Article

Comparative Study on the Influence of Different Forms of New Tubular Roof Method Construction on Railway Tracks

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Abstract: Through the research on the effect of underpass construction using the New Tubular Roof Method on the overlying strata and railway tracks, the characteristics and pros/cons of three forms of the New Tubular Roof Method are analyzed. Based on the geological conditions and structural dimensions of the Taiyuan Yingze Street Underpass Project, by analogy to the structural sections of Shenyang Metro Xinle Ruins Station and Seoul Metro Lot 923 Station, three research cases were designed: “Taiyuan method”, “Shenyang method” and “Seoul method”. The numerical models are established, and the construction process of three cases are simulated. The results show that the three forms of the New Tubular Roof Method have different characteristics. The impact of construction on the rail is mainly reflected in the absolute settlement, while the offset of the rail and the relative displacement between the rails are small, which are not enough to pose a threat to the driving safety. “Taiyuan method” has the best control effect on the deformation of the rails in general, but there is a complex superimposed interference effect during tube jacking, and the settlement in the tube jacking stage accounts for a large proportion. “Taiyuan method” and “Shenyang method” adopt the integrated inner-connecting tube roof structure to cover the entire excavation area, which have excellent effect of isolating excavation disturbance. However, “Shenyang method” has the problem of excessive settlement during the stage of steel tube incision. The settlement caused by the construction of “Seoul method” in the tube jacking stage is relatively small, and there is no need to perform the complicated and dangerous tube incision. However, the excavation disturbance of “Seoul method” will partially escape from the side of the structure, and the excavation influence range is significantly larger than other methods.

Keywords: underground engineering; New Tubular Roof Method; underpass engineering; railway track

1. Introduction

With the increasing concentration of the population in cities, space has become the most important resource in major cities around the world. The demand for the development of urban underground space is increasing day by day. However, there is an inherent contradiction between the development value of urban underground space and the difficulty of construction. High-value underground space is often located in densely populated urban areas with heavy traffic and buildings, and the value of shallow space is generally higher than that of deep space. Unfortunately, the construction of shallow underground space will inevitably disturb the ground environment, which poses a hazard to the safety of various urban facilities. Therefore, one of the core issues faced by the construction of urban underground space is how to complete the construction safely and efficiently on the premise of disturbing the sensitive urban environment as little as possible.

In particular, some urban ground facilities are sensitive to deformation, such as ancient buildings, high-rise and long-span buildings, and rail transit lines. If underground
structures are to be constructed within a very short distance of such facilities, construction methods with strong formation deformation control capability must be adopted. If the span of the structure to be built is small, the pipe jacking method or tunnel boring machine method is commonly used at present. For example, the literature [1] reported an engineering case in which a tunnel boring machine was used to pass three operating rail transit lines in the construction of the Mass Rapid Transit system in Singapore. The literature [2] reported a case of an overlapping double-track subway tunnel using tunnel boring machines to pass a railway line at a short vertical distance. In the above cases, the cumulative settlement of the existing rails during the construction period is all about 10 mm, which shows that the settlement control technology for the small-span tunnel underpass construction is relatively mature. However, if the structure to be built is of a large volume (such as a subway station), the undercutting method is almost the only option. The undercutting method does not have the ability to balance the stratum stress similar to the tunnel boring machine, so it is necessary to control the stress release of the strata after excavation by reducing the single excavation area, applying temporary support, improving the strength and timeliness of the initial support, and strengthening the strata in advance, so as to reduce the deformation of the stratum [3–5]. However, even for the most commonly used “Pile-Arch-Beam method” in China, the stratum settlement caused by its construction generally reaches the order of 10 cm [6]. For rail transit lines, especially ballasted rails, the settlement of this magnitude will not only directly affect the static smoothness of the rails, but also induce the formation of voids in the ballast and void areas at the bottom of the sleepers, resulting in a significant increase in the dynamic load and vibration of the track, uneven stress on the sleepers, and rapid deterioration of the track geometry [7,8]. Therefore, the current construction of large-span underground structures passing through sensitive facilities often requires settlement compensation by means of grouting or mechanical lifting, but these measures are not satisfactory in terms of construction difficulty, effect reliability and cost.

Aiming at the above problems, a special underground engineering construction method with the main feature of using large-diameter steel tube group as the load-bearing structure has been developed. This method first appeared in the 1970s and was applied to the construction of the underground structure of Antwerp Central Station in Belgium [9]. In the 1990s, P Lunardi proposed an underground engineering construction method called “Cellular Arch Method”, which was applied to the VENZIA station in Milan, Italy [10]. The Cellular Arch Method shares many similarities with the construction method of Antwerp Central Station. In both cases, several large-diameter tubes are firstly excavated along the contour of the tunnel arch, and then the tubes are cut and connected by concrete to form a permanent structure. After that, South Korea further developed the technology and named it “New Tubular Roof Method” (hereinafter referred to as NTRM), and adopted this method to complete the famous Lot 923 Station of Seoul Metro Line 9 in 2006 [11,12]. China introduced this method in 2009 and successfully applied it to the Xinle Ruins Station of Shenyang Metro Line 2 [13,14]. Since then, this method has been officially recognized by the Chinese engineering community and has become one of the options for many large-span underpass projects.

The newly completed Taiyuan Yingze Street Underpass Project (hereinafter referred to as the TYSU Project) was built in Taiyuan, the capital of Shanxi Province, China. The project aims to connect the main roads separated by the Taiyuan Railway Station. It is the world’s first NTRM project using a rectangular full-section tubular roof structure. The project successfully formed an underground structure with a single-span span of 18 m and a buried depth of only 3.6 m without interrupting the operation of the existing railway station. This project is another iconic success story of the NTRM. Based on this project, Lei Shengxiang [15] analyzed the impact of the NTRM construction on the existing railway station lines and auxiliary structures. Xiao Dongdong [16] and Liu Wenyu [17] studied the reasonable arrangement and jacking sequence of tubes of full-section tubular roof structure.
Ren Gaofeng [18] conducted a detailed monitoring and analysis of the surface settlement caused during the construction of the project.

Through the successful application and research in the above-mentioned projects, the NTRM has gradually matured as a solution for the micro-disturbance construction of underground engineering in sensitive urban areas. However, looking at the above engineering examples, it can be found that although the current projects using the NTRM follow the same design concept, which is that there is no unified structural design paradigm: the tube roof of the Lot 923 Station only covers the vault, and the inner space of the steel tubes are not connected, concrete rib are used to connect the steel tubes as a permanent structure of the arch. The Xinle Ruins Station uses a tube roof to form arch and side wall, which are connected by cutting and welding. The soil excavation adopts the traditional step undercutting method (with temporary support). After the excavation, the inverted arch is applied to form a closed structure. The TYSU Project adopts an approximately rectangular cross-section. The tube roof wraps the entire tunnel outline. The connection form between the tubes is similar to that of the Xinle Ruins Station. The excavation is protected by the fully enclosed tube roof structure. Obviously, although the construction methods of the above projects are all classified as the NTRM, their construction steps and structural bearing mechanisms are significantly different, which will undoubtedly confuse designers in future projects. In addition, due to the lack of unified conceptual definitions, design principles, design specifications and comprehensive study of existing cases, the NTRM has not yet formed a complete technical system. Thus, it is difficult to obtain sufficient reference and guidance when applying the NTRM in new project, which limits the application of the NTRM.

Therefore, this paper takes the TYSU Project as the main background, referring to the structural form and construction process of Shenyang Xinle Ruins Station and Seoul Metro Lot 923 Station, to study the effect of underpass construction on overlying strata and railway track. The advantages and disadvantages of three forms of NTRM are compared and analyzed to provide reference for subsequent projects.

2. Research Background

2.1. Characteristic of the New Tubular Roof Method

The name “New Tubular Roof Method” is to distinguish it from the traditional tube roof method, which often used in the undercutting method as an advance reinforcement measure. The main features of the NTRM and its similarities and differences with the traditional method are as follows:

- The NTRM generally uses large-diameter steel tubes (about 2 m in diameter), while the tube diameter of the traditional tube roof method is generally less than 1 m, mostly 600~900 mm;
- The NTRM generally adopts a pre-built integrated structure. The tube roof directly forms a permanent structure with concrete filled in, and no extra lining is applied. The traditional tube roof method generally adopts a composite lining design, and the tube roof is only used for advanced support and initial support;
- The NTRM usually adopts cutting-welding and inter-tube grouting to realize the connection between the tubes, while the traditional tube roof method usually adopts the tenon connection.

The NTRM mainly has two core features: one is a large-stiffness integrated load-bearing structure composed of large-diameter steel tubes and concrete, and the other can be summarized as “support first, excavation later”. These two characteristics determine that the NTRM has great advantages in controlling the deformation of strata and ensuring the stability of the excavation face. In addition, the NTRM also has the advantages of being environmentally friendly, low vibration and noise, and freer in section form and size. However, in contrast, the construction process of the NTRM is more complicated, leads to longer construction period and higher cost. These shortcomings limit the application of the NTRM.
2.2. Project Overview of TYSU Project

2.2.1. General Situation of Project

Taiyuan Yingze Street Underpass Project aims to connect the east and west sections of Yingze Street, which are divided by Taiyuan Railway Station. There are 4 platforms, 10 main railway lines and 10 arrival and departure lines in the station. The project is mainly divided into two parts: the north passageway (with a total length of 235 m) and the south passageway (with a total length of 238 m). The minimum vertical distance between the outline of the passageway and the railway bed surface is only 3.5 m. The schematic diagram of the project is shown in Figure 1, and the longitudinal sections of the passageways are shown in Figures 2a and 2b, respectively.

![Schematic diagram of Taiyuan Yingze Street Underpass Project.](image1)

**Figure 1.** Schematic diagram of Taiyuan Yingze Street Underpass Project.

![Longitudinal section of passageways.](image2a) ![Longitudinal section of passageways.](image2b)

**Figure 2.** Longitudinal section of passageways. (a) North passageway; (b) south passageway.

2.2.2. Strata Condition

The strata in the area involved in the project are mainly Quaternary fill soil and new loess, see Table 1 for details.


Table 1. Strata condition of TYSU Project.

| Geochronology | No. | Type            | Color | Status             | Composition                                      | Layer Thickness/m |
|--------------|-----|----------------|-------|--------------------|-------------------------------------------------|------------------|
| Q₄ₘl         | 1-1 | Miscellaneous fill | Grey  | Loose, slightly dense | Construction waste, gravel, loess clay           | 1.5–15.4         |
|              | 1-2 | Plain fill      | Tan   | Low plasticity     | Gravel, loess clay                               | 4.0–9.6          |
| Q₄₉₉₅ₘ + pl  | 2-1 | Loess          | Tan   | High plasticity    | Loess clay, ginger stone, a small amount          | 3.5–15.9         |
|              | 2-2 | Loess          | Tan   | Low plasticity     | of white mycelium                               | 29.9(Not explored to the bottom) |

2.2.3. Cross Section and Structural Form of Passageway of TYSU Project

The passageway adopts a four-lane combination of 3.5 + 3.5 + 3.25 + 3.25 m, with an internal net height of 5.0 m, a total structural width of 18.2 m, and a total height of 10.5 m. The cross-section of the passageway is shown in Figure 3. The passageway is composed of 20 steel tubes with a diameter of 2 m to form a tube roof structure, including 7 tubes each on the ceiling and the floor, and 3 tubes each on the two side walls. The wall thickness of the three steel tubes near the center line of the structure at the ceiling is 30 mm, and the rest are 20 mm. The tube spacing is 165–265 mm. The layout of tubes is shown in Figure 4. The single-section length of the steel tube is 4 m, and the socket-type joint is used for connection.

![Figure 3. Structural section of passageway.](image-url)

![Figure 4. Schematic diagram of steel tube arrangement.](image-url)

2.2.4. Construction Process

The schematic diagram of the construction process of the passageway is shown in Figure 5. The details are as follows:
2.2.4. Construction Process

The schematic diagram of the construction process of the passageway is shown in Figure 5. The details are as follows:

**Figure 5.** Construction process of passageway. (a) Pre-support and tube jacking; (b) tube incision, connection and inner supporting; (c) concrete pouring in tube; (d) excavation and internal structure construction.

1. **Construction of working shafts:**
   Build the departure shaft and receiving shaft of the tube jacking machine, install the reaction walls, guide rails and jacking equipment, etc.

2. **Pre-supporting by the pipe shed:**
   In order to enhance the stability of the strata during the tube jacking process, a row of pipe sheds with a diameter of 180 mm and a spacing of 300 mm are installed along the top and bottom outline of tube roof. The net distance between the pipe shed and the tube roof is not less than 300 mm. See Figure 4 for the pipe shed layout. After the installation of the pipe shed, concrete is filled inside to enhance the rigidity of the pipe shed.

3. **Tube jacking:**
   The tube roof structure of TYSU Project is composed of 20 steel tubes. The number and density of tubes are the highest in the current projects constructed by the NTRM. In order to reduce the superposition effect of unfavorable disturbances when a group of tubes are jacked in, the tube jacking is carried out according to the principle of “from bottom to top, from middle to side, balance left and right, alternate construction at intervals.” The jacking sequence is:
   - No. 1 jacking machine: A14, A16, A15, A18, A17, A19, A2, A1, A3.
   - No. 2 jacking machine: A12, A13, A10, A11, A8, A9, A6, A7, A4, A5.

   The tubes located on the floor and side walls (A8~A20) are jacked by earth pressure balance tube jacking machine, and the tubes located on the ceiling (A1~A7) are jacked with open-type tube jacking machine.

4. **Tube incision, connection and inner support:**
After all the steel tubes are jacked in, the soil between the tubes is grouted. Then, part of the tube wall and the soil between the tubes are cut and removed, the adjacent steel tubes are connected with steel plates with a thickness of 20 mm, and the steel tubes with a diameter of 114 mm are welded between the steel plates for support. The incision and connect are carried out in three layers from bottom to top (as shown in Figure 5c), and each layer is further divided into several sections along the longitudinal direction of the tunnel. The length of each section is 5~6 m in the under-rail area and 8~9 m in the non-under-rail area. In each section, the incision is performed in two spaced jumps, see Figure 6.

![Figure 6. Schematic diagram of tube incision, connection and inner support. (a) Cross section; (b) A-A section view.](image)

5. In-tube rebar binding and concrete pouring:

In the steel tube, rebars are bound and concrete poured to form a permanent main structure. The section division of rebar binding and concrete pouring is the same as that of tube incision.

6. Excavation and internal structure construction:

After the concrete of the main structure reaches the expected strength, the soil is excavated, and the excess steel tubes on the inside are cut and removed. Then, the pavement and interior decoration are carried out.

2.3. Overview of the Projects to Be Compared

2.3.1. Xinle Ruins Station of Shenyang Metro Line 2

Xinle Ruins Station belongs to Shenyang Metro Line 2, just below Huanghe North Street. Huanghe North Street is a two-way 8-lane urban main road with a width of about 50 m. There are dense buildings on both sides of the street and many pipelines below. The minimum vertical distance between the pipeline and the station vault is nearly 1.25 m. Xinle Ruins Station is a single-arch, double-deck, island-type, large-span subway station with no columns on the hall floor. The station is 179.8 m long, with a theoretical clearance width of 23.552 m and a theoretical outline height of 18.203 m. It is constructed using the NTRM. The cross-sectional view is shown in Figure 7. The tube roof of Xinle Ruins Station is composed of two types of steel tubes: 19 tubes with a diameter of 2 m and a wall thickness of 20 mm form the main part of the arch and side wall, and two steel tubes with a diameter of 2.3 m and a wall thickness of 22 mm are located at the junction of the side wall and the inverted arch. The construction steps of Xinle Ruins Station are shown in Figure 8, as follows:
1. Build the working shafts and perform tube jacking. The jacking sequence is from the top of the vault to the foot of the side wall, left and right symmetrically (Figure 8a);
2. Carry out grouting reinforcement of the soil between the tubes and the arch foot area. During this period, the shield machine responsible for the construction of the section tunnel passed through the station from the platform level (Figure 8b);
3. Perform the incision, connection, rebar binding and concrete pouring in the steel tubes on the station hall level (Figure 8c);
4. Perform the incision, connection, rebar binding and concrete pouring in the steel tubes on the platform level (Figure 8d);
5. After the concrete reaches the expected strength, the internal soil is excavated in layers. During this process, the shield segments are removed, and temporary steel supports are arranged at the lower part of the side wall (Figure 8e);
6. Construct the inverted arch, and then complete the internal structure from bottom to top (Figure 8f).
2.3.2. Lot 923 Station of Seoul Metro Line 9

Lot 923 Station is a high-speed bus-subway passenger station in the first phase of the Seoul Metro Line 9 project. It is located in the core urban area of Seoul. There are two existing subway lines, large underground shopping centers, high-rise residences and several busy urban arterial roads near the station. The station is 200 m long, 29.6 m wide and 20.85 m high. It was the largest underground subway station in Korea when it was built. The vertical distance between the vault of the station and the tunnel of Seoul Metro Line 3 is only 15 cm. Above the Metro Line 3 is a large underground shopping mall, which requires extremely high strata deformation control. Therefore, the project contractor decided to adopt the NTRM for the construction of the main structure of the station. Lot 923 Station has two types of sections, see Figure 9 for details. The station adopts steel tubes with a diameter of 2 m (13 tubes for type-I section, 10 tubes for type-II section) to form the arch of the station. After the construction of the side wall, the tubes are connected to each other and the side wall through several transverse ribs, thus forming the permanent structure. The construction steps of the station are shown in Figure 10, and the details are as follows:

**Figure 9.** Structural section of Lot 923 Station. (a) Type-I section; (b) type-II section.

**Figure 10.** Construction process of Lot 923 Station. (a) Tube jacking; (b) excavation of pilot tunnel; (c) in-tube concreting; (d) first stage excavation and concrete rib construction; (e) second stage excavation; (f) inverted arch and internal structure construction.
1. Since the site does not have the conditions for building a working shaft, several long steel tubes with a diameter of 2.5 m and short steel tubes with a diameter of 1.5 m are used to build a rectangular tunnel for the tube jacking. Then, the steel tubes of the station mainframe are jacked in. The stratum near the tubes is grouted for reinforcement (Figure 10a);
2. Excavate two pilot tunnels on the left and right sides under the tube roof, then construct the side walls of the station inside the pilot tunnel (Figure 10b);
3. The side walls are extended upwards and connected to the tube roof, and concrete is poured in the tube (Figure 10c);
4. The soil above the arch line of the tube roof is excavated. Then, the transverse ribs and temporary supports are applied (Figure 10d);
5. The remaining soil is excavated downward in layers to the bottom of the structure. During this, the support of the pilot tunnel is removed, and the inverted arch is poured. Finally, the internal structure of the station is completed.

2.4. Research Plan

This paper sets up the following three research cases:

- Case 1: Closed-type, inner-connecting, near-rectangular tubular roof structure, completely supported first and then excavated. Case 1 is called “Taiyuan method”, taking the TYSU Project as the prototype;
- Case 2: Open-type, inner-connecting, arch-shaped tubular roof structure. Excavation takes place after the tube roof is formed. The inverted arch is applied lastly to form a complete structure. Case 2 is called “Shenyang method”, taking Xinle Ruins Station as the prototype;
- Case 3: Open-type, external-connecting, half-arch-shaped tubular roof structure. The lower section of the structure is constructed through the pilot tunnel, and the inverted arch is constructed with the excavation. Case 3 is called “Seoul method”, taking Lot 923 Station as the prototype.

In order to make the comparative study meaningful, the TYSU Project is selected as the benchmark. The strata conditions, burial depth and surrounding environmental conditions of all cases are designed according to the actual situation of the TYSU Project. The structural size and construction process of case 1 completely simulate the actual situation of the prototype project. As for case 2 and case 3, the structural form and construction process of the corresponding prototype project are retained, and the structural size is redesigned according to the building clearance of the TYSU Project, as shown in Figure 11. Under each case, the steel tubes used in the main structure are 2 m in diameter and 20 mm in wall thickness (except for the three steel tubes near the center line of the structure at the ceiling in case 1, and the two steel tubes at the junction of the side wall and the inverted arch of case 2).

![Figure 11. Structural section of case 2 and case 3. (a) Case 2; (b) case 3.](image-url)
3. Numerical Model

3.1. Basic Assumptions

The numerical model is established based on the following basic assumptions:

- The strata are assumed as continuous homogeneous isotropic medium, distributed in horizontal layers, no relative displacement between layers;
- Steel tubes, concrete and structures such as pipe sheds, bolts, and inner support of the tube roof are assumed to be working in an elastic state during construction, their materials are assumed to be isotropic linear elastic media;
- Since the longitudinal axes of the three prototype project are all straight lines, the cross-sectional forms are symmetrical, and the construction steps are basically symmetrical, the symmetry problem can be considered;
- Since the natural water level of groundwater in the TYSU Project, which is the benchmark project, is lower than the structural floor, the effect of groundwater is not considered;
- The effect of tunnel underpass construction on each railway track is the same and independent of each other, so only one track is selected for research.

3.2. Model Geometry and Meshing

3.2.1. Model of Case 1

The model of case 1 is established with reference to the actual situation of the TYSU Project. According to the basic assumption, a semi-model is established to reduce the computational cost. The model is 99.14 m wide, 104.1 m high and 60 m long. The X axis is parallel to the width direction, the Y axis is parallel to the longitudinal axis of the structure, the Z axis is right-handed with the X and Y axes, and the gravity points to the negative direction of the Z axis. The highest point of the outer edge of the tunnel structure is located at the origin of the model coordinates, with a vertical distance of 3.6 m from the surface. The element type of the model adopts the scheme of 8-node hexahedron element as the main, and 6-node wedge element as the supplement. The overall mesh is an extruded mesh along the Y axis direction. In the main body of the tunnel structure and its surrounding areas, the average mesh sizes are:

- The cross-sectional plane: 0.2 m;
- The axial direction, middle area of the model: 0.5 m;
- The axial direction, near the model boundary: 1.0 m.

In order to further control the degree of freedom of the model, the mesh size of the model is gradually changed in the tunnel cross-section direction and the tunnel axial direction. Finally, 274,684 8-node hexahedral elements and 365 6-node wedge elements are divided, with a total of 284,619 nodes. The model geometry and mesh division are shown in Figure 12. The red numbers in the figure represent the tube jacking sequence in the model.

![Figure 12. Model geometry and mesh of case 1—“Taiyuan method”.](image_url)
Passenger Line has a design speed of 250 km/h and mainly uses China’s “CRH5” EMU. Each group of “CRH5” type train has a length of 211.5 m, a weight of 451 t, and a traction power of 5500 kW. The unit basic resistance of the train is 11.39 N/t, and the unit curve resistance is 700 g/R. The Shijiazhuang-Taiyuan Passenger Line adopts continuous welded ballasted track with a cross-section of “Type-60” specified in the Chinese standard “Code for Design of Railway Tracks” [19]. The track mass is 60.64 kg/m, the track cross-sectional area is 77.45 cm², and the railway track gauge is 1435 mm. The track cross-section is shown in Figure 13a. The sleeper adopts type-III concrete sleeper, 1667 pieces/km, and the size of the sleeper is shown in Figure 13b. The axis of the railway track in the model is parallel to the X axis and is set at Y = 30 m.

![Figure 13](image1.png)

Figure 13. Schematic diagram of rail and sleeper section. (a) Type-60 rail; (b) type-III concrete sleeper.

In addition, the model takes into account the temporary track reinforcement measures adopted in the construction of the TYSU Project. Track reinforcement methods are as follows:

1. Every other sleeper, insert a 3.5 m long wooden sleeper as a beam;
2. Several steel rails (the size is slightly smaller than the main line rail) are spliced together to form a buckle beam and placed on the wooden sleeper. There are 3 buckle beams in total. The buckle beams on two sides of the track are assembled with 3 rails, and the buckle beam in the middle of the track is assembled with 5 rails. The cross-sectional diagram is shown in Figure 14;
3. After adjusting the position of the buckle beam, use the buckle plate and bolts to connect the buckle beam to the sleeper;
4. After the underpass construction is completed, remove the fasteners, buckle beams and wooden sleepers in sequence, then add ballast and tamping, and adjust the shape of the rail.

![Figure 14](image2.png)

Figure 14. Schematic diagram of track reinforcement measures.

In the model, the sleepers are modeled by solid elements, and the rails are modeled by 2-node beam elements. In order to simulate the effect of rail fasteners, fixed constraints are adopted between the rail and sleeper in the X-axis direction (track axis direction) of the global coordinate system, and elastic constraints are adopted in the Y-axis direction (horizontal direction perpendicular to the track axis) and Z-axis direction (gravity direction). The elastic coefficient of constraints is 50 kN/mm in the Z direction, and 25 kN/mm in the
Y direction. The mesh of the railway and the strata are independent of each other, and are associated through hard contact algorithm. The railway model is shown in Figure 15.

![Figure 15. Railway track model and connection nodes.](image)

3.2.2. Model of Case 2 and Case 3

The overall dimensions, coordinate system definition, element type, mesh size, and railway models for case 2 and case 3 are consistent with those of case 1 model. Case 2 model is divided into 275,541 8-node hexahedral elements and 516 6-node wedge elements, with a total of 285,829 nodes. Case 3 model is divided into 292,262 8-node hexahedral elements and 36 6-node wedge elements, with a total of 302,554 nodes. The models for case 2 and case 3 are shown in Figure 16.

![Figure 16. Model geometry and mesh of case 2 and case 3. (a) Case 2—“Shenyang method”; (b) case 3—“Seoul method”.](image)

3.3. Constitutive Model and Material Parameters

3.3.1. Strata

The excavation of strata will inevitably lead to the rise in the average stress and deviatoric stress in the strata, and then generate positive volumetric strain. If the constitutive model of the strata is unable to reproduce the effects of the stress history and the difference in the compression/swelling performance, the bulk strain may be significantly larger than the actual situation. Especially in the kind of shallow underground engineering studied in this paper, the excess volume strain will lead to abnormal phenomena such as uplift on the surface after excavation. To avoid this problem, the strata adopts the Modified Cam-clay Model [20].

The Modified Cam-clay Model is an incremental form elasto-plastic model based on plastic flow theory. Its basic assumptions are as follows:

- The elastic stress–strain relationship of the material obeys Hooke’s law;
- The yielding of the material is only related to the first stress invariant and the second stress invariant, not the third stress invariant;
- Adopts isotropic hardening, and the size of yield envelope is related to the plastic volume compression of the material;
- The associative plastic flow rule is used;
- The plastic work of the material is assumed to be

\[ dW^p = \sqrt{\left( p \, d\varepsilon^p_v \right)^2 + \left( q \, d\varepsilon^p_\lambda \right)^2}, \]

where \( W^p \) is the plastic work, \( p \) is the mean stress, \( p = -\sigma_{ii}/3. \varepsilon^p_v \) is the plastic volume strain, \( \varepsilon^p_v = \varepsilon_{ii}, q \) is the deviatoric stress, \( q = \sqrt{3}J_2 \), \( J_2 \) is the second deviatoric stress invariant, \( \varepsilon^p_\lambda \) is the plastic deviatoric strain, \( \varepsilon^p_\lambda = 2\sqrt{3}J_2/3 \), and \( J_2 \) is the second deviatoric strain invariant.

Under the above assumptions, the projection of yield envelope on the deviatoric plane is circular, while in the \( p-q \) space, it is an ellipse as shown in Figure 17. The yield condition can be expressed as

\[ f(p, q) = q^2 + M^2(p - p_c), \]

where \( p_c \) is the Normal Consolidation Pressure, which will be described in detail below. \( M \) is the material constant. In the \( p-q \) plane, \( M \) is the slope of the Critical State Line (CSL) of the soil.

![Figure 17. Projection of yield envelope on \( p-q \) space.](image)

It can be seen from Equation (2) that when the material constant \( M \) is given, the shape of the yield envelope is uniquely determined by \( p_c \), which is related to the stress history of the material. According to a large number of test results, the stress–volume relationship of soil under isotropic compression/swelling obeys the logarithmic linear relationship shown in Figure 18. In the figure, the ordinate \( v \) is the specific volume, \( v = V/V_0 \), where \( V \) is the total volume of the soil, and \( v_s \) is the specific volume of the solid grain. The straight line CD in Figure 18 is the Normal Consolidation Line (NCL) with a slope of \( \lambda \) in the \( \ln p-v \) space. Rays CA and DB are Swelling Lines (SL) with a slope of \( \kappa \). For a particular state of the material, the abscissa of the intersection of NCL and SL is \( p_c \) of this state. The equations of NCL and SL are

\[
\begin{align*}
\text{NCL} : & \quad v = v_{\lambda} - \lambda \ln \frac{p}{p_1} \\
\text{SL} : & \quad v = v_{\kappa} - \kappa \ln \frac{p}{p_1}
\end{align*}
\]

where \( p_1 \) is the reference pressure, \( v_{\lambda} \) is the specific volume corresponding to the reference pressure on the NCL, and \( v_{\kappa} \) is the specific volume corresponding to the reference pressure on the SL.

Let the initial state of the material at a certain point in the soil correspond to the point A in Figure 18, i.e., \( (\ln p^A, v_{\lambda}^A) \). If the soil is further compressed, the corresponding point of its state in \( \ln p-v \) space will move to the right along the ray CA until it reaches point C. The abscissa of point C is the Normal Consolidation Pressure of the soil, which remains unchanged during the movement of the soil state from point A to point C. If the soil continues to compress, its state will move along the straight line CD, together with the change in Normal Consolidation Pressure. Correspondingly, the projection of its yield envelope in \( p-q \) space will also be changed accordingly. If the load is removed after being
compressed to point D, the soil will swell back to point B along the ray DB, and the Normal Consolidation Pressure stays at the abscissa of point D during the swelling. It can be seen that the NCL of the soil is unique, but there are innumerable SLs corresponding to different normal consolidation pressures. However, the experiment shows that the slopes of different SLs in the In$p$-$v$ space are basically the same. Therefore, $\lambda$ and $\kappa$ are both material constants, which can be obtained from the current stress and volume state of the soil.

Figure 17. Projection of yield envelope on p-q space.

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Figure 18. Normal Consolidation Line and Swelling Line in In$p$-$v$ space.

In summary, the parameters that need to be input to the Modified Cam-clay Model are:

- Material constants: $M$, $\lambda$, $\kappa$.

$M$ can be converted from the effective friction angle $\phi'$ in the Mohr-Coulomb criterion. For the case where the three principal stresses are all pressure, $M$ can be calculated by the following equation:

$$M = \frac{6 \sin \phi'}{3 - \sin \phi'}.$$  

Theoretically, $\lambda$ and $\kappa$ should be obtained through isotropic compression and unloading tests. However, if testing is not possible, $\lambda$ and $\kappa$ can also be obtained through oedometer test. Assuming that the ratio of the horizontal stress $\sigma_H$ to the vertical stress $\sigma_V$ in the test is constant (many test results show that this assumption basically holds during loading, but only approximately holds during unloading), the slope of the $\epsilon$-In$\sigma_V$ curve and the $v$-In$p$ curve are the same. Therefore, $\lambda$ and $\kappa$ can be converted from the compression index and swelling index obtained by the oedometer test as follows:

$$\begin{cases} 
\lambda = C_c / \ln 10 \\
\kappa \approx C_s / \ln 10
\end{cases}.$$  

- Elastic parameters: Poisson’s ratio $\nu$ or shear modulus $G$.

Differentiating the SL line equation (the second term of Equation (3)) and dividing by $\nu$, the elastic volumetric strain of the soil can be obtained as

$$d\epsilon_v = -\frac{dp}{\nu} = \frac{\kappa}{\nu p} dp.$$  

Thus, the tangent bulk modulus of the soil can be naturally determined by the following equation:

$$K = \frac{\nu p}{\kappa}.$$  

It can be seen that the bulk modulus of the Modified Cam-clay Model is an internal variable in the model and does not need to be given additionally. Therefore, only the second elasticity parameter needs to be given. The shear modulus can be automatically obtained from the current tangent modulus when the Poisson’s ratio is given, or vice versa.

- The coordinates of the reference point on the NCL.
When $\lambda$ is already given, a reference point coordinate on the NCL needs to be given to locate the NCL on the $p$-$q$ plane. The best way to determine this coordinate is still the isotropic compression test, but it can also be obtained by the undrained shear test. For a reference pressure $p_1$, the specific volume of the soil in a critical state when $p_c = p_1$ can be obtained from Equation (3) as

$$\Gamma = v_{\lambda} - (\lambda - \kappa) \ln(2).$$

(8)

On the other hand, the undrained shear strength $c_u$, and the corresponding specific volume when shear failure occurs, $v_{cu}$, can determine another set of coordinates on the NCL, so we have

$$v_{cu} = \Gamma - \lambda \ln \frac{2c_u}{M_p}.$$  

(9)

Assuming that the same batch of soil samples are used for the undrained shear test and the porosity test, since the specific volume of the soil remains unchanged during the undrained shear process, it can be considered that $v_{cu} = 1 + \varepsilon_0$, where $\varepsilon_0$ is the natural porosity measured in the laboratory. On this basis, substituting Equation (8) into Equation (9), we can obtain

$$v_{\lambda} = 1 + \varepsilon_0 + \lambda \frac{4c_u}{M_p} \cdot \kappa \ln(2).$$

(10)

After determining $\lambda$ and $\kappa$, and given $p_1$, the reference specific volume $v_{\lambda}$ corresponding to $p_1$ can be obtained from the undrained shear strength and natural porosity through Equation (10).

- Initial stress field and normal consolidation pressure field of soil.

After the above parameters are determined, the Modified Cam-clay Model also needs to input the initial average stress $p_0$ and Normal Consolidation Pressure $p_{N0}$ at each point in the soil to initialize the position and specific volume of the current SL. In general, $p_0$ can be obtained by geo-stress balancing. Therefore, $p_{N0}$ can be obtained by multiplying $p_0$ by the Over Consolidation Ratio (OCR) while the soil is in an isotropic compression state. For the general stress state ($q \neq 0$), by substituting $(p_0, q_0)$ into Equation (2) and deforming, we can obtain

$$p_{N0} = p_0 \left[ 1 + \left( \frac{q_0}{M_p} \right)^2 \right] \cdot OCR.$$  

(11)

In the model, some simplifications are made to the strata. The miscellaneous fill and plain fill in the prototype project are combined into one layer, and a layer of subgrade is added on it. According to the geotechnical survey and test results of TYSU Project, and referring to the literature [21–26], the parameters of the strata material in the model are shown in Table 2.

### Table 2. Material parameters of strata.

| Parameter | Description/Unit | Value |
|-----------|------------------|-------|
| $t$       | Thickness of layer/m | Subgrade | Fill Stratum | Loess (2-1) | Loess (2-2) |
| $\rho$    | Density/kg·m$^{-3}$ | 1900 | 1600 | 1700 | 1780 |
| $\nu$     | Poisson’s ratio   | 0.3 | 0.38 | 0.38 | 0.3 |
| $\varphi$ | Friction angle/°   | 20 | 15 | 18.03 | 16.32 |
| $c_u$     | Undrained shear strength/kPa | 35 * | 13.03 | 42.26 | 48.49 |
| $C_C$     | Compression index  | 0.09 * | 0.18 * | 0.138 | 0.115 |
| $C_S$     | Swelling index     | 0.01 * | 0.009 * | 0.0069 | 0.0053 |
| $\varepsilon_0$ | Natural void ratio | 0.75 * | 0.93 | 0.63 | 0.63 |
| OCR       | Over consolidation ratio | 1.5 * | 1 * | 1 * | 1 * |
| $p_1$     | Reference pressure/MPa | 0.1 |

Note: * means that the parameter is an estimated value.
3.3.2. Building Materials

All building materials adopt the isotropic linear elastic model, and the material parameters are shown in Table 3.

### Table 3. Building material parameters.

| Parameter | Description/Unit   | Value     |
|-----------|--------------------|-----------|
|           | Steel              | Concrete  | Grouted Soil |
| \( \rho \) | Density/kg·m\(^{-3} \) | 7850      | 2300        | 2000        |
| \( E \)   | Young's modulus/GPa | 206       | 34.5        | 10          |
| \( v \)   | Poisson’s ratio    | 0.3       | 0.2         | 0.2         |

3.3.3. Special Element Parameters

In the model, the rails, the pipe shed (used in case 1), and the inner support of the tube roof (used in case 1 and case 2) are simulated by Euler beam elements. The bolts (used in case 3) are simulated by cable elements. The element parameters are calculated according to the section size and material parameters, and the results are shown in Table 4. Among them, since concrete will be injected into the pipe shed for reinforcement after construction, its element parameters are derived from a concrete-filled steel tube section with an equivalent elastic modulus of 56.15 GPa.

### Table 4. Parameters of special elements.

| Parameter   | Description/Unit                  | Value     |
|-------------|-----------------------------------|-----------|
|             | Main Rail                         | Buckle Rail (Middle) | Buckle Rail (Side) | Pile Shed | Inner Support | Bolt |
| \( A \)    | Cross-sectional area/cm\(^2\)     | 77.45     | 329         | 197.4     | 254.5       | 15.48 | 4.9  |
| \( I_y \)  | Moment of inertia alone the main bending direction/cm\(^4\) | 3217      | 10,185      | 6111      | 7490        | 232.4 | /    |
| \( I_z \)  | Moment of inertia perpendicular to the main bending direction/m\(^4\) | 524       | 29,039      | 6109      | 7490        | 232.4 | /    |
| \( I_p \)  | Polar moment of inertia/m\(^4\)  | 3741      | 39,224      | 12,220    | 14,980      | 464.8 | /    |

3.4. Boundary Conditions

The boundary surface perpendicular to the \( Y \)-axis (hereinafter referred to as the \( Y \)-boundary) of the model corresponds to the wall of working shaft in the prototype project, so the symmetrical boundary conditions are adopted initially. After the geo-stress balancing, all translational degrees of freedom on the \( Y \)-boundary are constrained. In addition, the pipe shed in case 1 intersects the \( Y \)-boundary. Considering that the shaft wall has a weak rotation constraint effect on the end of the pipe shed, the rotational freedom of the beam element on the \( Y \)-boundary is not constrained. The other boundaries adopt the boundary conditions commonly used in the static calculation of underground engineering, see Table 5 for details. In the table, \( X \), \( Y \), and \( Z \) represent the translational degrees of freedom along the three axes, respectively. \( X_R \), \( Y_R \), and \( Z_R \) represent the rotational degrees of freedom around the three axes, respectively (only valid for beam elements).
### Table 5. Model boundary conditions.

| Boundary Direction | Boundary Type                  | Degree of Freedom |
|--------------------|--------------------------------|-------------------|
|                    |                                | X     | Y     | Z     | Xₚ   | Yₚ   | Zₚ   |
| X-Positive         | Infinity field boundary        | fix   | -     | -     | -     | -     | -     |
| X-Negative         | Symmetry plane                 | fix   | -     | -     | fix   | fix   | fix   |
| Y-Positive         | Semi-rigid wall                | fix *  | fix   | fix * | -     | -     | -     |
| Y-Negative         | Semi-rigid wall                | fix *  | fix   | fix * | -     | -     | -     |
| Z-Positive         | Free surface                   | -     | -     | -     | -     | -     | -     |
| Z-Negative         | Infinity field boundary        | fix   | fix   | fix   | -     | -     | -     |

Note: * means that the degree of freedom is constrained after the geo-stress balance is completed.

3.5. Initial Conditions and Simulation Flow

The simulation includes the following steps:

1. Primary geo-stress balance:
   - The Modified Cam-clay Model requires the initial stress as the model input parameter. Therefore, the Mohr-Coulomb model is used to carry out a geo-stress balance first.

2. Secondary geo-stress balance:
   - After the primary geo-stress balance, extract the stress components at the centroid of each element in the model to calculate the average stress \( p \). Then, change the constitutive model of the strata to the Modified Cam-clay Model, and input the average stress \( p \) as a model parameter into the corresponding element. Finally, perform the secondary geo-stress balance to remove residual unbalanced forces caused by changing the constitutive model.

3. Tube jacking:
   - Starting from the \( Y = 0 \) plane, with a step length of 4 m, the strata are gradually removed and the steel tube element is activated. The shell element is used to simulate the shell of the tube jacking machine in a range of one step closest to the face. On the interface between the shell and the strata, a distributed force directed to the positive \( Y \)-axis is applied to the strata to simulate the friction force during the tube jacking process. The friction force is calculated by the following equation in reference [11]:

   \[
   P_f = \frac{f \gamma_{avg} D [2H + (2H + D) \tan^2(45^\circ - \phi/2)]}{\pi D} + f \omega,
   \]

   where \( P_f \) is the friction force per unit area on the tube-stratum interface, \( \gamma_{avg} \) is the weighted average density of the strata overlying the tube, \( D \) is the tube diameter, \( H \) is the burial depth of the tube, \( \phi \) is the friction angle of the stratum where the tube is located, \( \omega \) is the weight of the tube per unit length, and \( f \) is the friction coefficient between the tube wall and the stratum.

   In addition, for the tubes that been jacked by earth pressure balance tube jacking machine, the normal pressure should be applied on the face to simulate the balance force, which is calculated by the following equation:

   \[
   P_b = \gamma_{avg} h (1 - \sin \phi),
   \]  

   where \( P_b \) is the balance force per unit area, and \( h \) is the burial depth at the calculation point. Other symbols have the same meanings as before.

   Considering the fast construction speed of tube jacking, 80% stress release rate is considered in the calculation of each jacking cycle to simulate the short-term equilibrium state. After all the tubes are jacked in, all the stress is released, and the model is calculated to equilibrium. Subsequent construction steps also adopt this method and will not be repeated here.

4. Incision, connection, inner supporting and concreting of tube roof:
In order to simulate the incision process, elements of steel tube are partially removed in sections, as shown in Figure 6, and the connecting plates and inner support elements are activated (for cases 1 and 2). The inner support is simulated by two-node cable element. In the model, the length of each section is 4 m in the under-rail area ($Y = 20$ m to $Y = 40$ m), and 8 m in the non-under-rail area. The solid elements inside the tube are then activated to simulate in-tube concrete pouring.

5. Soil excavation and permanent structure construction:

Taking 2 m as step length, the excavation, the application and removal of temporary support, and the construction of the inverted arch are carried out sequentially until the tunnel runs through.

6. Final stress balance:

Fully release the unbalanced force, calculate the model to equilibrium to simulate the long-term balancing state after the construction.

3.6. Measuring Point Layout

There are three measurement lines and three measurement points in the model. As shown in Figure 19, each measurement line is parallel to the $X$-axis and extends from the symmetry plane ($X = 0$ plane) to the opposite $X$-boundary. The measuring points are all located on the symmetry plane. Among them, the No. 1 measurement line and point monitor the surface settlement, the No. 2 and No. 3 measurement lines and points monitor the vertical and transverse displacement (hereinafter referred to as “rail settlement” and “rail offset”, respectively) of the rail. The relative settlement and relative offset between two rails will also be calculated and recorded, defined as

\[
\begin{aligned}
\Delta VR & = \Delta V_1 - \Delta V_2 \\
\Delta HR & = \Delta H_2 - \Delta H_1
\end{aligned}
\]

(14)

where $\Delta VR$ and $\Delta HR$ are the relative rail settlement and relative rail offset, respectively, and $\Delta V_i$ and $\Delta H_i$ ($i = 1, 2$) denote the settlement and the offset of the $i$th rail, respectively.

![Figure 19. Schematic diagram of model measuring point layout.](image)

4. Calculation Results and Analysis

4.1. Case 1

Figure 20 shows the final strata displacement contour and the surface settlement history curve at measuring point 1 of case 1. The abscissa of the history curve is the number of calculation cycle steps. It can be seen from the figure that the width of the settlement tank of case 1 is close to the structural span, and the settlement distribution is relatively uniform along the horizontal direction. The surface settlement mainly occurs in the tube jacking stage and the excavation stage. The final settlement value of the surface at measuring point 1 is 24.39 mm, which is close to the measured maximum surface settlement value of 21.2 mm reported in the literature [13], and it proves that the calculation results are
reasonable. The overall horizontal displacement of the strata is relatively insignificant, and there is only a displacement of about 10 mm in the local area near the side wall, indicating that the structure of case 1 has a good effect in resisting horizontal deformation.

![Diagram]

Figure 20. Calculation results of strata displacement in case 1. (a) Final displacement contour (at Y = 30 section); (b) settlement history curve (at measuring point 1).

Figure 21 shows the settlement history curve and the pie chart of the average settlement of the rail in case 1. Overall, the magnitude and development law of the settlement of the rail is similar to the surface settlement at the midline of the structure. Except for the excavation stage, the relative settlement of the rail is always small, so it can be considered that the rail settlement is mainly the overall displacement caused by surface settlement. It can be seen from Figure 21b that the settlement of the rail of case 1 mainly occurs in the tube jacking stage. The average settlement in this stage is 13.58 mm, accounting for 56.33% of the final settlement. The settlement of the rails caused by excavation and incision is also very considerable, accounting for 21.31% and 16.61% of the final settlement, respectively. The relative settlement of the two rails increases significantly during the excavation stage, reaching a maximum of 1.45 mm. The reason for this phenomenon is that the step length of excavation in the model is 2 m, and the two rails are located at the Y = 29.2825 m section and the Y = 30.7175 m section, respectively, causes the face of each excavation cycle to be misaligned with the rails, leading to the difference in the restraint effect of the face on the two rails. This phenomenon also shows that even under the protection of the tube roof, the impact of excavation on the rails cannot be ignored. The settlement caused by the final balance stage only accounting for 5.76%, which means that the structure and strata will reach a state of equilibrium soon after the excavation is completed. This reflects the advantages of the large-stiffness full-section tube roof structure.

![Diagram]

Figure 21. Calculation results of the rail settlement in case 1 (at measuring points 2 and 3). (a) History curve; (b) pie chart of the proportion of the average rail settlement in each stage.
Figure 22 shows the distribution curves of rail settlement (Figure 22a) and relative settlement (Figure 22b, defined by Equation (14)) on measurement lines 2 and 3 at the end of each stage. It can be seen that in case 1, the impact of underpass construction on the rails mainly occurs within a range of about 10 m from the axis of the structure, which is slightly larger than the span of the structure. The shape of the settlement distribution curve in each stage is basically the same, and the influence range of the excavation on the rail is slightly larger than that in the other stages. On the other hand, it can be seen from Figure 22b that the maximum relative settlement of the rails mostly occurs above the structural axis, and there is a positive correlation between the relative settlement and the absolute settlement on the whole.

Figure 22. Rail settlement distribution curve of case 1. (a) Absolute value; (b) relative value.

Figure 23 shows the rail offset history curve and the distribution curve on measurement lines 2 and 3 of case 1. Obviously, the rail offset caused by the tube jacking and incision stage in case 1 is almost negligible. After the excavation face passes the rail, the rail offset continues to increase until the end of the excavation, the maximum offset during this period is about 3 mm. The offset is partially recovered during long-term stress adjustment. The relative offset of the rails also reaches the maximum in the excavation stage, but its magnitude is small, which is generally not enough to affect the safety of train. Compared with the settlement, the offset value is smaller but the influence range is larger. At the end of the excavation stage, the rails within a range of about 20 m from the structure axis have obvious offset and relative offset, and there is a little bidirectional bending near the structure axis.

Figure 23. Calculation results of rail offset in case 1. (a) History curve (at measuring points 2 and 3); (b) distribution curve in each stage.
Case 1 adopts a special tube jacking sequence, and the settlement of the tube jacking stage accounts for the largest proportion, so it is necessary to conduct a detailed analysis of this stage. Figure 24 shows the histogram of the rail settlement increment caused by the jacking of each tube and the rail settlement distribution curve at the end of tube jacking in case 1. Since the symmetric half-model is used in this simulation, except for the No. 1 and No. 10 tubes located on the symmetrical plane, the settlement increments caused by the jacking of the other tubes should be regarded as the superposition of itself and its symmetrical mirror. Therefore, in the figure, the virtual settlement increment of No. 1 and No. 10 is supplemented by 1 time (the bar with mesh pattern in the figure), so that each tube can be compared under the same standard. It can be seen from the figure that in case 1, the settlement increment caused by the jacking of each tube has no obvious regularity. The maximum settlement increment was 3 mm, accounting for 22.1% of the total settlement of the tube jacking stage, occurs when the No. 7 tube (corresponding to the A9 tube of the prototype project, located in the middle of the side wall) is jacked. Moreover, the jacking process of the No. 6 and No. 4 tubes, which were located on the side wall, and the No. 3 and No. 2 tubes located on the floor produced a settlement increment of more than 1 mm. The settlement caused by the remaining tubes was less than 1 mm. It can be seen from Figure 24b that before and after the jacking of the No. 7 tube, the shape of the rail settlement distribution curve has changed significantly. In the area around 10 m from the center line of the structure, the rail settlement turns to show a decreasing trend, which may be due to the thrust action caused by tube jacking. To sum up, under the tube jacking sequence of case 1, the influence of tube jacking on the rail has a complex superimposed interference effect, and the tube jacking at the side wall and the floor has a significant impact on the rail.

![Figure 24](image-url)

**Figure 24.** Rail settlement analysis diagram of tube jacking stage in case 1. (a) History curve and incremental histogram (mean value of measuring points 2 and 3); (b) settlement distribution curve of each stage.

Overall, thanks to the greater structural stiffness and the construction sequence of supporting first and then excavating, the construction method of case 1 is more effective in controlling the deformation of the strata and rail. Although the final maximum settlement of the rail reaches 24.25 mm, if measured by 10 m chord length, the maximum rail surface height difference is about 11 mm, which exceeds the limit for rail smoothness in the Chinese standard [27]. However, considering the structural span of more than 18 m and the burial depth of only 3.5 m for the TYSU Project, it is very difficult to achieve such a settlement control effect. On the other hand, the settlement of the rail develops relatively gently, and the gauge, direction and height difference in the rails are relatively small during the construction process. As long as the rail is lifted and corrected in time during the construction process, it should not affect the normal operation of the railway. However, the tube jacking stage of case 1 are more complicated, and this stage also has the greatest impact on the strata and rails. The superimposed interference effect of multiple tube jacking
Among them, the settlement increment of the incision stages is the largest, accounting for 56.27% of the final settlement, and the settlement increments of the other stages are relatively average. It is worth noting that the settlement percentage in each stage of case 2 is significantly different from that of case 1.

4.2. Case 2

The final strata displacement contour and the surface settlement history curve of case 2 are shown in Figure 25. Obviously, the influence range of case 2 on the strata is significantly larger than that of case 1, and the maximum settlement of the stratum near the structural vault reaches 54.21 mm, which is much larger than 36.01 mm at the surface. The horizontal displacement of the strata in a large range near the structure is more than 10 mm, and a local peak of 25.64 mm appears near the No. 3 tube. It can be seen that the deformation of the tube roof in case 2 is more obvious, which may be due to the large tube spacing (300 mm) and the thin thickness of the connecting area between the tubes (800 mm), resulting in lower bending stiffness than other cases. In addition, different from case 1, the settlement area of the strata in case 2 extends to the bottom of the tube at the arch foot, which means that the arched tube roof will bear a lot of vertical loads and transmit it to the stratum below. Thus, the grouting quality of the stratum at the arch foot is as high as important. Although the local stratum displacement is large in case 2, the impact of construction on the surface is still controlled within an acceptable range. This is due to the more favorable arched cross-section of case 2.

![Figure 25](image)

**Figure 25.** Calculation results of strata displacement in case 2. (a) Final displacement contour; (b) settlement history curve.

The history curve and the pie chart of rail settlement in case 2 are shown in Figure 26, and the settlement distribution curve is shown in Figure 27. It can be clearly seen that the settlement percentage in each stage of case 2 is significantly different from that of case 1. Among them, the settlement increment of the incision stages is the largest, accounting for 56.27% of the final settlement, and the settlement increments of the other stages are relatively average. It is worth noting that the settlement percentage of the final balance stage reached 15.41%, which was significantly higher than that of case 1. In addition, the relative settlement of the rail in case 2 is relatively obvious. During the incision stage of the arch, the relative settlement is rapidly increased to about 2 mm and remains at a high level during the entire construction process. On the other hand, case 2 has a larger influence range on the rail during the tube jacking stage. Significant settlement occurred on the rails as far as 25 m from the axis of the structure. In the subsequent stage, this range is slightly reduced to about 20 m, which is roughly equivalent to twice the structural span.
Figure 26. Calculation results of the rail settlement in case 2. (a) History curve; (b) pie chart of the proportion of the average rail settlement in each stage.

Figure 27. Rail settlement distribution curve of case 2. (a) Absolute value; (b) relative value.

Figure 28 shows the rail offset history curve and the distribution curve of case 2. In case 2, the tube jacking and incision stages hardly cause the rails to offset. Even in the excavation stage, the rails do not offset more than 1 mm. However, after the structure and the strata reach a complete equilibrium state, the rail has an offset of about 4 mm, equivalent to 1 mm offset measured by 10 m chord length, which is still within the acceptable range. Combined with the higher proportion of rail settlement in the final balance stage of case 2, this result shows that case 2 has a longer stress adjustment period after the construction.

Figure 28. Calculation results of rail offset in case 2. (a) History curve; (b) distribution curve in each stage.
Figure 29 is the histogram of the rail settlement increment and the settlement distribution curve of the tube jacking stage in case 2. It can be seen from the figure that the tubes that cause a large increase in settlement are No. 4, No. 3 (located at the spandrel) and No. 7 (large diameter tube at the arch foot). The maximum rail settlement during the tube jacking is not always above the centerline of the structure, but changes its position continuously. It can be seen that the influence of the tube jacking process on the rail in case 2 is also very complicated, but the total settlement of the stage and the settlement increment of a single tube are both lower than those of case 1.

![Figure 29](image.png)

**Figure 29.** Rail settlement analysis diagram of tube jacking stage in case 3. (a) History curve and incremental histogram; (b) settlement distribution curve of each stage.

In general, when applied to the specific conditions of the TYSU Project, “Shenyang method” has a slightly poorer control effect on the settlement of the strata and the rail than case 1. The final settlement of the rail and the impact range are larger, but still within a safe and controllable range. The main problem of case 2 is the large tube spacing and the insufficient thickness of the connecting area between the tubes, which leads to excessive settlement during the incision stage. However, once the arched tube roof structure is formed, it can effectively isolate the influence of excavation on the strata and rail. Since the parameters such as the tube spacing used in case 2 are directly analogous to the prototype project, if targeted optimization is carried out, more ideal results should be achieved.

4.3. Case 3

The final strata displacement contour and surface settlement history curve of case 3 are shown in Figure 30. Apparently, due to the existence of the pilot tunnel, the strata settlement tank in case 3 is very large, and the horizontal displacement of the strata is concentrated at the side wall of the pilot tunnel. Similarly to case 2, the strata settlement area of case 3 also extends below the structure, but since the structural side wall constructed first in the pilot tunnel has a larger base area, case 3 does not require base reinforcement. From the history curve, the surface settlement of case 3 shows a uniform development trend, and the final surface settlement at measuring point 1 is 34.01 mm, which is close to that of case 2.

The history curve and the pie chart of rail settlement in case 3 are shown in Figure 31, and the settlement distribution curve is shown in Figure 32. It can be seen from the figure that in case 3, there is also a relatively large relative settlement of the rails in the excavation stage, and there is a further increasing trend in the final balance stage. The rail settlement mainly occurred in the excavation stage, accounting for 37.48%, followed by the final balance stage and the tube jacking stage, accounting for 26.84%, respectively. Since the grouting layer of the tube roof has been formed firstly, the excavation of the pilot tunnel has little impact on the rails, and the resulting settlement accounts for only 13.43%. From the settlement distribution curve, the construction of case 3 will have a significant impact.
on the rails within a range of about 40 m to the center line of the structure, but the areas with obvious relative settlement are concentrated in the range of 15 m to the structure centerline. The maximum relative settlement reaches 2.21 m, which is close to the rail height difference limit of 3 mm in Chinese standards.

![Figure 30](image)

**Figure 30.** Calculation results of strata displacement in case 3. (a) Final displacement contour; (b) settlement history curve.

![Figure 31](image)

**Figure 31.** Calculation results of the rail settlement in case 3. (a) History curve; (b) pie chart of the proportion of the average rail settlement in each stage.

![Figure 32](image)

**Figure 32.** Rail settlement distribution curve of case 3. (a) Absolute value; (b) relative value.

The history curve and the distribution curve of rail offset in case 3 are shown in Figure 33. It can be seen that the rail offset of case 3 is also significant in the excavation
stage, and further increases in the subsequent stage. The relative offset also increases significantly, but its value is less than 0.1 mm most of the time. In case 3, the range of rail offset is more than 40 m, which is larger than that in case 1. In addition, it is worth mentioning that at the end of the excavation stage, the maximum point of rail offset is located 10 m away from the centerline of the structure, and during the subsequent stress adjustment process, the point of maximum offset gradually returns to the centerline of the structure. It can be seen that the semi-arched tube roof only has an isolation effect on the excavation disturbance in the area covered by it, while at the side wall without tube roof protection, the disturbance will still be transmitted to the surface through the strata.

![Figure 33. Calculation results of rail offset in case 3. (a) History curve; (b) distribution curve in each stage.](image)

Figure 33 shows the histogram of the rail settlement increment and the settlement distribution curve of the tube jacking stage in case 3. Although the number of tubes that need to be jacked in case 3 is less, the total settlement caused by tube jacking is greater than that of case 2, which indicates that the shape of the tube roof and the tube spacing are closely related to the mechanical response of the tube jacking stage. In case 3, the influence of the jacking of each tube on the rail increases gradually, and the maximum settlement increment occurs during the jacking of No. 4 and No. 5 tubes, which just located at the spandrel. Combined with the analysis results of case 2, it can be inferred that for the arched tube roof structure, the jacking of the tube near the spandrel may have the most significant impact on the strata due to the existence of superposition.

![Figure 34. Rail settlement analysis diagram of tube jacking stage in case 3. (a) History curve and incremental histogram; (b) settlement distribution curve of each stage.](image)

Generally speaking, when applied to the specific conditions of the TYSU Project, the effect of “Seoul method” is similar to that of “Shenyang method”. The final rail settlement
of case 3 is about 35 mm, which is much larger than that of case 1. The semi-arched tube roof structure of “Seoul method” is closer to the advanced pipe shed used commonly in the undercutting method, except that it has a larger radius and higher rigidity. Since the tube roof is not connected to the lower structure until the excavation stage, part of the excavation disturbance will spread to the side of the structure, resulting in a larger settlement area.

4.4. Comparison of Research Cases

Figure 35 presents the histograms of the average settlement, maximum offset, maximum relative settlement and maximum relative offset of the rail under the three cases. It can be seen intuitively from Figure 35a that case 1 and case 2 have obvious advantages in controlling the settlement caused by the excavation stage. The absolute value and the proportion of settlement are significantly ahead of that of case 3. This shows that the integrated inner-connecting tube roof structure that completely covers the excavation area has a significant effect in isolating the excavation disturbance. The cumbersome tube jacking process of case 1 leads to a large settlement during the tube jacking, which is its main disadvantage. Case 2 has excellent settlement control in both the tube jacking stage and the excavation stage, but more than half of its settlement occurs in the incision stage, making it the largest final settlement among the three cases. Case 3 avoids the risky tube incision process, but because it is not the construction mode of “support first and then excavate” in the true sense, the settlement during the excavation stage becomes the main part. In addition, since the permanent structure of case 3 is composed of multiple parts that are implemented in different periods, the structural integrity is poor, and the stress adjustment period after the construction is longer, leading to significant settlement in the final balance stage. Figure 35b–d show that case 1 has the best effect in controlling the rail offset and relative settlement, while case 2 has certain advantages in controlling the relative offset of the rail. However, in general, the influence of the three construction methods on the rail smoothness is mainly reflected in the settlement. The relative settlement and offset of the rail will not significantly affect the safety of the vehicle.

![Figure 35](image_url)

**Figure 35.** Comparison of rail deformation characteristics in different cases. (a) Average settlement (mean value of measuring points 2 and 3); (b) maximum offset; (c) maximum relative settlement; (d) maximum relative offset.

Figure 36 shows the rail settlement and offset distribution curves of the three cases after the final balance stage. Comparing each curve, it can be found that case 1 has comprehensive and obvious advantages over the other two cases. The shape of the rail settlement curve of case 2 is similar to that of case 1, but the range of the settlement area is significantly larger. In addition, the overall rail offset of case 2 is significantly larger than that of other cases, and the offset direction is opposite to that of other cases. The offset control effect of case 3 is generally acceptable, but the influence range of settlement far exceeds that of other cases. To summarize, under the premise that the cost allows, case 1 is undoubtedly the preferred construction method under the background of the TYSU Project.
Case 2 and case 3 have their own advantages and disadvantages. If targeted optimization is carried out, it is also an excellent construction method.

According to the above calculation results and analysis, the advantages and disadvantages of the three forms of NTRM are summarized in Table 6.

| Method       | Advantages                                                                 | Disadvantages                                                                 |
|--------------|----------------------------------------------------------------------------|-------------------------------------------------------------------------------|
| Taiyuan method | - The closed-type integrated pre-support structure has high structural rigidity. During construction, the load is directly transferred to the bottom face of the structure, so the requirement of bearing capacity of the lower stratum is relatively low.  
- The control effect of strata and rail displacement is excellent. Except that the height difference in the rail near the center line of the structure may exceed the limit, it will not seriously affect the smoothness of the rail.  
- It can isolate the impact of excavation on the strata and railway to the greatest extent, and increase the safety of excavation.  
- With rectangular cross section, it is suitable for highway and urban tunnels and has high space utilization.  
- The integrated arch-shaped pre-support structure has better mechanical performance, and the number of steel tubes required to form a section of the same size is less than “Taiyuan method”. It has a good control effect on the strata and rail deformation in the tube jacking stage and the excavation stage.  
- It is suitable for the buildings with high internal clearance and large cross-section, such as metro stations.  
- The excavation method can be selected flexibly, and it is compatible with the construction equipment of traditional undercutting method. | - The number of steel tubes is large; the superimposed interference effect of tube jacking is significant and complex, leads to the difficulty in controlling the strata deformation during the tube jacking stage.  
- If it is applied to projects with larger height or span requirements, the cost may increase exponentially, and the nearly rectangular section form may no longer be applicable.  
- A large number of tube jacking construction makes the cost relatively expensive. |
| Shenyang method | - The settlements caused by each stage are similar, and the settlement development is even and gentle.  
- During construction, the load is reliably transmitted to the base, and the requirements for the reinforcement of the lower stratum are low.  
- There is no need for complicated procedures such as tube incision, in-tube rebar binding and concrete pouring, which are difficult to ensure efficiency and quality. | - The control effect on the smoothness of the rail is slightly poor, especially during construction, the rail may have a large height difference and obvious direction change.  
- During construction, the load is directly transferred to the steel tube at the arch foot, so it is necessary to carry out high-quality grouting reinforcement for the stratum at the foot of the arch. |
| Seoul method   | - Same as “Shenyang method”, it is unfavorable to control the height difference in rails.  
- After the construction is completed, the stress adjustment period is long, and the possibility of additional deformation in the later stage is high.  
- The construction steps of the lower structure are cumbersome. The excavation of the pilot tunnel will expand the disturbance to the strata, causing a larger range of strata deformation.  
- The upper and lower structures are connected in sections, the construction is slightly more difficult, and the structural integrity is poor. As a result, the deformation of the strata and the rails during the excavation stage is relatively large.  
- Thick concrete rib reduces its interior space utilization. |
5. Discussion

This paper relies on three successful cases of the NTRM—the Taiyuan Yingze Street Underpass Project, Shenyang Metro Xinle Ruins Station, and Seoul Metro Lot 923 Station—and analyzes the similarities and differences between the construction methods of the three cases. Through numerical simulation, the influence of the construction on the strata and the railway track is studied when the three construction methods are applied to the background conditions of the TYSU Project. As a special construction method for underground engineering with a short birth time, the NTRM has proven excellent deformation control effect, and a high engineering cost on the other hand. Currently, the very limited NTRM projects are quite different in terms of background conditions, structural forms and construction methods. Thus, it is difficult to compare those projects under the same benchmark. The research in this paper is based on the geological conditions and structural dimensions of the TYSU Project, so the conclusions have certain limitations, which need to be revised and supplemented in future engineering practice and calculation analysis. As for “Shenyang method” and “Seoul method” mentioned in this paper, the research is based on the form of their analogous sections, which cannot represent the prototype project, and are only for reference when selecting construction methods for future projects. In addition, limited by the model scale and computing power, the modeling of the ballast track in this paper is relatively simplified, and the mesoscopic interaction between the ballast and the sleeper cannot be reproduced, which needs to be continuously improved in future research.

However, through the research of this paper, it can be found that there is no absolute difference between the various forms of the NTRM. The feature of flexibility and customizability is also one of the advantages of the NTRM. It is foreseeable that a project using the NTRM in the future is not necessarily limited to the three forms studied in this paper. However, the results of this paper also show that the construction of different forms of NTRM may have very different mechanical responses, which leads to an inevitable increase in the design difficulty and construction risk of the NTRM. The main problems that need to be further studied in the NTRM are as follows: (1) The analysis model of the superimposed interference effect when the large-diameter steel tube group is jacked. (2) The general rule and optimization method of the influence of tube diameter, tube spacing, tube arrangement and jacking sequence on stratum deformation. (3) The adaptability evaluation method of the NTRM. (4) The standardized design and construction process of the NTRM. It is expected that future research will further perfect the NTRM.

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