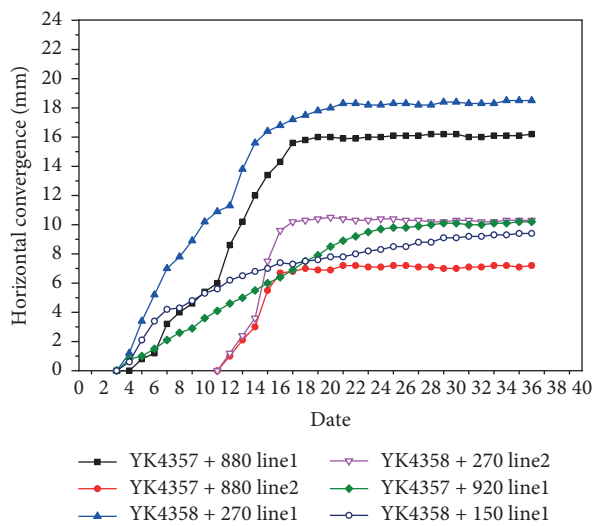


Figure 17: Horizontal convergence development curve.

convergence development stage, and the maximum horizontal convergence rate of line 2 is 7.2 mm and 10.3 mm, respectively. The horizontal convergence rate of line 2 is 0.8 mm/d and 1.2 mm/d. Although the horizontal convergence value is small, the deformation measurement and deformation rate are equal to that of line 1. The horizontal convergence deformation patterns of section 920 and section 150 in the second group are slightly different. Section 150 grows faster in the early stage and lasts longer in the later stage. Section 920 tends to be stable around the 24th day, and its early deformation is slow and stable. The maximum deformation values of the two sections are 10.2 mm and 9.4 mm, respectively; the average convergence value is 9.8mm, and it is in the rapid development stage. The horizontal convergence rates are 0.5 mm/d and 0.5 mm/d, respectively. The average horizontal convergence deformation of the second section is 56.3% of that of the first section, and the horizontal convergence rate is about half of that of the first section. Whether it is convergent deformation or settlement deformation, the deformation of the cobble grounds is small, and the deformation reserved for 15 cm in the original design is obviously redundant. The surrounding rock condition of the second section is obviously worse than that of the first section, but the deformation of the second section is better than that of the first section, mainly due to the optimization of excavation and construction methods.

5.2.2. Surrounding Rock Pressure Result

(1) Result of Pressure between Surrounding Rock and Initial Branch. In the actual test, 1–7# point pressure boxes are buried in section 150 and 1–5# pressure boxes are buried in section 920. The measured temporal variation curve of surrounding rock pressure is shown in Figure 18. Two points can be seen in the figure: (1) the measured surrounding rock pressure value is small; (2) most pressure boxes do not show a significant increase trend with the increase of time. The inhomogeneity of cobble grounds leads to the uneven stress on the surface of pressure box, which results in some surrounding rock pressure measuring points being overhead and unable to bear force. Some measuring points have concentrated pressure, which is very large, but a little small. The 2# measuring point in section 920 supports this rule. Even if calculated according to the maximum pressure, the surrounding rock pressure of shallow cobble formation is really not large. The pressure profile of surrounding rock is shown in Figure 18. The maximum pressure of surrounding rock of section 150 is 40.99 kPa, which is located at the upper part of the right arch wall. The minimum value is only 9.21 kPa, and the average value is 18.6 kPa. The average pressure of section 920 is 30.4 kPa, and there is obvious pressure concentration at the waist position of 2# measuring point. The maximum value of measured surrounding rock pressure is 96.7 kPa, only 26.1% of the calculated value. The pressure box may be affected by the boulder stone. The results show that the little surrounding rock pressure of boulder-cobble mixed grounds is less than the pressure of collapse body in the size of 2 m. According to the failure mode of surrounding rock and the distribution of overbreak, the top of the boulder-cobble mixed grounds is easy to collapse and destabilize, resulting in loose pressure, and the top is the most serious, easing to both sides.

(2) Pressure Results between Primary Support and Secondary Lining. The spatial distribution of the pressure stabilized values is described, and the spatial distribution is shown in Figure 19. The contact pressure between the second liner and the initial support is uniform. The 4# point pressure of section 150 is 0.18 kPa. It is presumed that this part may be void, so it is not considered. The pressure of the surrounding rock in the two sections is large at the top and decreases in sides. The maximum contact pressure of section 150 is 10.1 kPa; the average is 6.4 kPa, and the maximum contact pressure of section 920 is 20.1 kPa; the average is 13.2 kPa. Compared with some shallow loess tunnels and metro tunnels measured in China [40], the contact pressure is very small. If the calculation of shallow-buried tunnel is carried out according to the current "Highway Tunnel Design Code" (JTG-D70-2004, in Chinese) [41], the surrounding rock pressure of the grounds is mainly loose pressure, the surrounding rock weight is 21.0 kN/m^3, the buried depth is 24 m, and the vault pressure is about 370.4 kPa. The maximum value of measured surrounding rock pressure is 96.7 kPa, only 26.1% of the calculated value. Therefore, it is inferred that the main value of surrounding rock pressure is the loose pressure in the loosening zone, and the shallow boulder-cobble mixed grounds have certain self-bearing capacity; it can be seen that under the current design, the secondary lining is basically used as a safety reserve, combined with the analysis of contact pressure between the initial support and the secondary lining.

5.2.3. Results of Anchor Test

(1) Bolt Penetration Test. The bolt can't work in original design, so it is necessary to carry out bolt penetration test

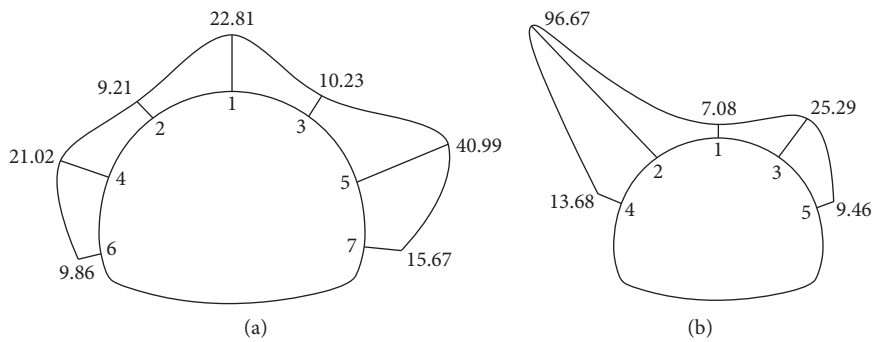


Figure 18: Contact pressure distribution between surrounding rock and initial branch (unit: kPa). (a) Pressure distribution of section 150. (b) Pressure distribution of section 920.

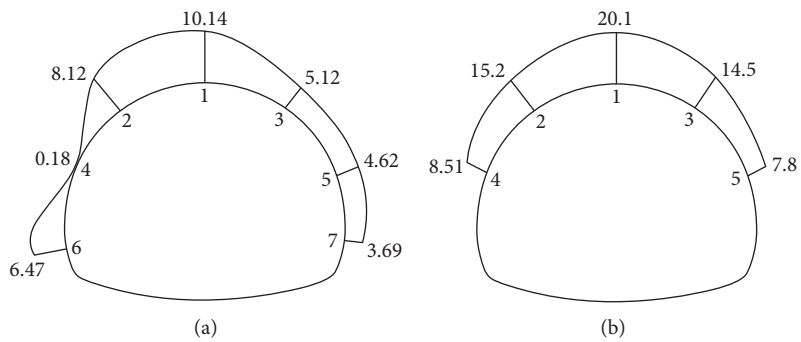


Figure 19: Contact pressure distribution between primary support and secondary lining (unit: kPa). (a) Pressure distribution of section 150. (b) Pressure distribution of section 920.

and pull-out test in order to correct the design Field observation shows that the penetration depth of anchor rod is closely related to the diameter of anchor rod. Reducing the diameter of anchor rod is beneficial to increase the penetration depth. According to field technicians, it is easier to penetrate anchor rod without drill bit. Based on this, the penetration test is designed. We choose the location of the middle step arch wall to carry out two penetration tests of 25 mm-in-diameter and 51 mm-in-diameter bolts, respectively. In each penetration test, five bolts of the same diameter with and without the bit were selected to penetrate the formation. The length of the 25 mm-in-diameter and 51 mm-in-diameter test bolt is 3.5 m and 2.5 m. We choose the position of flat arch wall to penetrate vertically. "Record the remaining length of bolt and its statue after penetrating into the limited depth The 25 mm-in-diameter bolt penetration test was carried out when three steel frames were erected on the middle steps. The first point distribution is shown in Figure 20. The bolts are arranged in the shape of quincunx, but the spacing is slightly less than the design value. For safety, we penetrate the rock bolt to minimize the swing of the bolt body after the steel frame and the reinforcing steel mesh are installed (Figure 21). The bottom collapsed gravel should be cleared before penetrating the lower bolt.

Statistics show that the average penetration depth of 25 mm-in-diameter with drill bit is 1.34 m, only 38.2% of the designed depth. The bolt without the drill bit is 1.65 m, only 47.1% of the designed depth (Figure 22). The average

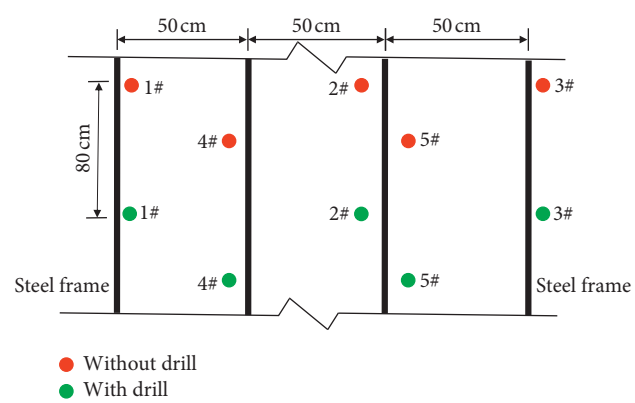


Figure 20: Distribution of first test boreholes.

penetration depth of bolt without the drill bit is 0.31 m and 23.1% more than that with the drill bit. So, it is easier to penetrate without the drill bit, but it takes more time. Records show that five bolts with drill bits and eight bolts without drill bits are twisted. It is related to the penetration depth. The deeper the penetration is, the easier the torsion failure will occur. 51 mm-in-diameter bolt penetration test is carried out at the bottom of the side wall of the middle step. The distribution of points is similar to 25 mm-in-diameter bolt but closer to the steel frame so that the steel frame can be reinforced later. The total number and type of bolts are the same as 25 mm-in-diameter bolt. As shown in Figure 23, the average penetration depth of 51 mm-in-diameter bolt with a

Figure 21: Bolt penetration test process.

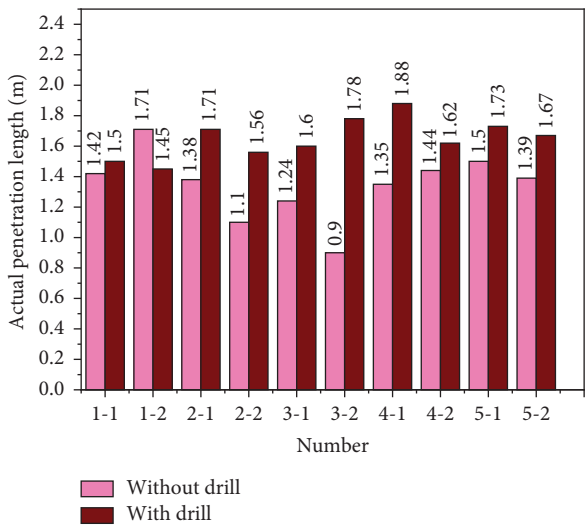


Figure 22: mm-in-diameter 25 bolt penetration depth columnar.

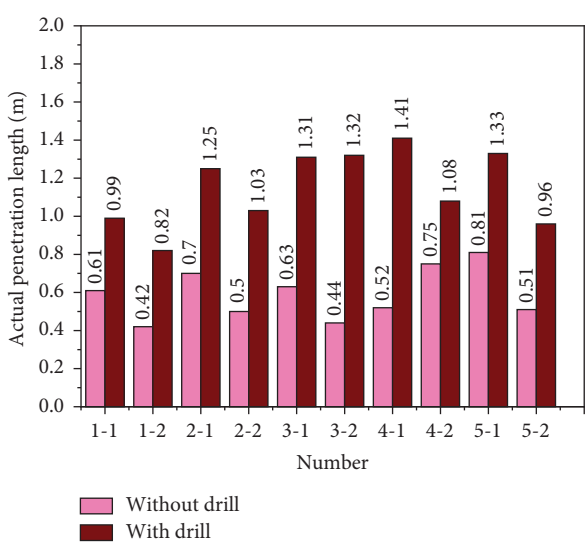


Figure 23: 51 mm-in-diameter bolt penetration depth columnar.

bit is only 0.59 m, and the end of the bolt can be pulled out with a little force. Therefore, the drill bit has never been penetrated in the actual construction. The average depth of the bolt without drill bit is 1.15 m, which can only reach 38.3% of the designed depth of feet reinforcement bolt and 46.0% of the advanced bolt. The reason why we stop penetrating the 51 mm-in-diameter bolt is that it is unable for us to continue. Through the test, we can find that the maximum penetration depth of bolt under different conditions is less than 50% of the designed depth. For the same diameter bolt without drill bit, the depth is larger than that with drill bit; for the different diameter bolt, the penetration depth of small one is larger.

The reason why the penetration test shows this result is mainly determined by the characteristics of boulder and cobble mixed grounds and the self-propelled bolt drilling mechanism. Self-propelled bolt drilling depends on two ways: one is rod threading and the other is bit drilling. The bolt without drill pushes in by thread. The self-propelled bolt with a drill is drilled onto a cobble or boulder, and the medium around the bolt is easy to be disturbed and loose, resulting in collapse. The fragmentation of law brought about collapse instead. Figure 24 shows an outcrop of 51 mm-in-diameter bolt at the middle step when excavating the lower step. Obviously, both bolts are unable to advance because they are stuck on the rocks.

(2) Drawing Test. Under these grounds, self-propelled anchor is a kind of compact penetration, and the grounds' frictional resistance of bolt is very large. Referring to "Code for Design of Highway Bridge and Culvert Foundation and Foundation," the standard value of grounds' friction of the boulder and cobble is 400 kPa. Under the action of no biting force, the drawing force in the length range of 1.5 m is calculated to be 47.1 kN which is almost equal to the designed drawing force of 50 kN. The destructive drawing test of bolt penetration test was carried out by using the MSL-200 drawing instrument. The maximum loading value is 100 kN. After the penetration test, we select 7 25mm-in-diameter bolts with proper angle and without damage to the end for drawing test. Without grouting, the drawing test was carried out three days after completion of shotcrete. The pulling force data obtained are shown in Table 1. It is found that when the bolt is subjected to the ultimate tension, the pulling force will decrease rapidly and stabilize quickly at a certain pulling force value. The analysis shows that the initial drawing force is balanced by the bonding force of concrete and the friction force of surrounding rock which can affect the bolt. When the pulling force is loaded to the limit value, the interface between anchor and surrounding rock shows relative dislocation, and the friction force decreases or disappears instantaneously. The bonding force of concrete to anchor bears the residual load. According to the results of research in reference, the bounding force of 20 kN is used to correct the ultimate drawing force. From the data, it can be found that the average length of the three bolts with drill is 1.32 m, the theoretical reverse pull-out resistance is 41.5 kN, and the actual average drawing resistance is about 55.2 kN, which is close to each other, but without drill bits, the theoretical reverse pull-out resistance of the four bolts in average length of 1.76 m is 55.3 kN, while the actual average force is

Figure 24: The end of the 51 mm-in-diameter bolt is above the floated stone.

Table 1: ∅25 ultimate pull-out strength of bolts.

Number	Bolt condition	Actual penetration length (m)	Ultimate pull-out strength	Causes of termination	Gripping force corrected value (kN)
3	Including drill	1.38	62.5	When loaded to 62.5 kN, it suddenly fell back to 22.1 kN and could not continue to increase	37.4
5	Including drill	1.24	74.3	Loading to 74.3 kN reduces the drop to 28 kN	47.6
7	Including drill	1.35	72.2	Loading to 72.2 kN reduces the drop to 30.2 kN	49.9
9	Including drill	1.50	82.5	Achieves a predetermined value of 100 kN	≥80.0
3	Excluding drill	1.71	92.2	Loading to 92.2 kN reduces the drop to 42.0 kN	70.1
4	Excluding drill	1.94	100	Achieves a predetermined value of 100 kN	≥80.0
5	Excluding drill	1.60	86.6	Loading to 86.6 kN reduces the drop to 38 kN	64.5
6	Excluding drill	1.78	100	Achieves a predetermined value of 100 kN	≥80.0

above 74.7 kN. Obviously, without the drilling bit, it is easier for us to penetrate the bolt and more difficult to pull out, which is also more suitable for the actual situation of the formation.

(3) Axial Force Test of Feet Reinforcement Bolt. In this test, after the feet reinforcement bolt is penetrated, the bolt with larger penetration depth and better stability is selected to weld the tail of the bolt, and the clearance between the reinforcement bar and the bolt is filled to make the steel bar axonometric-apparatus welded firmly. The other end of the apparatus is directly welded on the steel frame. The installation is shown in Figure 25.

Figure 25: Installation site of feet reinforcement bolt axonometric-apparatus.

In the actual test, there are two sections of feet reinforcement bolt. Section 1 is located in section 871 near the entrance of the right line. The bolt of this section is designed according to the original design, and the number of feet reinforcement bolts is two. Section 2 is located in section 918 at the entrance of the right line, and the feet reinforcement bolt is optimized to four steel frames per section. The bolt and test results are shown in the following Figure 26.

It can be seen in the figure that the axial force distribution of feet reinforcement bolt is different in different parts. The upper step feet-lock bolt is subjected to tension, the middle and lower step one is subjected to pressure, and the absolute value of tension on the upper step is far greater than the absolute value of pressure on the middle and lower step. The analysis shows that the feet reinforcement bolt of the upper step arch foot is close to the horizontal and bears a large vertical load. The tension of the upper step is mainly caused by the bending of the end due to the top load; when the middle and lower segments are installed, the feet reinforcement bolt and the horizontal line are in the angle of 30–50 degrees, the moment when the vertical pressure effect

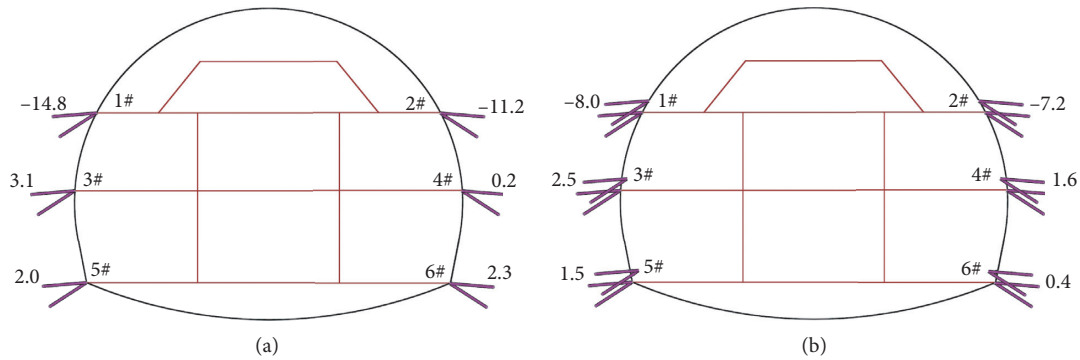


Figure 26: Installation of drawing of feet reinforcement bolt axis dynamometer.

is more obvious, which makes the whole step bear compressive stress. The maximum tensile force of section 871 is −14.8 kN, which is converted to 16.5 MPa, less than 10% of the yield strength of the bolt, and the compressive stress is even smaller. Combining the results of the drawing test in the previous test, the boulder and cobble mixed grounds have a very strong bonding force and wrapping effect on the bolt. In fact, the steel frame is also supported by the lower step grounds in the construction stage. Its stress is affected by bending moment, vertical load, and horizontal load. The test value of axial force is not good to evaluate. However, it can be seen from the distribution of feet reinforcement bolts that the absolute value of axial force of bolts decreases when the number of feet reinforcement bolts is strengthened. Anchorage failure occurs in 4# bolt in section 871 and 6# bolt in section 918, so it is necessary to increase the number of bolts. The variation of bolt axial force with installation time is shown as shown in Figures 27 and 28. In these two sections, it can be seen that the anchor force at the upper step is affected by the lower step, which increases rapidly after excavation, and enters a stable state after the completion of the middle step construction and basically does not increase. The bolt at the middle and lower step changes into a stable state 7 days after the completion of construction, and then the strength of concrete has been formed, and the longitudinal constraints between supports are approximately equal. The beam effect is also strong, so it does not change.

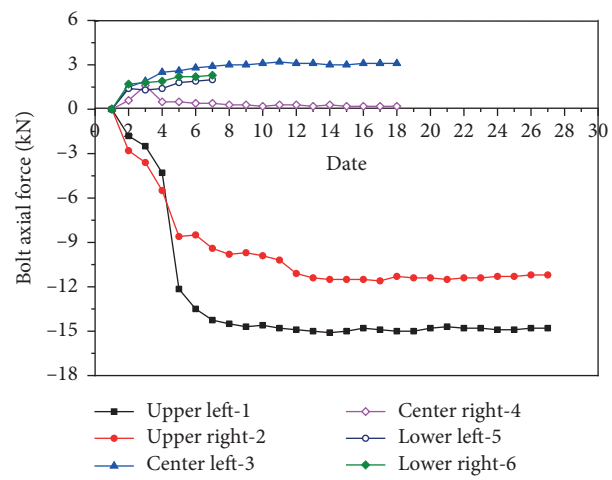


Figure 27: Section 871 feet reinforcement bolt axis force change curve.

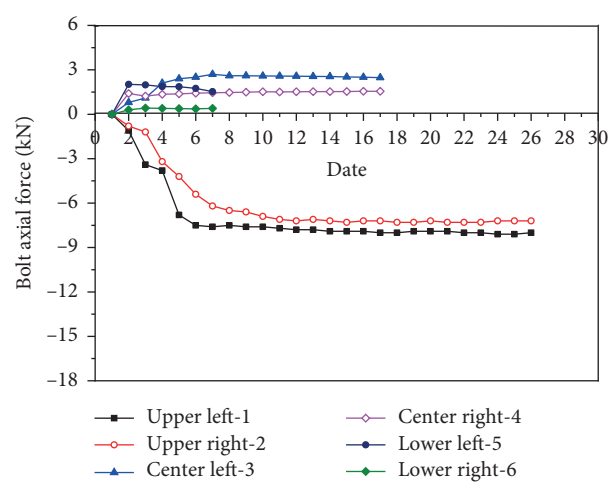


Figure 28: Section 918 feet reinforcement bolt axis force change curve.

6. Discussion

At present, some tunnels have been built in floated rock and sandy formation in China. The grounds conditions are different, but as granular media, many engineering problems are similar. Many uncertain problems in underground engineering can be solved by experience analogy. Therefore, it is of practical significance to summarize the similar projects in the past and seek the optimum reference. At present, the tunnels constructed in floated rock and sandy cobble grounds are mainly urban subway tunnels, urban municipal road tunnels, mountain highway tunnels, and railway tunnels. The construction methods include shallow tunneling, shield tunneling, and so on. The main areas include Beijing, Chengdu, Changsha, and other places. The literature of similar projects has been extensively investigated. Seven project data have been collected. The project data statistics have been carried out from the aspects of geological characteristics, engineering problems, supporting and excavation systems, etc. Detailed information is shown in Table 2.

Table 2: Statistics of tunnel construction in cobble and sandy-cobble grounds.

Number	1	2	3	4
Name of tunnel	Sikewan Tunnel of Lanhai Expressway, Gansu Province [42]	Beijing metro line 9 [37]	Gangudun Tunnel, Longnan, Gansu Province [43]	Chengdu metro line 1 [44]
Geological survey	Medium-dense gravel grounds are a foot-slope accumulation layer with high obliquity and large porosity. Clastic media such as sandy soil, mudstone, and mudstone are filled between cobble and gravels, which have poor cementation, strong permeability, well-developed groundwater, and large water content.	The excavation of Q4 cobble bed and Q3 strongly weathered conglomerate bed has large cobble size, high content, and good cementation. In the whole conglomerate layer, the thickness of the dome conglomerate layer is 1.4–1.6 m, the groundwater is abundant, and the conglomerate layer is hard.	The tunnel passes through strata of Q4 diluvial fine breccia soil, alluvial coarse sand, fine cobble soil, and coarse cobble soil, and underlying bedrock is phyllite and limestone.	In dense sand and cobble grounds, the average diameter of cobble is 130 mm and the maximum diameter is 400 mm. Tunnel is located below the groundwater level without precipitation. It is easy to collapse without precipitation. The stability is greatly improved after precipitation. The permeability coefficient is $2.0*10^{-2}$ cm/s. When the excavation span is 3 m, the face has 10-hour self-stabilization ability, which is conducive to construction.
Engineering problems	(1) Ring and oblique cracking of the initial branch, with the maximum crack of 6 mm. (2) The cobble is easy to collapse from the leading small catheters, and the cobble above the arch waist is easy to fall off and collapse on a small scale. (3) During the construction of arch frames on both sides, groundwater in the side wall flows out in a linear or femoral shape, and the water accumulation is serious.	Because the hardness of conglomerate layer is bigger and smaller, the grouting effect is unsatisfactory, and there are many continuous seepage phenomena in the tunnel, and the original design of advance small pipe grouting is not applicable.	A large amount of water is produced between the sandy cobble soil layer and phyllite layer, which concentrates on the arch foot position on both sides of the upper and middle guide. The water output is more than 700 m3/d, and the settlement deformation is large.	(1) Tunnels are located below the groundwater level and collapse easily without precipitation. (2) Φ42 small conduit is easy to deflect and deviate when it is penetrated. It can be solved by using PH150 bolt drill and erecting steel cone head.
Construction/ excavation method	(1) Shallow burial and subsurface excavation, upper and lower step excavation, and buried depth of about 15–25 m. (2) The maximum deformation is over 300 mm. (3) After strengthening the support, the deformation is reduced by more than 50%.	Shallow buried excavation, upper and lower step excavation, and upper step reserved core soil.	Construction of three steps and seven parts excavation method.	(1) Shallow excavation method with a depth of 13 m. (2) Reserved deformation of 50 mm, 0.5 m per foot. (3) CD excavation, the ground settlement is less than 10 mm, the settlement in the tunnel is less than 5 mm, and the convergence is less than 10 mm. (4) The upper and lower steps are excavated with reserved core maps, and the lower steps are divided into left and right sections. The deformation of the lower steps is very close to that of the CD method, but the construction disturbance is smaller.

Table 2: Continued.

Number	1	2	3	4
Support system	The original design adopted Φ4.2 m long 42 small conduit, system bolt was 3.5 m long D25 grouting bolt, I20a steel spacing was 0.75 m, reserved deformation was 12 cm, and C25 reinforced concrete was used as secondary lining. (2) Later stage support was strengthened, leading small conduit was changed to double layer, length was adjusted to 1.2 m and 2 m, steel frame spacing was 0.5 m, and feet reinforcement bolt group was changed to double group.	The penetration and grouting tests ofΦ25, Φ30, and Φ32 self-propelled bolts were carried out in the tunnel, and YT28 gas leg drill was used to penetrate the bolt. It was found thatΦ30 was the most suitable, and the air outlet of the bit was optimized.	Composite lining design of general mountain tunnel	(1) The original design of 3.5 m Φ42 small conduit for advance anchor rod, lock foot anchor rod is Φ32 self-drilling anchor rod, the second lining is 40 cm reinforced concrete, and I22 I-shaped steel. (2) After precipitation, the grounds are stable, and the unsystematic bolt reduces the disturbance, which is beneficial to the ground stability. (3) The test of steel frame internal force and surrounding rock pressure shows that the maximum of steel frame is only 12 kN, the earth pressure is only 80 kPa, the surrounding rock pressure is small, and the supporting structure is safe.
Technology summary or breakthrough	Water-rich sandy cobble strata are summarized as follows: small footage excavation, strong support means, appropriate grouting technology, strictly prohibit blasting, and manual or mechanical excavation is needed to reduce disturbance and destruction of strata. High water content leads to the whole ground softening and loss of bearing capacity, which is the cause of large deformation and lining cracking.	The original design of vault 1.8 m long Φ25 small conduit was changed to 2.5 m long Φ30 self-propelled bolt, the water outlet of vault was improved, and the effect of grouting reinforcement was obviously improved.	The whole section radial grouting technology is adopted to stop water. The grout is cement water glass slurry, and the number of rock bolts is strengthened. The deformation of surrounding rock is improved.	(1) It is necessary to dewater before excavation below the groundwater level. (2) The deformation of surrounding rock by the CD method is similar to that by upper and lower step method, and it does not exceed the reserved deformation. (3) The construction speed of the step method is faster and the disturbance to construction is smaller, so priority should be given to it. (4) Sand and cobble strata have good stability, and less disturbance should be given priority, followed by strong support.

Statistical analysis of engineering cases of cobble and sandy cobble tunnels shows that there are many common laws in their construction characteristics, which are summarized as follows:

(1) When the water-rich grounds and the tunnel are located below the groundwater level, the stability of the tunnel will be greatly reduced, which will bring about many construction problems such as large deformation, instability, and destruction. If conditions permit, precipitation should be considered first and then excavation.

(2) Almost all the existing engineering cases emphasize the importance of less disturbance. As a material cemented by granular particles, the disturbance is not disturbed and stable for a long time. Once disturbed, it is easy to be destroyed. Its deformation and surrounding rock pressure are not large, and the design of strong support has not played a full role.

(3) In Beijing and Chengdu metro, the shallow-buried and underground excavation method is mainly the step method. Compared with the CD method, the

step method has less disturbance to the grounds and can make full use of the limited stability of the grounds for rapid construction.

(4) The transverse damage effect is greater than the longitudinal one. The common problem in shallow excavation is that there are more unstable blocks at the top, less side wall collapse, and less extrusion and collapse failure at the face of the palm.

(5) Whether in gravel or gravel grounds, the main problems faced by the construction of ordinary anchors and small conduits are the difficulty of hole-forming and the serious collapse of holes. Even with high-power hole-forming machinery, the maximum penetration depth of small conduits is only 2.5 m; self-propelled anchors have better adaptability, but their length is not more than 2.5 m.

(6) In boulder-cobble mixed stratum, the penetration depth of the system bolt is insufficient, and it can hardly play the role of anchorage. Its construction disturbance is too large, so it is better not to do it. Most of the relevant engineering cases have been cancelled.

(7) Feet reinforcement bolts play a role in construction but it is not easy to implement. Therefore, the importance of feet reinforcement bolts is generally emphasized and should be strengthened.

There are also many differences in the construction characteristics of boulder-cobble mixed stratum, which are summarized as follows:

(1) Compared with the tunnel excavation in sandy-cobble mixed stratum, the deformation of surrounding rock in boulder-cobble mixed stratum is smaller, and there is no large deformation of surrounding rock and cracking of lining in Nianggaicun tunnel.

(2) In the case of excavation of strongly weathered conglomerate stratum, the length of advanced small catheter be increased in the design change to achieve a larger reinforcement area. In this case, the length of advanced small catheter is short, and the double-layer advanced small catheter is used to achieve a larger reinforcement area.

(3) In boulder-cobble mixed stratum, the ability of resisting construction interference is less than that of sandy-cobble mixed stratum.

7. Conclusions

As a typical granular bulk medium, construction technology is different from that in continuous medium. Through on-site investigation and testing, the conclusions are as follows.

According to the two selected groups of typical sections, the average settlement of vault is reduced from 26.2 mm to 13.8 mm by using three-bench complementary cyclic excavation method, and the average horizontal convergence value of horizontal convergence line 1 is reduced from 17.4 mm to 9.8 mm. The conclusion shows that it is reasonable to adjust the reserved 15 cm deformation of original design to 8 cm. The surrounding rock condition of the second section is obviously worse than that of the first section, but its deformation condition is better than the first one, mainly due to the optimization of construction methods. Combined with the phenomenon of over-excavation and the pressure distribution of surrounding rock, it can be seen that the three-step complementary cyclic excavation method is superior to the CD method in granular media. The shallow-depth floated-cobble grounds have a certain self-bearing capacity. According to the contact pressure analysis between the initial support and the secondary lining, it can be seen that under the current design, the secondary lining basically exists as a safety reserve. The optimization of excavation method can shorten the construction period and reduce the cost.

The maximum penetration depth increased from 8 m to 15 m after adjusting the support of the pipe shed at the entrance to 76 mm-in-diameter self-propelled hollow grouting pipe shed. The problem of deviated hole and nonpenetration was solved.

In view of the serious problem of overexcavation in construction, a two-step contour excavation scheme is proposed, that is to say, after underexcavating 20–30 cm by mechanics, the design contour line is drilled artificially, which effectively alleviates the phenomenon of overexcavation.

In the process of penetration of feet reinforcement bolts, the initial support optimization can effectively solve the problem of surrounding rock disturbance and block falling. At the same time, by reserving grouting holes of backwall grouting, it can effectively solve the problems of incompatibility between initial support and excavating face.

The penetration test of the bolt shows that the disturbance of the bolt to the surrounding rock can be alleviated by canceling the system bolt, and the thinner the bolt is, the easier it is to penetrate. In the case of bolt without the drill bit, the average penetration depth of 25 mm-in-diameter bolt is 1.65 m and the 51 mm-in-diameter bolt is 1.15 m. Without the drill bit, the disturbance of bolt to formation is smaller and easier to penetrate into formation.

The drawing test of the bolt shows that the pull-out resistance of 25 mm-in-diameter rock bolt with the drill bit is 55.2 kN, and the average pull-out resistance without the drill bit is above 74.7 kN. The bolt without the drill bit is generally greater than that with the drill bit. The axial force test of feet reinforcement bolts shows that the stress level of feet reinforcement bolts is much lower than the material yield strength and the tensile force of the anchor is also less than the pull-out resistance. With the increase of the number of feet reinforcement bolts, the stress of bolt tends to decrease, which can increase the restraint of arch foot and the safety reserve.

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 that they have no conflicts of interest.

Acknowledgments

This work was financially supported by the Fundamental Research Funds for the Central Universities, CHD (nos. 300102219723, 300102219716, and 300102219711), Key R&D Projects of China Communications Corporation (no. 2016-ZJKJ-ZX-1201), Project on Social Development of Shaanxi Provincial Science and Technology Department (no. 2018SF-382), and National Key R&D Program of China (no. 2017YFC0805306).

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