Research on design method of composite arch-cover excavation for super shallow-buried long-span metro station

Qichen Jiang*1, Kexian Li2, Peng Liu3 and Zhenjun Wang1

1 Geotechnical and Structural Engineering Research Center, Shandong University, Jinan, Shandong, 250061, China
2 Qingdao Subway Group Co., Ltd., Qingdao, Shandong, 266000, China
3 China Railway Sixth Survey and Design Institute Group Co., Ltd., Tianjin, 300133, China
*Corresponding author’s e-mail: xingzhi_2@163.com

Abstract: Arch-cover method is an effective construction method in upper-soft lower-hard strata, but its applicability in large-span excavation is still problematic. Based on a large-span and super-shallow-buried underground station excavation project of Qingdao Metro, a composite arch-cover method featured with double-layer lining and two-lining was proposed. The whole process of arch-cover excavation was simulated by finite difference method FLAC2D, and the evolution law of structural stress and stratum deformation were obtained. The results show that the stress of the arch cover can be evenly distributed by the interaction of the inner and outer lining, and the formation of concentrated stress zone can be avoided. The bearing capacity of arch cover can be improved, and the generation of lining cracks can be effectively reduced. Stress at the skewback of composite arch cover method should be strengthened. The maximum stress and displacement of rock and soil in the upper part of the station cavern should occur in the process of removing the middle pillar and applying the second initial support. Correspondingly, the monitoring frequency should be increased, and the support should be timely processed. This method is reasonable and feasible, and can provide reference for similar projects.

1. Introduction
The arch cover method transfers the load on the upper rock of cover to the surrounding rock through the skewback, and makes full use of the capacity of the surrounding rock to carry the load. It is a widely used method in urban subway tunnel excavation. In order to meet the requirements of different strata, a series of construction methods base on arch-over method came out successively.

Jia[1] introduced the construction technology and key points of the arch-cover method in detail, described the advantages of the arch-cover method over other construction technologies. Yang et al.[2] applied large-scale model tests to study development mode of surface settlement and the stress of surrounding rock during the excavation of arch-cap method. Du[3] compared the settlement values of arch-cover method under different construction procedures through numerical theoretical calculation. Wu[4] employed three-dimensional finite element method to simulate the construction of arch-cover method in soil-rock composite stratum. Mou[5] carried out comprehensive numerical analysis of stress field, displacement field and plastic zone in each construction procedure, and obtained the mechanical characteristics of arch-cover method. In the terms of the initial support arch-cover and the secondary-
lining support arch-cover method, Zhang[6] and Wang[7] adopted theoretical analysis, model test, field test and numerical simulation to analyze the variation law of stress and deformation of surrounding rock, surface settlement, etc. during the excavation of different arch-cover method. During the construction of metro tunnels in coastal cities, problems such as bad surrounding rock conditions and sensitive surface deformation are often encountered. In this paper, based on the large-span and super-shallow-buried subway station, considering both structural safety and shortening period, a composite arch-cover design method of two-layer initial support combined with secondary lining was proposed, the safety and stability of this method were also verified by numerical calculation.

2. Engineering background

2.1. Project Survey
A station of Qingdao Metro is set up as an island station with three layers of open-cut and two layers of underground excavation. It adopts the structure of single column, double span and two layers of single arch and straight wall. Its main body length was 138.95 m, and its standard section width was 20.3 m. Among them, the length of open-cut section was 60 m, and the length of subsurface section was 86.8m. The buried depth of the underground excavation section was only 5.7~7.3 m, while the maximum excavation span is 24.6 m, so it belongs to a large-span super-shallow-buried station. Crisscrossed municipal pipelines were laid in the upper stratum of the subsurface excavation section. Traffic on the surface was heavy. Therefore, the surface deformation was very sensitive during construction. In addition, in order to ensure the TBM (Tunnel Boring Machine) working in the tunnel between stations can freely pass through this station, the time limit of the construction method was also put forward high requirements.

![Figure 1. General situation of subway station.](image)

2.2. Engineering geology
The stratum distribution in the station area can be divided into miscellaneous filling layer (Q₄ml) and bedrock layer from the surface down. The compactness of the filling layer was poor, which mainly composed of silty clay, weathered sand and some gravel. The filling layer had low strength, poor self-stability, strong permeability, and with thickness of 1.40~4.80 m.

The bedrock was dominated by granite, and some lamprophyre dikes with late intrusion were also revealed. The distribution of rock strata fluctuated greatly. The vault of the underground excavation section was located in intensively and moderately weathered rock strata, and the cave body was located in moderately and slightly weathered rock strata, as shown in figure 2. The physical and mechanical parameters of the rock strata measured are shown in table 1.
Table 1. Physical and mechanical parameters of surrounding rock.

| Number | Classification of Surrounding Rock | Modulus of elasticity (Gpa) | Poisson ratio | Lateral pressure coefficient | Internal friction angle (°) | Natural density (g/cm²) | Permeability coefficient (m/d) |
|--------|-----------------------------------|-----------------------------|---------------|-----------------------------|-----------------------------|-------------------------|-------------------------------|
| 1      | Miscellaneous fill                | 0.01                        | 0.30          | 0.55                        | 10                          | 1.75                    | 15                            |
| 16₁    | Upper subzone of intensively weathered granite | 1                           | 0.24          | 0.32                        | 45                          | 2.30                    | 0.4                           |
| 16₂    | Lower subzone of intensively weathered granite | 1                           | 0.23          | 0.30                        | 50                          | 2.30                    | 0.4                           |
| 17     | Moderately weathered granite      | 5                           | 0.22          | 0.20                        | 55                          | 2.61                    | 0.2                           |
| 18     | Slightly weathered granite        | 22                          | 0.20          | 0.15                        | 65                          | 2.63                    | 0.06                          |

2.3. Hydrogeology

Groundwater in the station area was mainly bedrock fissure water, which was mostly distributed in strongly weathered granite and lamprophyre with abundant fissures and strong permeability. The depth of groundwater runoff was large and water was abundant in the lower part of undulating terrain. The depth of water level was 3.25~6.37 m and the elevation of water level was 0.48~3.54 m. The section of the underground excavation section exposed groundwater, so reasonable waterproofing measures should be taken to prevent the leakage of water in the tunnel.

3. Design methods of cover-arch

3.1. Technical measures for overcoming difficulties

The design method of composite arch cover was adopted as shown in figure 3. Double-layer initial supports were set to ensure the rigidity and stability of the vault and also reduced the sensitive deformation to the ground. The construction of the secondary lining provided sufficient strength
reserve for the tunnel. The excavation of the strata under the arch was carried out by bench-cut method, which shortened the construction period so that the TBM can be driven through the station easily.

3.1.1. Design of excavation scheme. The arch should be constructed by double-sided or ring-shaped drift method, so as to ensure that the steel frame connection joints were in the less stressed section. The lower part of surrounding rock should be constructed by layered excavation with middle groove, and the rate should not be too high for each slope. The width of reserved rock mass in side wall should not be less than 3 m. It was advisable to adopt layer-by-layer blasting construction to reduce disturbance to surrounding rock.

3.1.2. Design of skewback. The foundation beams of skewback should be checked groove before construction, and the bad geology situation should be reinforced. The skewback should be cleaned up, and there should be no poorly compacted dregs, so as to avoid additional settlement, which makes it difficult for the double-layer initial supports to work together. Foundation beams of skewback should be embedded with steel bars in advance and connected with the supporting steel bars of side wall. The first-layer initial support should be installed connecting bar in a certain range of skewback to ensure the coordinated force between the two layers of initial support. Controlled blasting scheme should be adopted in the 5 m range below skewback to reduce the disturbance of blasting to surrounding rock. One layer of surrounding rock with anchor spacing should be excavated at a time and bolt shotcrete support should be carried out in time.

3.1.3. Design of side wall support. Effective measures should be taken to control the over-excavation of side walls, and steel support can be increased if necessary. The middle groove should be used to excavate the lower bench in order to reduce the impact of blasting on side wall. Systematic bolt was used at the side wall, and the length depended on the height of side wall and the condition of surrounding rock. It was required to be more than 3.5 m.

3.1.4. Removal of temporary bracing. The second-layer initial support should be set between the diaphragms to avoid the risk in the removal of bracing. The diaphragms wall should be cleared after the shotcrete strength of the two layers initial support met the design strength. The monitoring frequency should be strengthened and the density of monitoring points should be increased to ensure the safety of initial support according to the monitoring data in time.

![Figure 3. Design of composite arch-cover in this scheme.](image-url)
3.2. Construction process of composite arch-cover method

Figure 4. Construction method of composite arch-cover.
(1) Excavating the rock mass on both sides of upper section by double-sided drift method, erecting grille arch, temporary support and reinforcing beam at the foot of arch cover, then building initial supports on the both sides.
(2) Excavating rock mass in the middle part of upper section, erecting grille arch, then constructing temporary support and initial support.
(3) Constructing the inner-layer initial support, dismantling temporary support, then erecting mid-piller.
(4) Removing mid-pillar, and constructing secondary lining of arch.
(5) Excavating lower section by bench-cut method, and constructing bolt-shotcrete support on side walls.
(6) Finishing the bottom plate, and constructing secondary lining of lower section. After the pass of shield, completing the internal structure and decoration of the station.

4. Design methods of cover-arch

4.1. Numerical mode

Figure 5. Numerical model and diagram of excavation sequence.
FLAC2D software was used to analyze the excavation, which considering the interaction of surrounding rock to structure and bench-cut construction process. The horizontal calculation range of the calculation model was 3 times of the station span, the vertical range was from free surface to the downwards depth of 3 times of tunnel height. The model size was 140 m x 78m, and the tunnel depth was set to 7 m. The computational structure model, as shown in figure 5, generates a total of 13650 elements.

4.2. Calculating parameters
The main physical and mechanical parameters of surrounding rock were based on the measured data as presented in table 1. Parameters of supporting structure are shown in tables 2.
### Table 2. Mechanical parameters of support structure.

| Item                  | Modulus of elasticity $\times 10^4$ N/mm$^2$ | Poisson ratio | Bulk density (kN/m$^3$) |
|-----------------------|---------------------------------------------|---------------|-------------------------|
| C25 Shotcrete         | 2.8                                         | 0.2           | 22                      |
| Ø25 Grouting bolt     | 20                                          | 0.3           | 78.5                    |
| C45 Secondary lining  | 3.35                                        | 0.2           | 25                      |

4.3. Analysis results of stress

The development of maximum principal stress in the whole excavation process are shown in figure 8. The maximum stress was relatively smaller during the left pilot tunnel was excavating, the stress was 1 MPa which equalled to the original in-situ stress. After the right pilot tunnel was excavated, the maximum stress rose to 1.08 MPa, with an increase of 8%. Therefore, the stress in the excavation of the two pilot tunnels was relatively stable. When the middle section of the arch was excavated, the maximum principal stress increased rapidly, which was 32.4% higher than that of the previous procedure. The maximum stress reached to 1.47 MPa when inner layer initial support was building after the finish of outer one. It was because the integral arch structure had not formed yet in the concrete injection of the inner part of the initial support. When excavating the lower half-section rock mass step by step, the maximum principal stress remained basically unchanged with the value about 1.45 MPa. Therefore, the most dangerous section of the whole construction was the rock excavation in the middle part of the arch. During this procedure, temporary support and mid-pillar should be set up in time, and double-layer support was necessary to ensure the stability of the structure.

As shown in figure 7, when excavating the left arch pilot tunnel and building the first initial support, the stress distribution was larger at the position of arch close to the pilot tunnel, with the value about 0.35 MPa. The stress at the bottom of the pilot heading near the temporary support was reduced to less
than 0.2 MPa. The stress near the arch bearing was relatively larger, with the value about 0.45 MPa, and the maximum stress occurred in the minimal range of the skewback, which was 0.98 MPa. The surrounding rock stress far away from the pilot tunnel generally satisfied the law of increasing with depth.

Figure 8 shows the process of excavating the middle rock of the arch and building the first-layer initial support. Arch cover began to play its role because of the removal of original supporting rock in the middle section. When the initial support was applied, the stress was higher than that in other places, with the value distributed in the range of 0.5~0.7 MPa. Because of the bearing effect of integral arch, the maximum principal stress was relatively small in a certain range of span. For example, the stress near bearing increased by 15%~30% compared with the previous procedure. The maximum principal stress 1.43 MPa appeared at the skewback, more than double the high stress area of the arch. Therefore, special attention should be paid to the reinforcement of the corresponding parts, such as setting large size of skewback, tension anchor cables, and encrypting the setting of connecting steel bars.

Figure 9. Stress variation for building second-layer initial support.

When removing the first-layer vertical support of arch and constructing the second-layer initial support and mid-pillar, as shown in figure 9, there was a stress concentration on the top of arch with the maximum value of 1.2 MPa. However, the shape of stress concentration area approximated to linear, and the overall stress distribution of the arch-cover was relatively uniform, indicating that the construction of the inner lining was conducive to the uniform distribution of the stress that was concentrated on the bottom of the arch, and then avoided great deformation at the vault which would induce large subsidence on the surface. The maximum principal stress at the skewback was 1.44 MPa.

Figure 10 (a), (b) respectively shows the process of excavating lower half-section rock and building the first-layer initial support. With the progress of the construction, the stress on the upper part of arch cover gradually decreased, and the stress concentration area gradually disappeared. All these findings indicated that the stress reached the maximum only during the excavation of arch, while the stress both on side wall and bottom plate was small and stable during the construction. Because the side wall was
not supported, the bearing capacity was limited, in order to prevent the side wall from bulging inwards, anchor-shotcrete support was ought to imply. The data strongly show that, stress of the side wall under the support decreased 2~3 times and distributed in the range of 2.5~5 MPa. In addition, due to the stiffness increase of side wall after shotcrete-anchor, stress in arch was further removed, and the stress concentration range near the bearing increased slightly, but the maximum principal stress in total procedure remained about 1.45 MPa.

The maximum principal stress of arch gradually tended to be stable in the construction process of the secondary lining. The excavation and support of the lower half-section had little influence on the stability of the rock arch. Finally, the stress distribution along the interface of the cavern was more uniform, which ensured the long-term stability of the subway station as a whole.

4.4. Analysis results of displacement

Figure 11. Maximum settlement and displacement of arch in excavation.

As can be seen from figure 11, the variation of displacement was similar to that of stress. When excavating both the left and right pilot tunnels, there was no obvious change in the maximum displacement. After excavating the middle part of arch, the surface settlement of the ground and the maximum displacement of the arch increased obviously. After removing the vertical support of first-layer lining of arch, and building the second initial support, the ground settlement and the maximum displacement of arch remained basically unchanged. The final value of settlement was 24 mm with the maximum displacement of 28 mm, which met the safety control standard of the station construction. Therefore, in the construction of the composite lining structure, the excavation and support of the rock mass in the middle of arch were very important for the construction safety of the whole station, and had a significant influence both on the ground settlement and the stability of the arch structure.

Figure 12 shows the process of excavating the right pilot tunnel of the arch and constructing the first layer of initial support. The displacement distribution at the arch and its upper shallow buried soil was more uniform, approximately in the range of 7.5~10 mm, and subsidence zone was distributed in the surface range of about 22 m. The variation was relatively stable and there was no concentrated deformation area. There was a wedge-shaped displacement increasing area near the vertical support of the two pilot tunnels. Its direction was from vertical inclined to tunnel gradually, with the displacement of about 10~12.5 mm. As a result, the vertical support on both sides should have enough stiffness. In addition, when excavating both sides of the pilot headings, there would be a large upward displacement at the bottom of tunnel with the maximum value more than 10 mm.
Figure 12. Displacement variation for excavation of right pilot tunnel.

Figure 13 shows the process of excavating the middle part of arch and constructing the first layer of initial support. In this stage, the stress of the shallow buried soil above the vault presented an anticlinal layered distribution, the thickness of the single layer was about 25 mm. The maximum displacement 22 mm of arch appeared at the position of vault, showing an elliptical distribution, and it can be observed that there was a core area for vertical displacement with the maximum of 25 mm. Meantime, the maximum surface settlement was 18 mm and decreased outward gradually distributed in layer. Therefore, it is the most dangerous stage of overall surface subsidence, and number of monitoring should be encrypted. Besides, due to the vertical support effect, there were large uplifts in the three pilot tunnels, the maximum value was about 22 mm, and original stress concentration area at the skewback disappeared.

Figure 13. Displacement variation for excavation of middle part of arch.

Figure 14. Displacement variation for construction of second-layer initial support.

When removing the first-layer vertical support of arch and constructing the second initial support and mid-pillar, as shown in figure 14, the displacement core area had developed to the vault, and the maximum value of vault was 28 mm distributed in the span range of about 12 m. The core area of
gradient displacement change had developed to the ground, resulting in the maximum settlement of 24 mm. The thickness of the layered displacement distribution zone decreased from 2.5 m in the previous stage to about 2 m near the surface, and the displacement zones of each layer were approximately vertical.

Figure 15. Displacement variation for secondary lining construction of lower half-section.

When the secondary lining of the lower half-section was constructed, as shown in figure 15, the maximum displacement of arch was 28 mm, distributing in the span range of about 8 m near the vault. The maximum settlement of the surface was 24 mm, and the distribution law of subsidence was approximately the same as that during the excavation of lower half section.

In the whole simulation of complete construction process, both the surface subsidence and displacement of surrounding rock changed the most at the process of removing the mid-pillar and constructing the second layer initial support. The displacement distribute strips were relatively dense, i.e., a large gradient might occur on the ground surface in a short distance with a great vertical settlement on the surface soil above vault. As a result, the risk was greatest at this stage, special attention should be paid to monitoring and measuring, and the second-layer initial support should be applied in time. But in general, the settlement and displacement of the above calculation results met the design requirements, so the surrounding rock and composite support structure were all in a safe state in the construction process.

5. Conclusions
(1) Validated by calculation and analysis and engineering practice, the construction of composite arch cover method can meet the requirements of strength and deformation for super-shallow and large-span tunnel. The maximum stress in excavation was about 1.5 times of initial in-situ stress, the final surface settlement value was 24 mm, and the maximum displacement of arch was 28 mm.

(2) The construction of the inner-layer initial support can not only work with the outer one to improve the bearing capacity of the arch cover, but also distribute the stress of the arch evenly. It can avoid the formation of concentrated stress zone, and effectively ensure the integrity of the double-layer support.

(3) In the process of excavating the middle part of rock arch and building the outer layer initial support, the maximum principal stress whose value was more than twice as high as the intensive stress area of arch occurred at the skewback. Therefore, special attention should be paid to the reinforcement of the corresponding parts, such as setting large size of skewback, tension anchor cables, and encrypting the setting of connecting steel bars.

(4) In the process of mentioned above, the displacement of shallow buried strata above the vault presented an anticlinal layered distribution. The maximum displacement occurred at the top of vault, and the maximum settlement appeared meanwhile. While, in the process of installing inner-layer initial support and mid-pillar, the stress reached the maximum value, the displacement layered distribution area continued to develop, and the displacement zones of each layer were approximately vertical. At this time, the monitoring frequency should be increased and the support measure should be timely implemented.
References
[1] Jia, G.B. (2011) Application of arch cover method in construction of metro station. Journal of Cooperative Economy and Science, (12): 125-127. [In Chinese.]
[2] Yang, Z.N, Ji, Z.Q, Liu, Q.W. (2014) Model test study on stability of surrounding rock of tunnel by arch cover method. Modern Tunneling Technology, 51(05): 85-91. [In Chinese.]
[3] Du, Z.J. (2014) Analysis of ground settlement in arch cover construction of long-span tunnel. Railway Standard Design, 58(03): 110-118. [In Chinese.]
[4] Wu, X.F. (2012) Three-dimensional finite element numerical simulation of arch cover construction in soil-rock composite stratum. Research on Urban Rail Transit, 15(08): 135-138. [In Chinese.]
[5] Mou, X.W. (2017) Stability analysis and dynamic risk assessment of arch cover method for metro station caverns. Master thesis, Shandong University, Jinan, China. [In Chinese.]
[6] Zhang, S. J. (2017) Study on mechanical effect and applicability of excavation of initial support arch cover method for rock metro station. Master thesis, Shandong University, Jinan, China. [In Chinese.]
[7] Wang, H.C. (2013) Research on application technology of double-deck initial support in large deformation tunnel. Master thesis, Southwest Jiaotong University, Chengdu, China. [In Chinese.]