

Article

Numerical and Experimental Study on the Influence of Large-Section Rectangular Pipe Jacking Construction on Existing Subway Tunnels: A Case Study

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Abstract

With the increasing density of urban underground space development, the soil disturbance induced by large-section rectangular pipe jacking poses a significant threat to the safety of underlying subway tunnels. Taking the Lihe Road utility tunnel project in Wuhan, which crosses over Metro Line 4, as the engineering background, a three-dimensional finite element (FE) model was established using Midas GTS NX to simulate the entire pipe jacking process. Field monitoring data from caisson excavation, ground improvement, pipe jacking, and backfill grouting were introduced for validation, enabling a systematic investigation of the influence mechanism of pipe jacking on existing tunnels. In the numerical simulation, the modified Mohr–Coulomb constitutive model was adopted for the soil, and a “portal-type” reinforcement system was introduced. The pipe jacking process was simulated equivalently with a 1.2 m advance per cycle. The results indicate that the ground settlement induced by pipe jacking exhibits a stage-wise accumulation pattern and eventually develops into a stable settlement trough. The vertical settlement of the tunnel follows an evolutionary law of “early occurrence in the near field, delayed response in the far field, and final convergence,” with peak settlements of 2.44 mm and 2.53 mm for the left and right lines, respectively. Ground improvement significantly mitigates soil deformation, reducing the maximum surface settlement from 45.5 mm to 11.1 mm, decreasing the tunnel’s peak vertical settlement by 37%, and reducing horizontal displacement by 64%, thereby effectively suppressing lateral soil extrusion. The proposed closed-loop analysis method of “numerical simulation–monitoring validation–measure evaluation” reveals the spatiotemporal evolution law of soil–tunnel interaction during pipe jacking construction and provides valuable reference for risk control in similar engineering projects.



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1. Introduction

Over the past three decades, China’s urbanization has led to increasing saturation of aboveground space, making underground construction essential for sustainable urban development in large cities. With the rapid expansion of municipal infrastructure, demand for underground utility tunnels has grown steadily [1–4]. Among construction methods,

pipe jacking is economical, safe, and efficient, promoting its widespread use in urban tunnel projects [5–7]. Disturbance to the surrounding soil during pipe jacking induces ground deformation, and tunneling beneath sensitive structures such as subway tunnels or pipelines may pose safety risks if not properly controlled. Reliable approaches for quantitative prediction and systematic assessment are therefore essential.

Recent studies have examined the effects of rectangular pipe jacking on subway tunnels and adjacent pipelines. Huo et al. [8] investigated twin rectangular pipe jacking effects on shield subway tunnels. Jiang et al. [9] analyzed large-section pipe jacking crossing existing subway tunnels and evaluated anti-uplift measures. Wei et al. [10] proposed methods to predict tunnel displacements under new tunnel crossings. Cheng et al. [11] summarized the main causes of pipe jacking-induced ground deformation. Xu et al. [12,13] studied vertical ground deformation in large-section EPB pipe jacking. Tang [14] reported subway tunnel uplift in Suzhou. Wang et al. [15] proposed analytical solutions for settlement under parallel construction. Feng et al. [16] used 3D FEM to analyze impacts on pipelines. Li et al. [17] investigated gas pipeline deformation, identifying a four-stage evolution pattern. Ma et al. [18] combined field monitoring and numerical simulation for small-clearance pipe jacking. Tian et al. [19] validated reinforcement measures in complex large-section scenarios.

In addition to these pipe jacking-specific studies, recent advancements in the broader field of tunnel-soil-structure interaction—including experimental and numerical investigations of existing tunnel performance under new tunneling, large-diameter pipe jacking effects on adjacent pipelines, and predictive modeling of surface settlement using variational and machine learning methods—provide valuable insights into soil-structure response mechanisms [20–22]. These studies motivate the need for a comprehensive investigation that combines full-process numerical simulation, field monitoring validation, and evaluation of reinforcement measures, as undertaken in this work [23–25]. To further support the investigation of soil-structure interactions, numerical modeling approaches—particularly finite element methods (FEM)—have been widely used in tunneling studies [26]. For example, Wang et al. [27] utilized FEM to simulate pile–soil interaction under seismic loading, analyzing the responses of pile-head acceleration, axial force, and pore water pressure, thereby providing theoretical guidance for pile foundation design. Furthermore, Han et al. [28] applied Midas GTS NX to the deformation analysis and optimization of deep excavation retaining structures, confirming the positive effect of concrete bracing servo control technology in achieving micro-deformation control during construction, demonstrating the reliability of Midas GTS NX in simulating soil–structure interaction and deformation control in underground engineering.

Based on this, the present study takes the Lihe Road utility tunnel pipe jacking project in Wuhan, which crosses over the “Ren-Gong section” of Metro Line 4, as a case study. A finite element model was established to systematically simulate the entire construction sequence, including caisson excavation, ground improvement, pipe jacking, and backfill grouting, in order to quantitatively analyze ground settlement and tunnel deformation characteristics under different stages and working conditions. High-frequency field monitoring data were further employed to validate the numerical results, ensuring the reliability of the simulation framework. On this basis, the effectiveness of reinforcement and construction control measures was evaluated. This study aims to reveal the deformation patterns and underlying mechanisms of large-section rectangular pipe jacking on underlying subway tunnels through a closed-loop framework of “numerical simulation-monitoring validation-measure evaluation,” thereby providing a technical pathway and empirical reference for risk control and tunnel protection in similar engineering projects.

2. Project Overview

2.1. Background

This study is based on the Lihe Road utility tunnel project in Donghu New Town, Wuhan, focusing on the structural impacts arising from its crossing beneath the operating shield tunnel section of Metro Line 4. Donghu New Town is one of the three major sub-centers of Wuhan, serving as a comprehensive urban service hub centered on Wuhan Railway Station. As shown in Figure 1, the project site is located in Hongshan District, Wuhan, and is oriented approximately north–south. The southern section starts at the intersection of Huanle Avenue and Lihe Road, extends northward along the existing Lihe Road to Tuanjie Avenue, then crosses the existing Metro Line 4, and continues northward to Youyi Avenue.



Figure 1. Geographical location of the utility tunnel and its spatial relationship with the shield tunnel.

The total length of the Lihe Road utility tunnel is approximately 2.39 km, constructed using rectangular pipe jacking with a cross-sectional size of $9.6\text{ m} \times 4.4\text{ m}$ and a burial depth of about 13 m. The tunnel adopts a three-compartment layout, with a standard section size of $9.8\text{ m} \times 4.6\text{ m}$, consisting of a comprehensive compartment (clear space $2.6\text{ m} \times 3.0\text{ m}$), a high-voltage power compartment, and a thermal utilities compartment (both with clear space $2.5\text{ m} \times 3.0\text{ m}$). From south to north, the tunnel crosses above the shield tunnel section of Metro Line 4 Phase I between Renhe Road Station and Gongye Fourth Road Station (outer diameter 6.9 m), with a minimum clear vertical spacing of about 5 m and an overcrossing length of approximately 19.6 m. According to Article 60 of the Technical Regulations for Planning and Management of Construction Projects in Wuhan, the proposed pipe jacking project falls within the planning control protection zone of the metro. The working shafts on both sides of the pipe jacking have a minimum horizontal clearance of about 20.1 m from the metro structure, and a dedicated assessment of its impact on the subway tunnel is required.

2.2. Geological and Hydrogeological Conditions

The original geomorphological unit along the project alignment is a fluvial alluvial plain, while the section crossing the subway belongs to the first terrace of the Yangtze River. According to borehole data obtained from the site, the soil and rock strata within the

influence zone of the utility tunnel and subway tunnel, from top to bottom, are classified as: 1-1 fill, 3-1 silty clay, 3-2 silty clay, 3-5 silty clay interbedded with silt and silty sand, 4-1 silt, and 4-2 fine sand, as shown in Figure 2b.

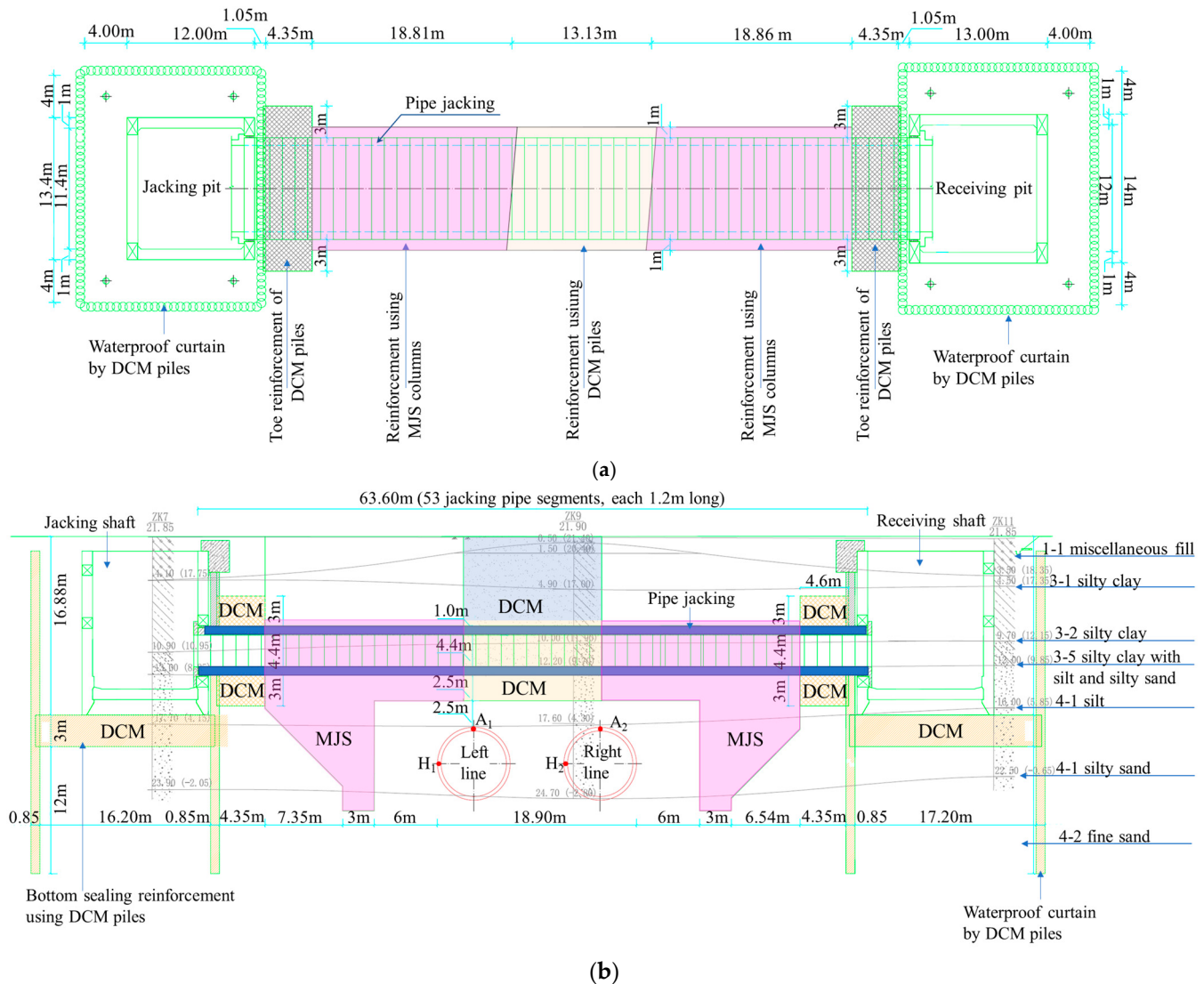


Figure 2. Utility tunnel layout: (a) plan view; (b) cross-sectional view.

Groundwater in the project site mainly consists of perched water and confined aquifers. The perched water occurs in the upper artificial fill layer, recharged primarily by precipitation infiltration and domestic water seepage, and can be pumped out and drained during construction. The depth of the perched water table generally ranges from 0.50 to 5.10 m, with an elevation of 17.04–22.59 m. The confined aquifer is mainly hosted in the Layer-4 fine sand stratum, while the 3-5 silty clay interbedded with silt and silty sand exhibits weakly confined characteristics. According to regional geological data of Wuhan, the Layer-4 fine sand stratum is relatively thick, water-rich, and hydraulically connected to the Yangtze River, with lateral recharge and discharge as the primary groundwater flow processes. The confined aquifer within Layer-4 has a burial depth of 5.01–5.43 m, corresponding to an elevation of 16.05–16.15 m above mean sea level (Yellow Sea datum).

The project extends in a north–south orientation and crosses two geomorphic units. In the 6-1 silty clay interbedded with a silt layer distributed in the second terrace, confined groundwater with weakly confined characteristics is present. Although hydraulically connected to the confined aquifer in the first terrace, this unit lies at a considerable distance

from the Yangtze River (approximately 4.5 km). Due to head loss along the flow path, the confined aquifer in this section exhibits a relatively low piezometric water level.

2.3. Construction Scheme of the Pipe Jacking Process

From October 2022 to January 2023, caisson sinking works were carried out at the Lihe Road utility tunnel across Tuanjie Avenue within the operating metro interval. Subsequently, from February to March 2023, pipe jacking construction was implemented. The on-site construction process is shown in Figure 3.

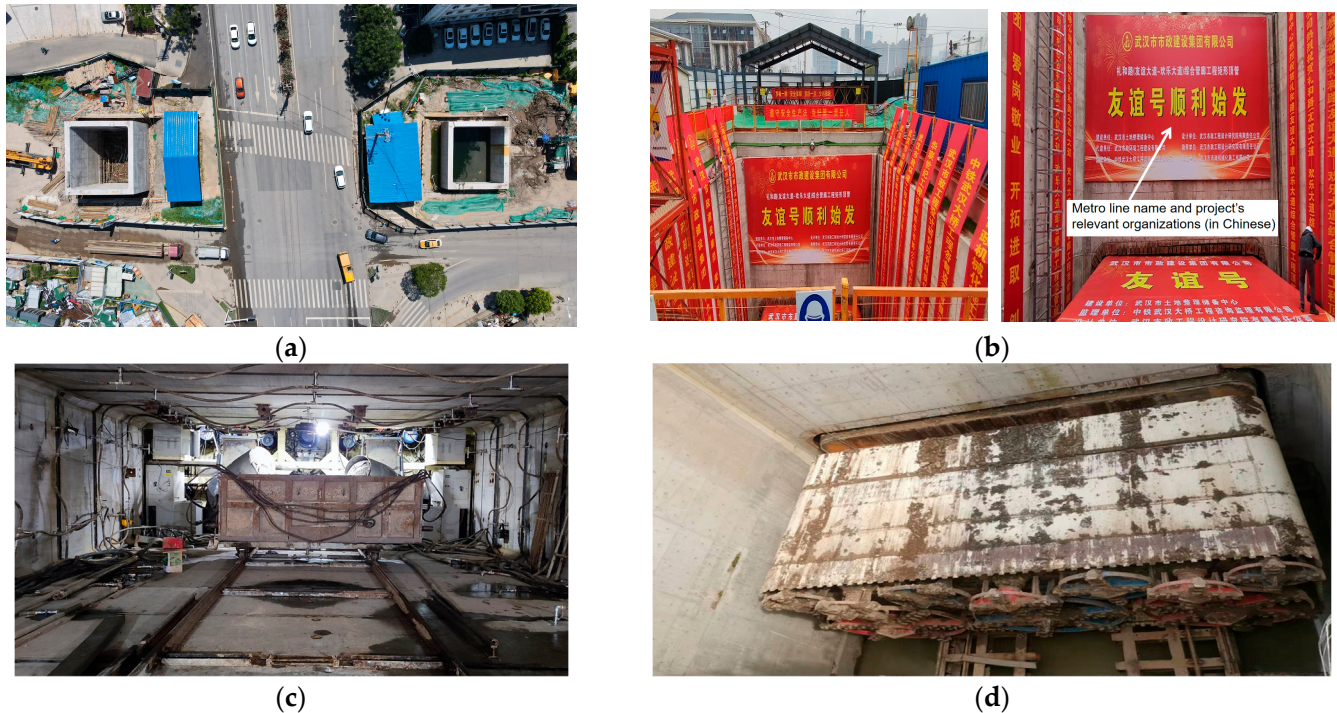


Figure 3. Construction process of the pipe-jacking project: (a) sinking of the caisson shaft; (b) launching of the pipe jacking; (c) jacking operation in progress; (d) breakthrough of the pipe jacking.

To ensure the safe overcrossing of the Lihe Road utility tunnel above the operating “Ren-Gong section” of Metro Line 4, and to control the disturbances induced by construction on the metro structure and surrounding facilities, the contractor adopted a technical scheme combining advanced ground improvement with pipe jacking control and protection measures. The construction was implemented in stages strictly according to the planned schedule (Figure 3). The project was divided into five major phases:

- (1) Construction preparation stage: Preparatory works included detailed site investigation, review of design drawings, installation of the monitoring system, and coordination with the metro operation authority. Control standards for tunnel deformation and an emergency response mechanism were also established.
- (2) Caisson shaft construction stage: Caisson shafts were constructed at both the northern and southern ends of the pipe jacking section. Considering the complex conditions of high soil permeability, abundant groundwater, and close proximity to the metro tunnel, a combined method of “five-side enclosure + curtain dewatering wells” was adopted to ensure excavation safety and effective groundwater control. The “five-side enclosure” refers to the installation of a diaphragm water-sealing curtain formed by Deep Cement Mixing (DCM) around the four sides of the shaft, combined with bottom grouting reinforcement, thereby establishing a three-dimensional water-sealing system to effectively block groundwater seepage. The “curtain dewatering wells” were

installed within the curtain area to actively control the groundwater table and reduce disturbances to the shaft and surrounding strata. Since the caisson was located within a confined aquifer, dewatering could potentially cause adverse effects on the metro structure. To mitigate these risks, particular attention was paid to the safety design of the dewatering system: (i) standby dewatering wells were uniformly installed inside the water-sealing curtain to prevent external hydraulic head expansion; (ii) during construction, the sinking attitude and rate of the caisson were strictly controlled to avoid differential settlement or sudden water inrush that could compromise the metro tunnel's structural safety. This construction method is especially suitable for caisson sinking in complex geological conditions such as proximity to metro lines, densely distributed pipelines, and water-rich sandy strata. The structural layout and reinforcement measures are shown in Figure 2.

- (3) Ground reinforcement stage: To control potential disturbances to the metro tunnel during pipe jacking and ensure the stability of the overcrossing foundation, a composite reinforcement scheme was adopted. This scheme combined the Deep Cement Mixing (DCM) and Multi-axis Jet Grouting System (MJS) methods, arranged in different zones to form a continuous and integrated “portal-type” reinforcement system. Specifically, in the area directly above and at both ends of the pipe-jacking section, DCM columns with a diameter of 850 mm and a spacing of 600 mm (D850@600) were installed. The reinforcement depth was approximately 15.03 m, consisting of a strong reinforcement zone of 7.4 m and a weak reinforcement zone of 7.63 m, which provided rigid support for the existing tunnel and significantly improved the shear strength and deformation resistance of the foundation. In the region above the pipe-jacking section, where multiple underground utilities intersected and construction space was limited, the MJS technique was applied to form inclined grouting piles with a reinforcement thickness ranging from 6.6 m to 7.4 m. The specific layout of the reinforced zones is shown in Figure 2. After completion, the reinforced body was required to be cured for 28 days, ensuring that the unconfined compressive strength exceeded 1.0 MPa and the permeability coefficient was less than 1.0×10^{-7} cm/s. In addition, horizontal exploration boreholes were arranged within the reinforcement area to examine the integrity, uniformity, and water-sealing performance of the strengthened stratum.
- (4) Pipe-jacking stage: During this stage, a total of 53 pipe segments with a length of 1.2 m each were installed, resulting in a total jacking distance of approximately 63.6 m (see Figure 2b). A laser-guided navigation system was employed throughout the process to precisely control the jacking alignment and orientation. In combination with synchronous grouting, short-step advancement, and special deformation-control measures for the metro section, these techniques effectively minimized disturbances to the existing tunnel structure during jacking.
- (5) Post-construction and monitoring stage: After the completion of the jacking operation, subsequent works—including pipe cleaning, structural installation, and backfilling and reinforcement of the working shafts—were carried out promptly. The monitoring program continued until the structural deformations stabilized, ensuring that the metro tunnel remained in safe and normal operation without any abnormal responses.

3. Three-Dimensional Numerical Model

3.1. Model Development

To investigate the disturbance effects of the pipe-jacking construction on the existing metro tunnel structure, a three-dimensional finite element (FE) model was established using Midas GTS NX. Midas GTS NX 2022R1 is a professional FE software specifically designed for geotechnical and underground engineering. It provides robust capabilities

for simulating soil–structure interaction, multi-stage construction processes, and nonlinear material behaviors under complex geological conditions. The software has been widely applied in the numerical analysis of tunnels, foundation pits, and pile foundations.

The numerical model encompasses the existing shield tunnel, pipe-jacking structure, ground reinforcement bodies (DCM and MJS zones), caisson working shafts, and surrounding soil mass, covering the full spatial extent of construction-induced influence (as shown in Figure 4). Based on the excavation depth and disturbance boundary, the computational domain was set to 150 m in length, 55 m in height, and approximately 100 m in width, ensuring that the model boundaries were sufficiently distant to eliminate boundary effects. The soil, concrete structures, and reinforcement elements were all discretized using three-dimensional solid elements. A mesh size of 3 m was used for the surrounding soil, while the tunnel lining, pipe-jacking section, and diaphragm walls were refined to 1 m. Local mesh refinement was applied to these critical regions to improve computational accuracy. The final model consists of approximately 617,354 elements and 106,744 nodes, providing a balance between mesh resolution and computational efficiency.

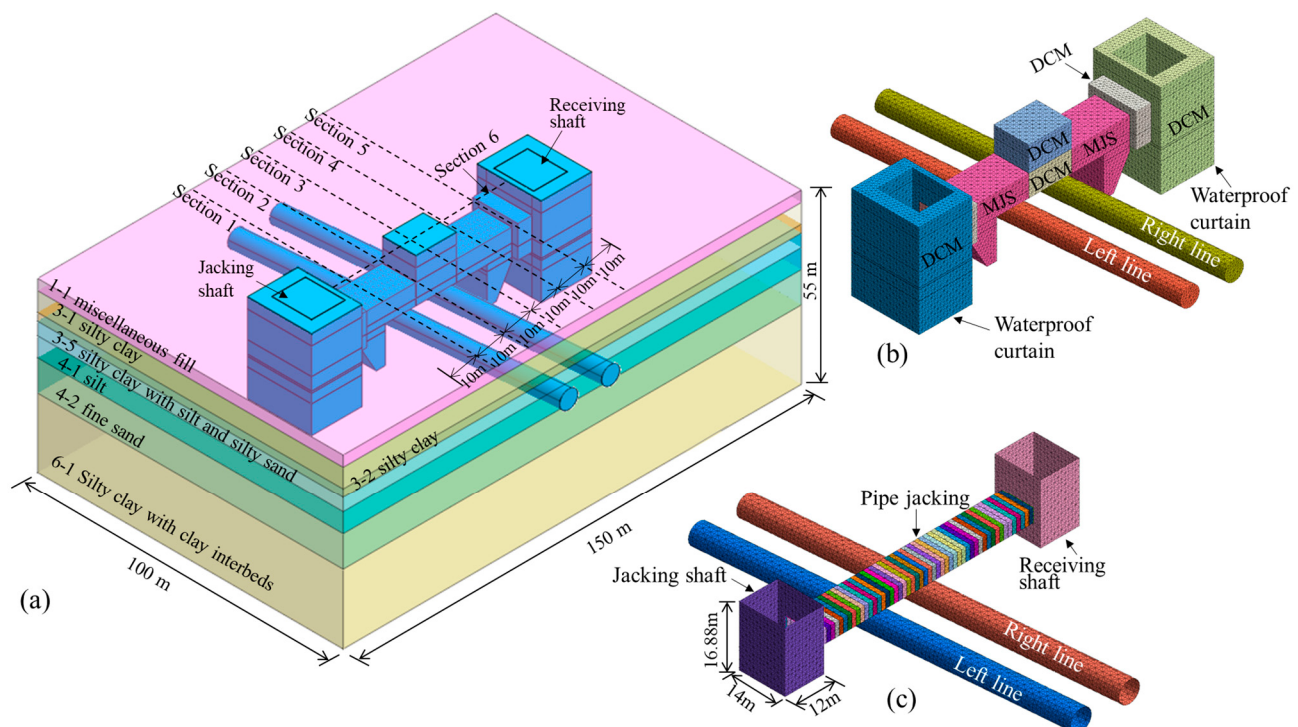


Figure 4. Numerical simulation model: (a) overall view; (b) distribution of reinforcement bodies; (c) tunnel, caisson segments, and shaft wall structures.

A gravitational acceleration of 9.8 m/s^2 was applied throughout the model, and an equivalent surface surcharge of 20 kPa was imposed on the ground surface to simulate traffic and construction loads during the pipe-jacking process. The bottom boundary was fully fixed, while the lateral boundaries were constrained in the normal direction but allowed tangential freedom, minimizing spurious boundary effects on the central analysis region. The initial geostress field was established using the gravity self-balance method to ensure that the initial stress state of the stratum corresponded to in situ conditions. It is noteworthy that although the site lies within a confined aquifer layer, groundwater seepage was not considered in the present model, nor were hydraulic head boundaries applied. This simplification was justified for two main reasons: (1) A combined “five-sided enclosure + curtain dewatering well” method was adopted during construction, forming an integrated seepage-control system in which all dewatering wells were located

inside the cut-off curtain, maintaining stable hydraulic conditions within the excavation pit; and (2) Post-reinforcement tests confirmed that the DCM and MJS reinforcement bodies exhibited adequate compressive strength and very low permeability, effectively impeding groundwater flow and preventing seepage-related effects on the analysis.

3.2. Material Parameter Configuration

3.2.1. Constitutive Model and Parameters of Soil

According to the detailed geotechnical investigation report, the stratigraphy within the modeling domain primarily consists of fill (1-2), silty clay (3-1, 3-2), silty clay interbedded with silty soil and silty sand (3-5), silt (4-1), fine sand (4-2), and silty clay with clay interbeds (6-1). To reasonably capture the nonlinear response of the soil during pipe-jacking-induced disturbance, the Modified Mohr–Coulomb (MMC) model was adopted for all soil layers. Compared with the conventional Mohr–Coulomb model, the MMC model introduces distinct unloading/reloading moduli, thereby overcoming the limitation of a single stiffness and providing improved accuracy in simulating complex deformation behaviors such as ground heave and base uplift.

The physical and mechanical parameters of the soils were determined based on the geotechnical site investigation report and calibrated with reference to similar engineering projects. The adopted parameters include natural water content, unit weight, cohesion, internal friction angle, Poisson’s ratio, and elastic modulus. In the MMC model, the dilation angle ψ controls the volumetric response during plastic shearing rather than the shear strength. For the studied urban soil layers, including unconsolidated fill, silty clay, and silty sand, dilatancy effects are usually weak and often neglected in engineering practice. Therefore, a non-associated flow rule with $\psi = 0^\circ$ was adopted. The parameters used in the analysis are listed in Table 1.

Table 1. Geotechnical Parameter Values.

Soil Layer Name	Constitutive Model	Water Content ω (%)	Unit Weight (kN/m ³)	c (kPa)	ϕ (°)	Poisson’s Ratio	Elastic Modulus (MPa)	Thickness (m)
2-1 Unconsolidated Fill	Modified Mohr–Coulomb Model	34.8	18.4	8	18	0.35	5.0	3.50
3-1 Silty Clay	Modified Mohr–Coulomb Model	28.5	19.1	20	20	0.38	9.0	6.16
3-2 Silty Clay	Modified Mohr–Coulomb Model	33.5	18.5	20	18	0.38	7.5	2.50
3-5 Silty Clay with Silt and Sand	Modified Mohr–Coulomb Model	33.2	18.3	15	23	0.35	10.0	4.10
4-1 Silty Sand	Modified Mohr–Coulomb Model	/	18.5	0	28	0.30	15.0	6.24
4-2 Fine Sand	Modified Mohr–Coulomb Model	/	18.8	0	30	0.28	20.0	10.00
6-1 Silty clay with clay interbeds	Modified Mohr–Coulomb Model	31.2	18.6	20	18	0.35	15.0	22.50

3.2.2. Modeling Parameters of Concrete Structures

In the numerical simulation, considering that concrete structures generally exhibit high stiffness, low deformability, and good material homogeneity, they are typically in an elastic working state during the construction stage. Therefore, the Linear Elastic Model was adopted to simulate the metro tunnel lining and the caisson shaft wall [29–31]. This model is based on a linear relationship between stress and strain and can effectively reflect the deformation behavior of the structure within the normal stress range.

The key parameters of the linear elastic model include density (ρ), elastic modulus (E), and Poisson's ratio (ν). The parameter values were determined with reference to the Code for Design of Concrete Structures (GB 50010-2010) [32], Load Code for the Design of Building Structures (GB 50009-2012) [33], and Technical Code for Ground Treatment of Buildings (JGJ79-2002/J220-2002) [34], combined with the actual structural design parameters of this project. To reasonably reflect the stiffness of the caisson shaft wall during the construction phase, a bending stiffness reduction factor of 0.85 was applied in the calculation. Although the DCM and MJS reinforcement bodies are not conventional concrete structures, they were treated using equivalent elastic parameters in the numerical model due to their similar material and mechanical characteristics. The physical and mechanical parameters of each structure are listed in Table 2.

Table 2. Structural mechanical parameters.

Structural Name	Density ρ (kg/m ³)	E (MPa)	Poisson's Ratio
Caisson shaft wall	2500	30,000	0.20
Jack-in pipe	2500	32,000	0.20
Deep Cement Mixing column	2000	2000	0.20
Multi-Jet System grouting column	2000	2000	0.20

3.3. Numerical Simulation of the Full Process of Pipe Jacking Construction

A three-dimensional finite element model was established to simulate the influence of the pipe-jacking construction crossing above the existing “Ren-Gong” shield tunnel section. Considering that the caisson shaft wall possesses high structural stiffness and induces limited disturbance during sinking, while the main focus of the analysis is on the impact of the pipe-jacking process on the underlying metro tunnel, the caisson construction was simplified in the numerical model by activating the shaft wall elements in a single step.

The pipe-jacking process was simulated by sequentially advancing the jacking segments with an equivalent step length of 1.2 m. In total, 53 jacking steps were simulated, and the detailed construction sequence is illustrated in Table 3 and Figure 5. During each jacking step, the structural elements of the newly installed segment were activated, while the soil elements ahead of the tunnel face were deactivated to represent cutterhead excavation. The interaction between the jacked culvert and the surrounding soil was modeled as frictionless, reflecting actual construction conditions in which the cutterhead over-excavates the annulus and lubrication slurry, together with synchronous grouting, reducing contact stress and shear transfer. In addition, frictionless contact represents a conservative, worst-case assumption for deformation because previous studies Vilca et al. [35] have shown that higher friction tends to reduce displacements.

Table 3. Key construction stages for pipe jacking.

Step No.	Simulation Stage	Description of Simulation
1	Initial Geostress Equilibrium	Establish the initial stress field considering the self-weight of the soil and surface surcharge. Gravity balance is achieved, and soil deformation is controlled within $< 10^{-5}$ m.
2	Metro Tunnel Structure Activation	Activate the shield tunnel lining elements and deactivate the surrounding soil to represent the existing metro tunnel structure.

Table 3. Cont.

Step No.	Simulation Stage	Description of Simulation
3	Ground Reinforcement Simulation	Replace the corresponding soil zones with equivalent DCM and MJS elements to simulate the formation of the “portal-type” composite reinforcement system.
4	Caisson Excavation	Directly excavate the soil within the caisson working pit and activate the shaft wall elements to simulate the caisson construction in a single step.
5	Sequential Pipe Jacking	Advance the jacking segment by 1.2 m per step, deactivate the soil ahead, and activate the current pipe segment and support elements until all 53 segments are installed.

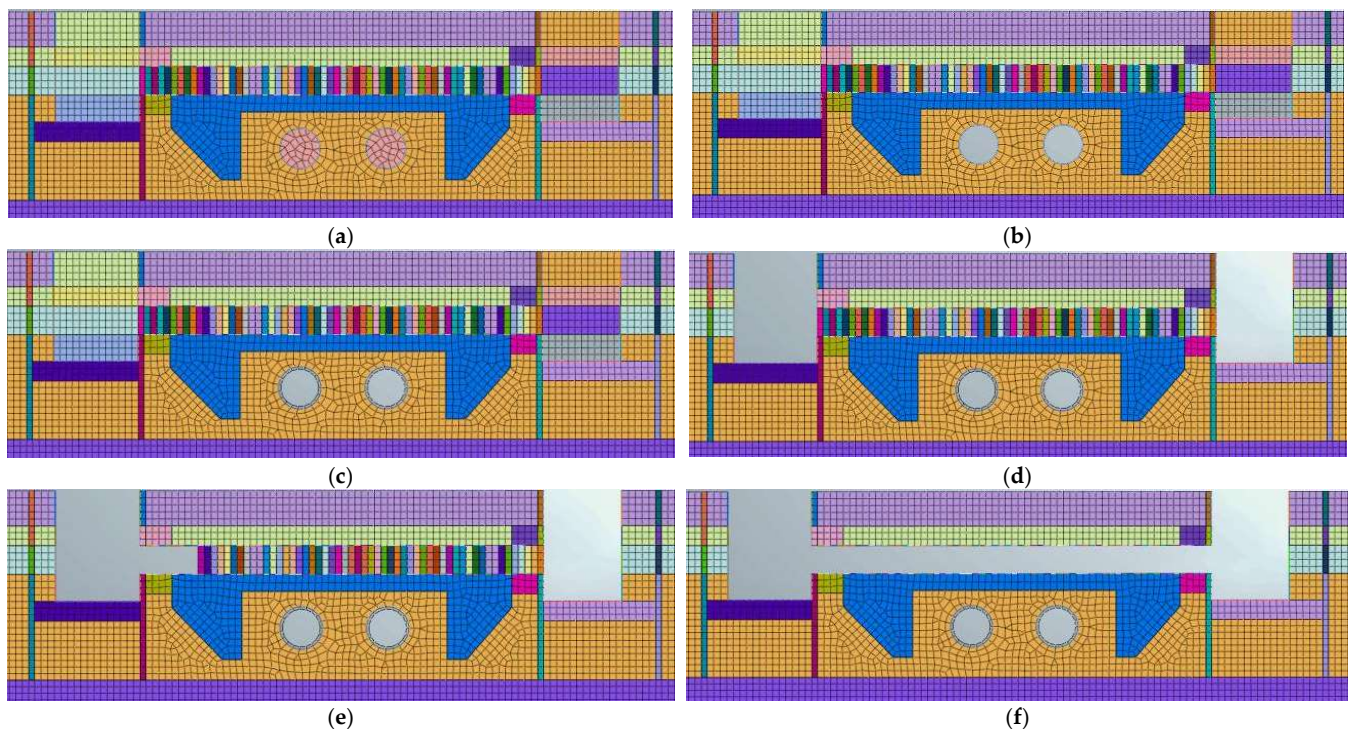


Figure 5. Numerical simulation procedure of caisson excavation: (a) initial stress equilibrium; (b) activation of metro tunnel structure; (c) ground reinforcement; (d) caisson excavation; (e) stepwise pipe jacking; (f) excavation completed.

4. Results

4.1. Analysis of Soil Deformation Characteristics

To systematically investigate the surface deformation induced by pipe-jacking construction, five result-extraction sections (Sections 1–5) were established at 10 m intervals along the tunnel axis in the numerical model, with their deformation results shown in Figure 6. In addition, Section 6, located directly above the tunnel cross-section, was used to examine surface settlement along the jacking direction, as presented in Figure 7. Their spatial arrangement is illustrated in Figure 4.

At the initial stage of jacking, i.e., the excavation reached 9.6 m (Figure 6a), noticeable settlements occurred in the near-field sections, Sections 1 and 2, with maximum values of approximately 4.7 mm and 0.8 mm, while displacements in the far-field sections (Sections 3–5) remained small. Settlements were concentrated near the excavation face, indicating that the soil in the near field experienced local stress release under excavation and pipe–soil friction, while far-field soil remained largely unaffected, showing delayed response. Similarly,

Section 6 shows that the surface settlement peak occurred about 10 m ahead of the working face, with negligible settlement beyond 20 m (as shown in Figure 7), reflecting that the disturbance was initially confined to a narrow zone around the face.

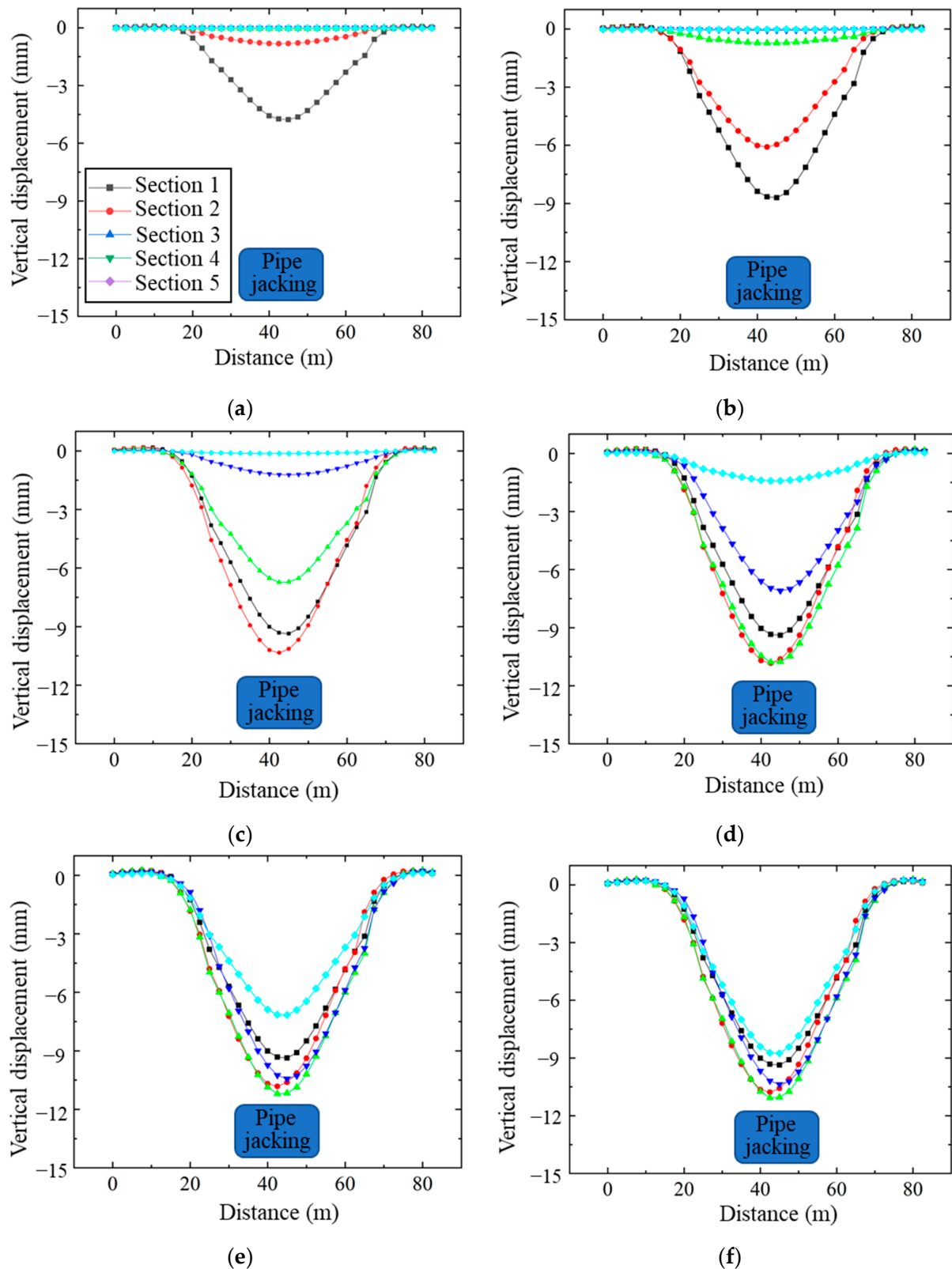


Figure 6. Ground surface settlement along five monitoring sections at different pipe jacking advancement stages: (a) 9.6 m; (b) 19.2 m; (c) 30 m; (d) 40.8 m; (e) 51.6 m; (f) 63.6 m.

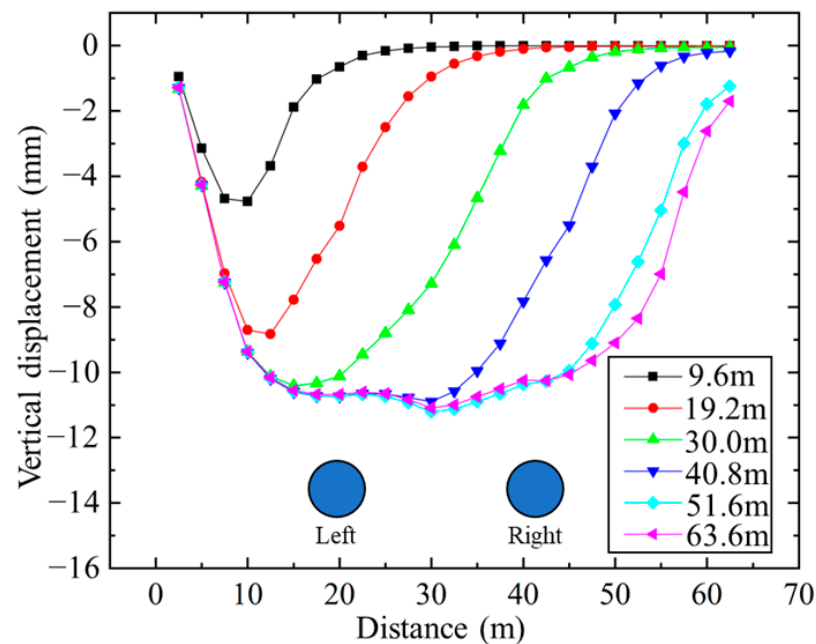


Figure 7. Ground surface settlement above the tunnel at different pipe jacking advancement distances (9.6 m, 19.2 m, 30 m, 40.8 m, 51.6 m, and 63.6 m).

As the excavation progressed to 19.2 m (Figure 6b), settlements in Sections 1–3 deepened, exceeding 8 mm, and the settlement area expanded significantly. Sections 4 and 5 also began to show noticeable settlement. The peak of surface settlement along the jacking axis (Section 6) shifted 10–15 m ahead of the working face, with a maximum of 8.7 mm and an influence zone extending to approximately 40 m. This stage shows rapid accumulation in the near field and gradual response in the far field. Near-field soil volume loss became significant, while disturbance waves propagated through the surrounding soil, causing progressive deformation in the far field. Grouting and ground improvement partially compensated for settlement but could not fully offset the cumulative loss.

At 30 m (Figure 6c), both the magnitude and spatial extent of settlements increased further. Sections 1–3 reached a maximum settlement of 10.33 mm, and Sections 4 and 5 exceeded 1 mm, with the settlement trough gradually connecting. Section 6 shows the maximum surface settlement exceeding 11 mm, with the trough deepening and widening, and the influence zone reaching 50 m. This stage reflects a continuous process of soil disturbance—volume loss accumulation—stress redistribution, in which near-field soil experiences rapid settlement, deeper layers gradually deform under the disturbance wave, and the overlying soil adjusts under redistributed stress, forming a longitudinal settlement gradient.

At 40.8 m and 51.6 m (Figure 6d,e), the settlement trough became most pronounced. Peak settlements in Sections 1–3 exceeded 10 mm, Section 4 reached 7 mm, and Section 5 was approximately 1.4 mm. Both longitudinal and transverse shapes of the settlement trough stabilized, with the transverse distribution consistent with Peck’s empirical curve [36], forming a typical settlement basin. Along the jacking axis (Section 6), the peak remained about 30 m ahead of the face, and the influence zone extended beyond 60 m. At this stage, near-field disturbance had fully developed, far-field soil gradually responded, and grouting and ground improvement partially offset settlement, maintaining the trough’s peak and shape. The settlement center remained near the tunnel axis, indicating effective control of pipe alignment.

By 63.6 m (Figure 6f), the overall settlement level was similar to that at 51.6 m, with peaks of 9–11 mm and a flattened curve, indicating that settlement had reached a stable

stage. After earlier disturbances, stress release, and gradual rebalancing, the soil's vertical and transverse stresses approached a new equilibrium, and the cumulative effect of volume loss weakened, stabilizing the trough width and depth.

Overall, surface settlement induced by pipe-jacking along the tunnel exhibits limited settlement at the early stage, rapid accumulation in the near field, delayed response in the far field, and eventual stabilization. The settlement trough gradually deepens and widens with jacking progress, ultimately forming a peak-stable settlement basin. The evolution of vertical soil stress along the pipe axis at 1 m above the pipe crown further explains this settlement pattern. Figure 8 shows that at early jacking stages, stress redistribution is concentrated near the launching shaft, with soil ahead of the pipe remaining close to in situ stress. As the pipe advances, the zone of stress concentration progressively propagates toward the receiving shaft, with larger compressive stresses accumulating behind the jacking face. This asymmetric stress evolution confirms that settlements initiate near the starting point and develop sequentially along the tunnel axis, reflecting the sequential excavation process and cumulative stress redistribution in the surrounding soil.

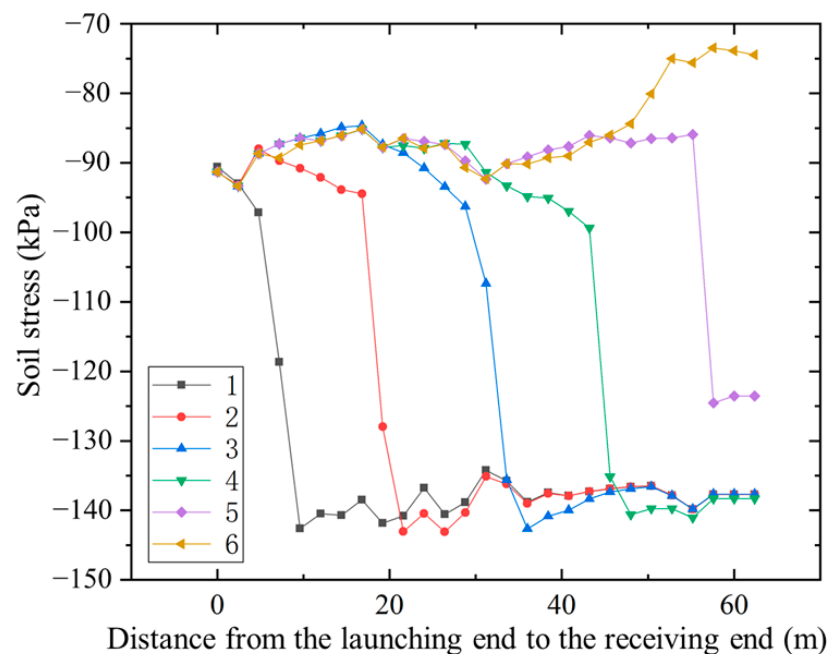


Figure 8. Vertical soil stress redistribution along the pipe axis at 1 m above the pipe crown at different pipe jacking advancement distances (9.6 m, 19.2 m, 30 m, 40.8 m, 51.6 m, and 63.6 m).

4.2. Tunnel Deformation Characteristics

4.2.1. Vertical Deformation of Tunnels

During the pipe jacking process, the two tunnels located beneath the initial jacking alignment exhibited distinct displacement responses. The left-track tunnel is located 25.7 m from the jacking launch shaft, and the right-track tunnel 37.7 m from the launch shaft, both approximately 5 m below the pipe axis. As shown in Figure 9, when the excavation advanced to 9.6 m, a measurable vertical displacement had already developed in the left-line tunnel, with a maximum settlement of about 0.202 mm, while the right-line tunnel remained almost unaffected. At this stage, construction-induced disturbance was mainly confined to the vicinity of the excavation face. Stress release and volumetric loss accumulated preferentially in the near field, causing the left-line tunnel to enter the deformation adjustment stage earlier, whereas the right-line tunnel largely retained its initial stress state.

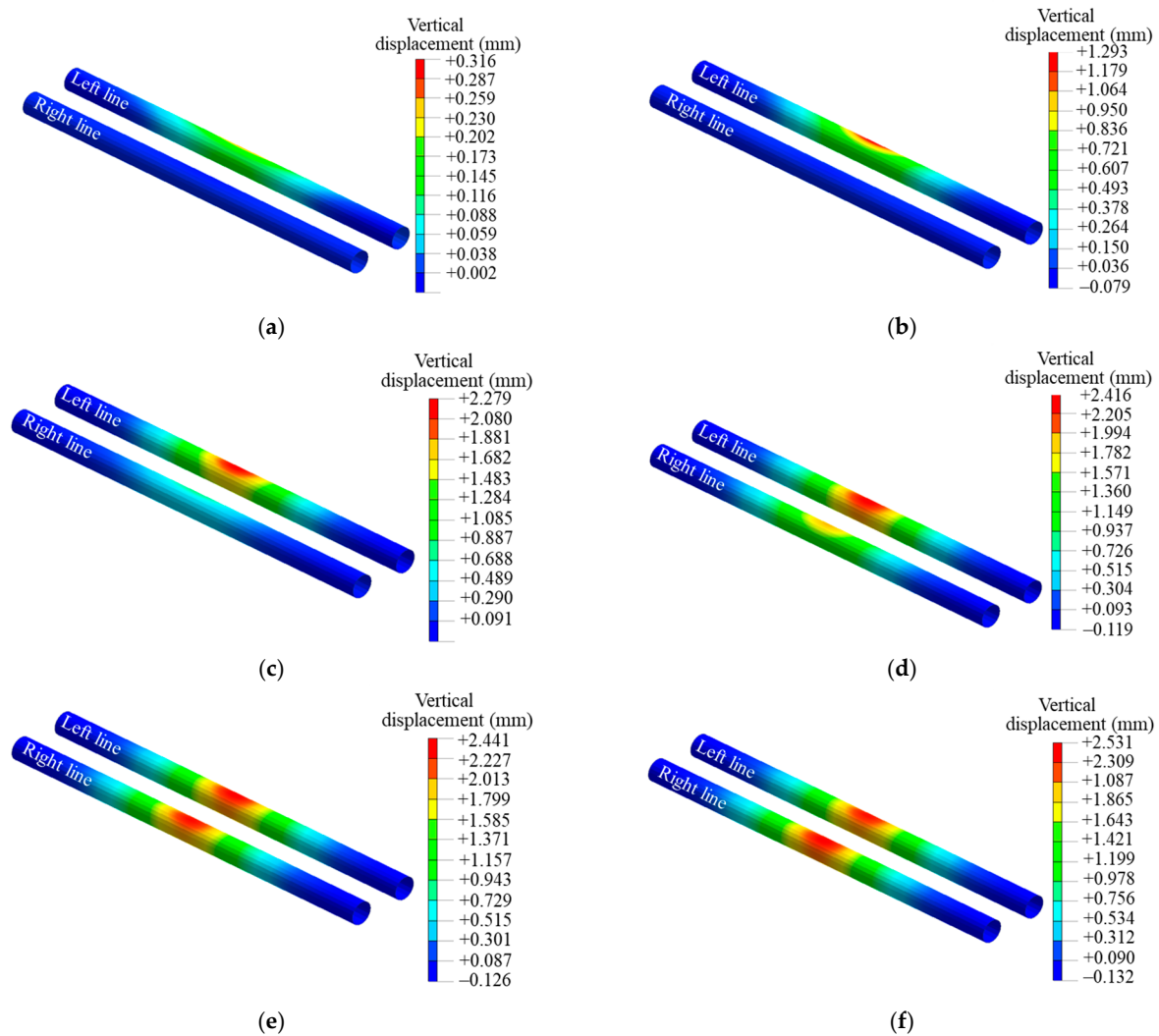


Figure 9. Contour maps of tunnel settlement deformation at different pipe jacking advancement distances: (a) 9.6 m; (b) 19.2 m; (c) 30 m; (d) 40.8 m; (e) 51.6 m; (f) 63.6 m.

When the jacking distance increased to 19.2 m, settlement of the left-line tunnel continued to develop, reaching a maximum of 1.293 mm, while no pronounced response was observed in the right-line tunnel. This indicates that disturbance propagation along the jacking direction exhibits a clear spatial lag, with its influence gradually extending but not yet fully reaching the farther tunnel. At the 30 m stage, the settlement zone of the left-line tunnel expanded significantly, and the peak value increased to 2.279 mm. Meanwhile, the right-line tunnel began to exhibit measurable settlement, with a maximum of approximately 0.489 mm, suggesting that stress redistribution in the surrounding soil had progressively affected the far-field tunnel.

As excavation proceeded to 40.8 m, settlements in both tunnels continued to increase, with maximum values of 2.416 mm for the left line and 1.994 mm for the right line, and the difference between the two became notably smaller. At this stage, construction-induced disturbance had accumulated over a longer jacking distance, and the overlying soil of both tunnels had experienced substantial stress adjustment, leading to increasingly similar deformation trends. When the excavation advanced further to 51.6 m and 63.6 m, the maximum settlements of the left and right tunnels reached 2.441 mm and 2.531 mm, respectively, and the difference between the two became negligible.

These results indicate that the tunnel displacement response is governed not only by its relative position but also by the spatial propagation and cumulative nature of construction-

induced disturbance in the soil. The near-field tunnel responds earlier and develops settlement more rapidly, whereas the far-field tunnel exhibits a delayed response. However, with the continued advancement of the jacking face, disturbance effects progressively expand and superimpose, and the soil gradually approaches a new global equilibrium state, ultimately resulting in comparable settlement magnitudes for both tunnels.

4.2.2. Analysis of Tunnel Horizontal Deformation

During pipe-jacking advancement, the two underlying tunnels exhibit distinct and stage-dependent horizontal displacement responses (Figure 10). At an advanced distance of 9.6 m, the left tunnel already shows a measurable horizontal displacement with a maximum value of 0.236 mm, whereas the response of the right tunnel remains limited, reaching only 0.079 mm. This indicates that, in the early stage, lateral deformation is primarily concentrated in the near-field zone beneath the initial jacking section.

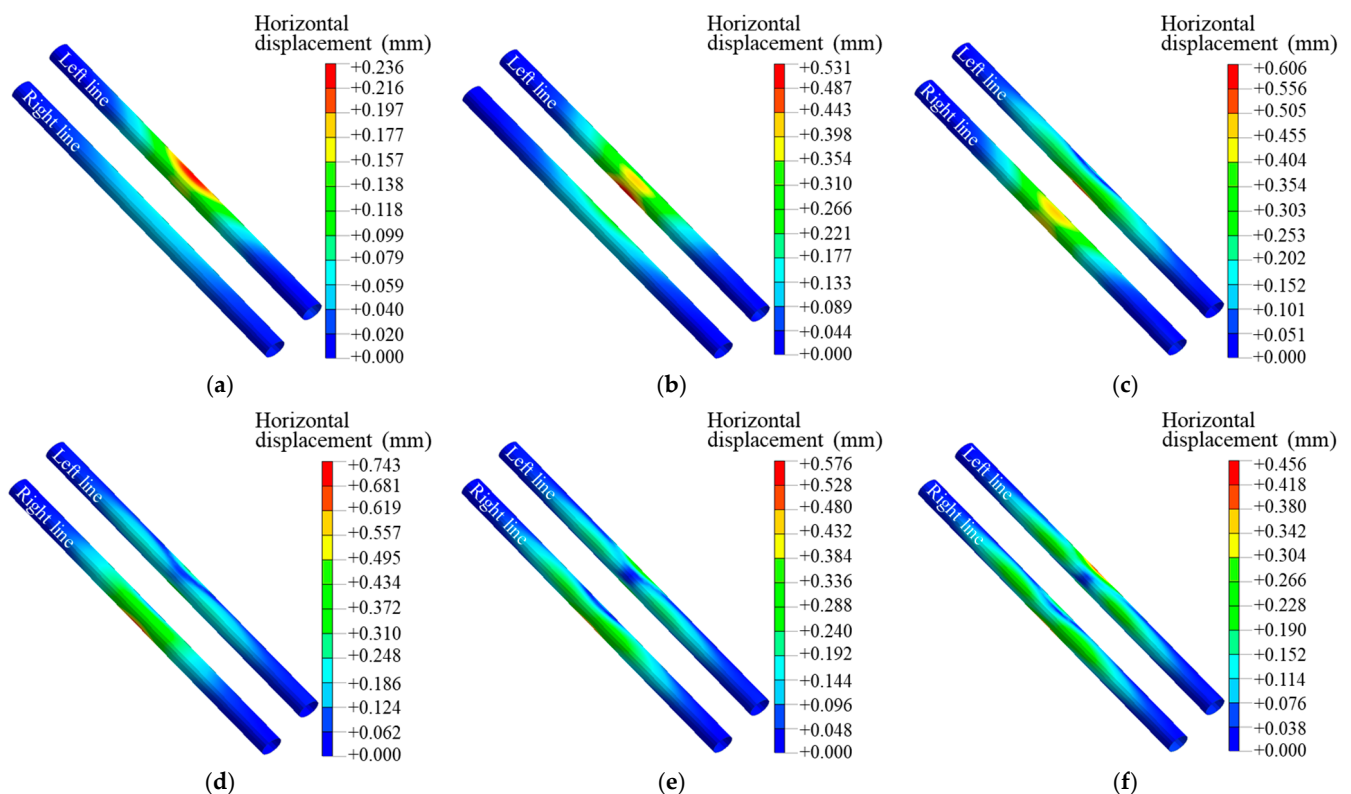


Figure 10. Contour maps of tunnel horizontal deformation at different pipe jacking advancement distances: (a) 9.6 m; (b) 19.2 m; (c) 30 m; (d) 40.8 m; (e) 51.6 m; (f) 63.6 m.

As the jacking progresses to 19.2 m, horizontal displacements of both tunnels increase significantly. The left tunnel reaches a maximum of 0.531 mm, while the right tunnel attains 0.266 mm, maintaining a clear difference in magnitude. At an advance of 30 m, a notable transition occurs: the right tunnel exhibits a maximum displacement of 0.606 mm, slightly surpassing the left tunnel's 0.556 mm. This reversal indicates that the dominant zone of lateral deformation shifts from the near-field to the far-field tunnel. Such a progression explains the formation of the characteristic S-shaped longitudinal displacement profile, which arises primarily from the sequential advancement of the pipe jacking. Initially, the left tunnel responds first, generating a downward trend in the displacement curve; subsequently, as the jacking-induced disturbance propagates and accumulates along the tunnel axis, the right tunnel responds more strongly, producing an upward trend and thereby creating the S-shaped pattern.

With further advancement to 40.8 m, the horizontal displacement of the left tunnel decreases significantly to 0.248 mm, whereas the right tunnel continues to experience increasing lateral deformation, reaching a maximum of 0.743 mm. This contrasting trend highlights a redistribution of lateral deformation, implying that the deformation mode evolves from a near-field-dominated response to a broader soil mass adjustment involving both tunnels. In the later stages (i.e., 51.6 m and 63.6 m), the maximum horizontal displacements of the left and right tunnels converge to comparable levels (i.e., 0.576 mm and 0.456 mm, respectively), and the difference between the two tunnels becomes negligible.

Overall, the evolution of horizontal displacement differs fundamentally from that of vertical settlement. While vertical deformation tends to accumulate monotonically with increasing the advanced distance, horizontal displacement exhibits a non-monotonic and asymmetric development pattern, characterized by a clear transition in dominance from the near-field tunnel to the far-field tunnel. This behavior reflects the progressive redistribution of lateral stresses and the cumulative nature of jacking-induced soil disturbance, which gradually promotes a more uniform deformation response in the later stages of construction.

5. Discussion

5.1. Comparison Between Numerical Simulation and Field Monitoring During Pipe Jacking

5.1.1. Comparison of Ground Surface Deformation Results During Pipe Jacking

To verify the reliability of the numerical simulation results, a comparative analysis of ground surface displacements induced by the pipe jacking was conducted. To monitor the deformation responses of both the ground surface and the tunnel structure in real time, a total of 27 monitoring points (D11–D38) were arranged within the construction influence zone. These points covered both sides of the construction axis as well as key upstream and downstream areas, providing a comprehensive reflection of the ground and tunnel deformation characteristics caused by construction. The monitoring work was carried out from 18 February to 15 March 2023, with daily data acquisition to obtain continuous and reliable deformation records. The specific layout of the monitoring points is shown in Figure 11. Comparison between numerical simulation and monitoring results of ground surface settlement are shown in Figure 12.

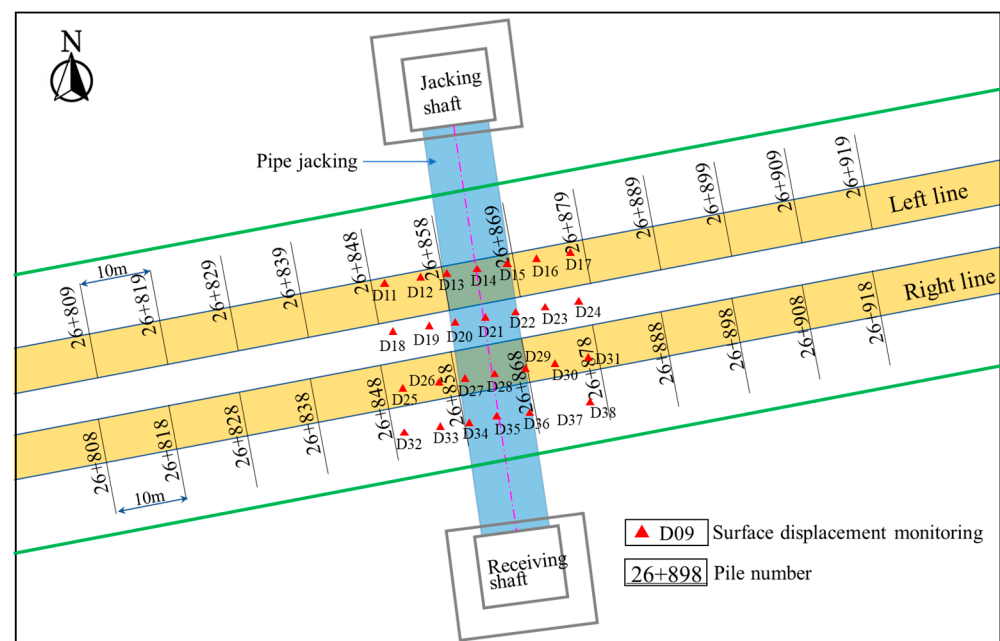


Figure 11. Layout of monitoring points in the subway section.

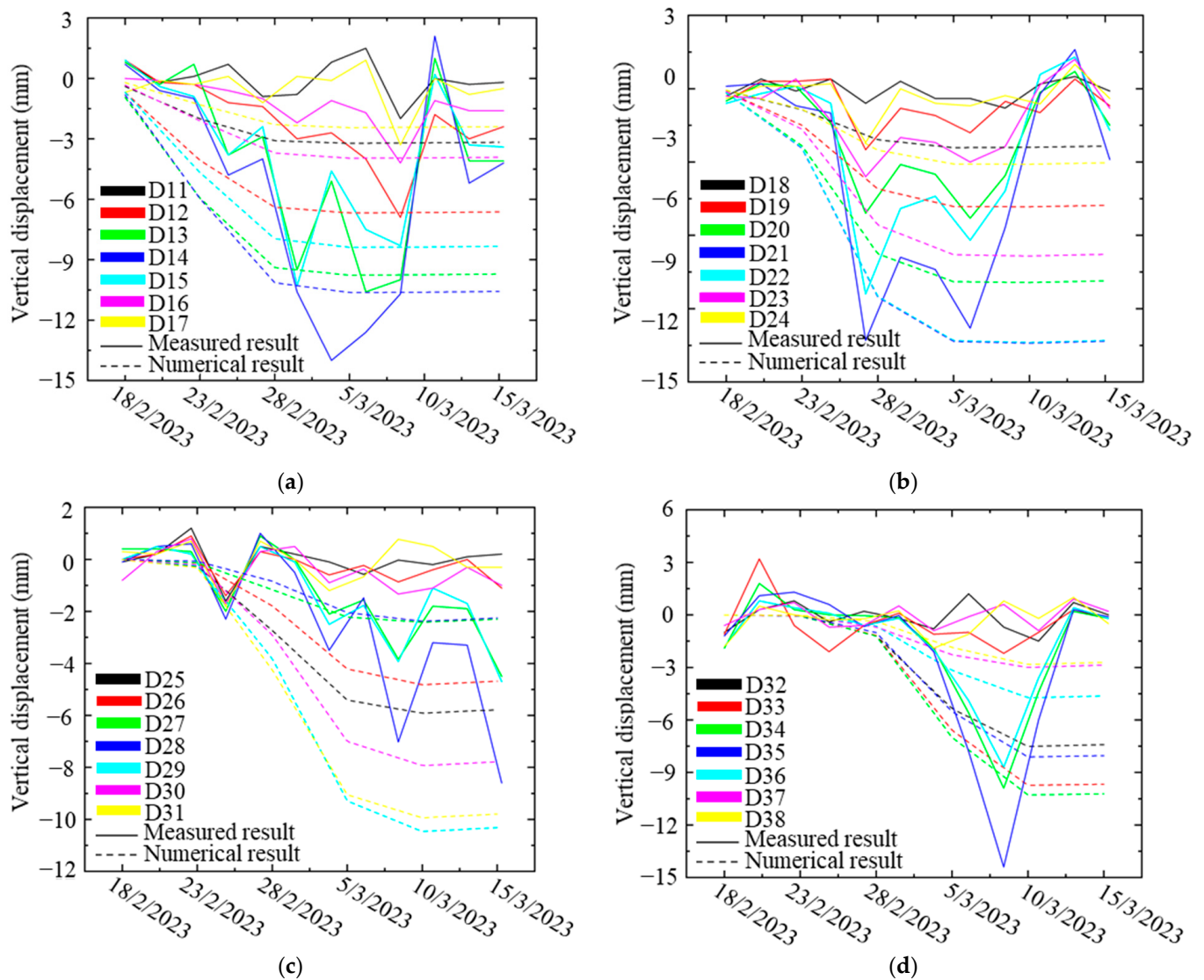


Figure 12. Comparison between numerical simulation and monitoring results of ground surface settlement. (a) D11–D17; (b) D18–D24; (c) D25–D31; (d) D32–D38.

During the pipe jacking process, the ground surface settlements at the 27 monitoring points arranged along the excavation axis exhibited a typical “trough-shaped” distribution (Figure 11). Overall, as the pipe jacking progressed from the launching end to 63.6 m, the settlement magnitudes gradually increased, with the central points experiencing significantly larger settlements than the sides, showing a progressive development pattern along the construction direction. Taking the D11–D17 row as an example, the central point D14 had a settlement of only 0.9 mm at the early stage of construction (excavation ~9.6 m, 18 February 2023), while at the final stage (63.6 m, 15 March 2023), it reached approximately 10.6 mm. The side points, such as D11 and D17, had final settlements of 3.2 mm and 2.4 mm, respectively, which were significantly lower than the central point. Similar patterns were observed at other central points, such as D21, D28, and D35: D21 reached a peak settlement of about 9.0 mm, D28 10.6 mm, and D35 a maximum of approximately 10.7 mm. These results indicate that settlements were prominently concentrated at the center, gradually decreasing toward the sides, consistent with the typical Gaussian-shaped settlement through distribution.

The comparison between numerical simulation and field measurements shows a high overall agreement. Taking D28 as an example, the simulated settlement at the final stage of

construction was 10.5 mm, while the measured value was 10.6 mm, with a difference of only 0.1 mm. For D35, the simulation gave 10.9 mm, and the measured value was 10.7 mm, resulting in a difference of 0.2 mm. The simulated settlements at the side points of each row generally differed from the measured values by no more than 0.5 mm. Overall, the peak settlements at the central points from the simulation closely matched the measured values, and the lateral distribution pattern was also well captured.

It should be noted that the measured data exhibit more irregular fluctuations in both spatial and temporal sequences. For instance, from late February to early March, certain points (e.g., D18, D26) showed temporarily larger settlements, with differences approaching 1.0 mm, whereas the numerical simulation results maintained a smooth transition. This discrepancy is primarily attributed to unavoidable external disturbances during field monitoring, such as traffic loads, groundwater level fluctuations, and instrument precision, which cause local variations in the measured curves while the overall trend remains consistent.

In summary, the finite element simulation not only accurately reproduces the temporal evolution of surface settlement but also reasonably captures its lateral distribution characteristics. Although the measured data exhibit some fluctuations, the overall differences are small, validating the reliability and feasibility of the simulation results. Considering the code requirement that surface settlement should not exceed 30 mm, the construction-induced disturbance of this project can be regarded as generally controllable. The numerical simulation thus provides a credible predictive tool and technical support for similar engineering projects.

5.1.2. Comparison of Tunnel Deformation Results

(1) Comparison of Vertical Displacement

Monitoring points for tunnel deformation were arranged along the alignment according to chainage. The left-line monitoring section covered K26 + 809 to K26 + 919, and the right-line section covered K26 + 808 to K26 + 918, with an interval of 10 m between adjacent points. Vertical displacement monitoring was conducted at the crown points of the left-line (A1) and right-line (A2) tunnels (as shown in Figure 2), and the specific layout of all monitoring points is presented in Figure 11.

As shown in Figure 13, the monitoring results indicate that the vertical settlement was relatively small during the early stage of construction, and slight uplift was even observed at some points. For instance, at K26 + 849, the initial settlement was about 0.48 mm, while K26 + 869 showed only 0.05 mm of settlement, and K26 + 879 exhibited a local uplift of 1.36 mm. As the pipe-jacking process progressed, the overall settlement gradually increased. By mid-March, the settlements at K26 + 849, K26 + 859, K26 + 869, and K26 + 879 reached 2.17 mm, 0.26 mm, 0.48 mm, and 1.21 mm, respectively. In contrast, the numerical simulation results presented a smoother growth pattern. The final settlement values were 2.07 mm, 1.50 mm, 2.26 mm, and 2.46 mm, respectively, which were slightly higher than the measured results. The differences were most noticeable at K26 + 859 and K26 + 869, where the simulated values exceeded the measured ones by approximately 1–2 mm.

The monitoring results of the right-line tunnel also exhibited noticeable fluctuations, as shown in Figure 13b. In the early stage of construction, a local uplift of −0.67 mm occurred at K26 + 878, while a settlement of 0.74 mm was observed at K26 + 848. With the advancement of the pipe jacking, the settlements in the middle section gradually increased, reaching maximum values of 1.89 mm and 1.63 mm at K26 + 858 and K26 + 868, respectively. The numerical simulation results, on the other hand, showed a smoothly increasing trend, with final settlements of 1.97 mm, 2.38 mm, 1.94 mm, and

1.23 mm at the corresponding locations. These results generally matched the measured peak positions, though the simulated amplitudes were slightly higher overall.

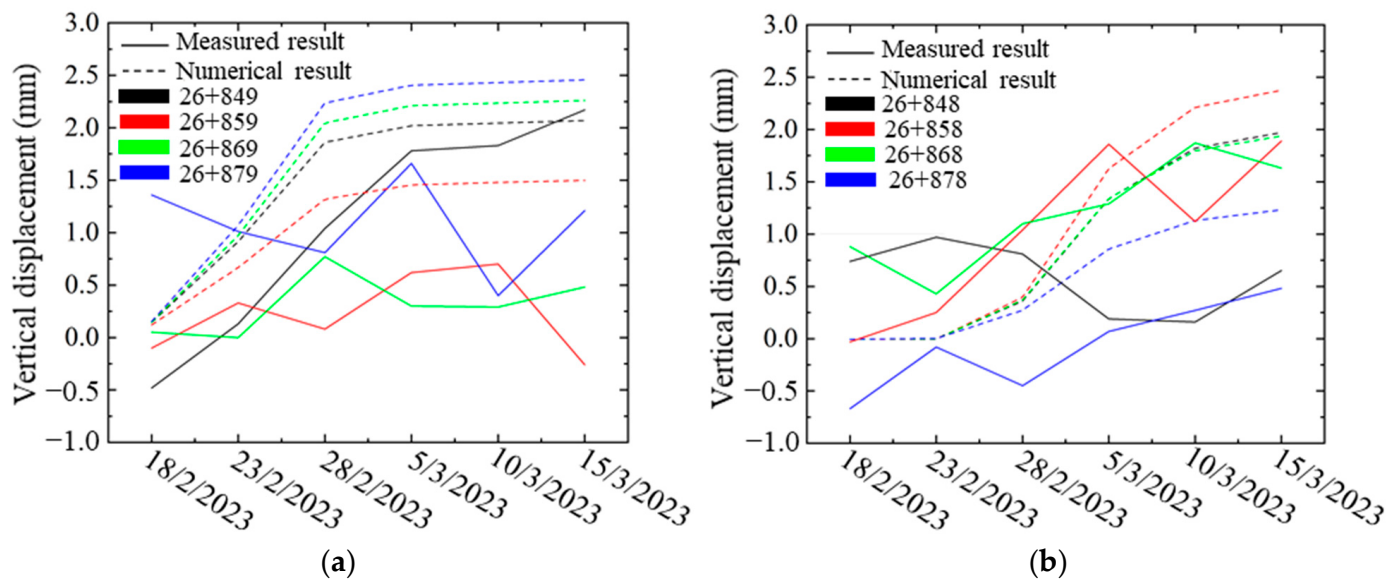


Figure 13. Comparison between numerical simulation and measured results of ground surface settlement of (a) left tunnel and (b) right tunnel.

A comparative analysis shows that the numerical and measured results exhibit good agreement in terms of the settlement development trend and spatial distribution, both revealing the characteristic pattern of greater settlement in the central section and smaller settlement at both ends. However, the following differences were observed:

① Amplitude difference: The simulated results were generally slightly larger than the measured values, with a maximum deviation of about 1–2 mm. This can be attributed to the conservative selection of soil parameters and the idealization of boundary and loading conditions in the numerical model.

② Process difference: The measured data displayed irregular fluctuations over time, including local uplifts and short-term rebounds, while the simulated results showed a smoother and more continuous curve. Such discrepancies are mainly due to the influence of random factors such as traffic loads, groundwater fluctuations, environmental disturbances, and monitoring errors on the field measurements, whereas these effects were excluded in the simulation, which reflects a more idealized soil–structure interaction process.

Overall, the finite element simulation accurately captured the settlement evolution and peak distribution, with deviations from the measured values remaining within an acceptable range—far below the 30 mm limit specified by design standards. This confirms that the proposed numerical modeling approach is reliable and feasible for predicting ground and tunnel settlement induced by pipe-jacking construction.

(2) Comparison of Horizontal Displacement of the Tunnel Pipeline

The monitoring points for the horizontal displacement of the tunnel were arranged along the alignment according to the pile numbers (i.e., the left line covering section K26 + 809 to K26 + 919, and the right line covering section K26 + 808 to K26 + 918). The monitoring points were installed at the left-line track bed (H1) and the right-line track bed (H2), as shown in Figure 2. As illustrated in Figure 14a, the measured horizontal displacement along the left line at K26 + 849, K26 + 859, K26 + 869, and K26 + 879 exhibited noticeable fluctuations. At the early stage, the displacement was relatively small—for instance, the maximum offset at K26 + 849 was approximately 0.32 mm, while at K26 + 869 it was only 0.03 mm. As construction progressed, certain locations showed

repeated variations, with significant offsets of -2.54 mm at K26 + 849 and -1.46 mm at K26 + 869 being recorded. By the end of construction, the horizontal displacement at K26 + 879 reached -1.73 mm. In contrast, the numerical simulation results demonstrated a smoother variation trend within the same section. The final displacements at K26 + 849, K26 + 859, K26 + 869, and K26 + 879 were -0.19 mm, -0.40 mm, -0.33 mm, and -0.19 mm, respectively, with overall magnitudes significantly smaller than the measured values.

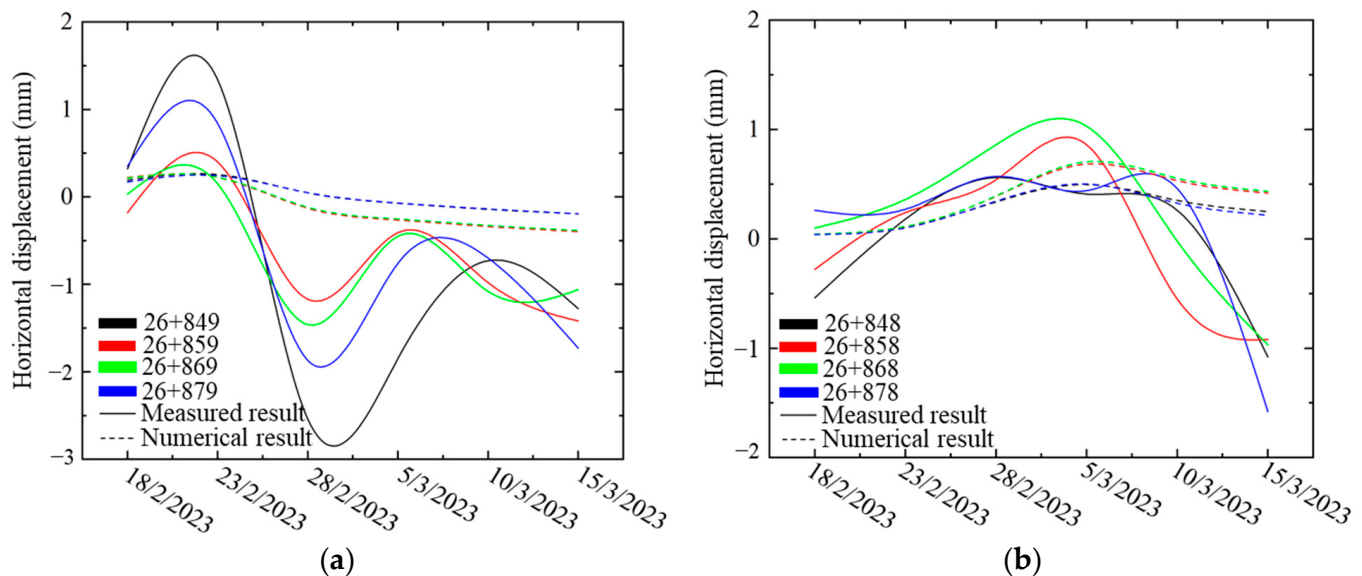


Figure 14. Comparison of measured and simulated horizontal displacements of metro segment lining: (a) left tunnel; (b) right tunnel.

The fluctuation characteristics of the measured results for the right line were even more pronounced (Figure 14b), showing localized stages of amplified displacement. For instance, at K26 + 868, a maximum offset of approximately 1.03 mm was recorded, while at K26 + 848, a -0.54 mm reverse displacement was observed, indicating that the field data were strongly influenced by construction disturbances and external environmental factors. In contrast, the numerical simulation results exhibited a continuously increasing and gradually stabilizing trend, with final displacements ranging between 0.22 mm and 0.43 mm. The displacement values at different points were close to each other, with smooth overall variation, and were clearly smaller than the measured peaks.

A comprehensive comparison shows that both the left and right lines revealed a cumulative trend of horizontal displacement during construction, with larger displacements occurring at the central pile numbers (e.g., K26 + 869 and K26 + 879). In general, the measured displacements were greater than the simulated ones, with local differences reaching 1–2 mm, and in some cases even showing alternating positive and negative fluctuations. By contrast, the numerical simulation results were relatively continuous and smooth. The main reason for this discrepancy lies in the fact that the measured data were affected by multiple environmental disturbances—such as vibrations from construction equipment, traffic loads, groundwater fluctuations, and instrument errors—causing local abnormal oscillations and even reverse displacements. On the other hand, the numerical simulation adopted uniform soil and structural parameters with idealized boundary conditions, eliminating random external influences; therefore, the results appeared smoother and more idealized.

Overall, the numerical simulation demonstrated good reliability in predicting the overall distribution characteristics and developmental trend of horizontal displacement. It accurately captured the tendency for larger displacements to occur near the central pile sections. It should be noted that the maximum tunnel displacement obtained from field

monitoring is slightly higher than the simulated value. This difference is mainly attributed to the idealization of soil constitutive behavior and the equivalent simulation of the pipe jacking process in the numerical model, which tends to smooth localized deformation responses. Although certain deviations existed in amplitude compared with the measured data, all discrepancies remained within the millimeter range—far below the 30 mm limit specified by the relevant code. This indicates that the finite element model established in this study possesses good feasibility and practical value for predicting horizontal displacement.

5.2. Analysis of Ground Reinforcement Effect

To scientifically evaluate the effectiveness of the ground reinforcement measures in mitigating the impact of pipe jacking construction, this study compared the responses of two working conditions—with reinforcement and without reinforcement—in terms of surface settlement, tunnel vertical displacement, and horizontal displacement. As shown in Figure 15, regarding surface settlement, the maximum settlement under the reinforced condition was approximately 11.14 mm (located about 47.5 m behind the excavation face), while under the unreinforced condition, the settlement continued to accumulate and eventually stabilized at around 45.5 mm. The difference between the two cases exceeded 34 mm, indicating that the reinforcement measures can significantly suppress the development of surface settlement. Furthermore, the settlement curve under the reinforced condition exhibited a gentle and converging trend, whereas under the unreinforced condition, the settlement trough was wider, and the settlement value continued to increase during the later stages of tunneling, which would markedly aggravate potential impacts on the ground surface and adjacent structures.

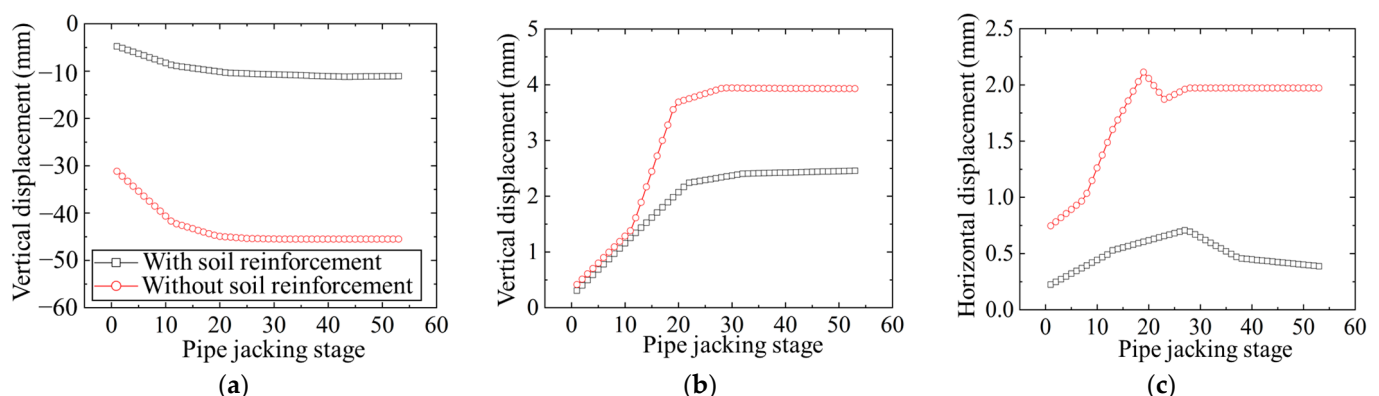


Figure 15. Surface and tunnel deformation during pipe jacking under reinforced and unreinforced soil conditions: (a) surface settlement, (b) tunnel crown settlement, and (c) horizontal deformation.

In terms of vertical displacement of the tunnel, the maximum settlement under the reinforced condition is only 2.46 mm, while under the unreinforced condition, it reaches about 3.93 mm, an increase of more than 50%. Taking a typical position as an example, at approximately 20 m from the excavation face, the settlement of the reinforced tunnel is about 2.07 mm, whereas that of the unreinforced tunnel is as high as 3.69 mm. It can be seen that the reinforcement measures not only reduce the peak settlement but also narrow the settlement influence range, effectively controlling the overall stress and deformation of the tunnel structure.

In terms of horizontal displacement, the maximum horizontal displacement under the reinforced condition is about 0.71 mm (at around 27 m), then gradually decreases to about 0.39 mm and tends to stabilize. Under the unreinforced condition, however, the peak horizontal displacement reaches 1.97 mm, nearly three times that of the reinforced case. Particularly in the stage before 20 m, the horizontal displacement of the unreinforced

tunnel increases rapidly, with the maximum value close to 2.11 mm, while after reinforcement, it is controlled within about 0.62 mm. These results indicate that the foundation reinforcement significantly weakens the horizontal thrust effect on the tunnel during pipe jacking construction.

In summary, the numerical results clearly demonstrate that under the unreinforced condition, the maximum surface settlement reaches about 45 mm, the tunnel vertical settlement is approximately 3.9 mm, and the tunnel horizontal displacement is around 1.97 mm, all of which are significantly higher than the corresponding values under the reinforced condition (11 mm, 2.46 mm, and 0.71 mm, respectively). The reinforcement measures show outstanding performance in reducing settlement amplitude, narrowing the deformation range, and mitigating horizontal thrust, effectively ensuring the safety of both the tunnel structure and the surface environment. These findings verify the feasibility and rationality of the adopted reinforcement scheme in controlling the adverse effects of pipe jacking construction and provide a reliable reference for design and risk management in similar engineering projects.

5.3. Parameters Sensitive Analysis

To assess the sensitivity of the numerical results to uncertainties in soil properties, a parameter sensitivity analysis was conducted by independently varying E , c , and φ by $\pm 10\%$ and $\pm 20\%$ relative to the reference values given in Section 3.2. The results consistently show that both horizontal and vertical displacements decrease with increasing E , c , and φ . Among the three parameters, E has the most significant influence. For $\pm 20\%$ variations in E , vertical displacement changes by approximately $-3.05\text{--}14.46\%$, while horizontal displacement varies by about $-3.01\text{--}13.74\%$ (Figure 16b). In contrast, the effects of c and φ are more moderate: $\pm 20\%$ changes in c lead to variations of less than about 7% in vertical displacement and 5% in horizontal displacement, whereas variations in φ result in displacement changes generally within about 6–7% (Figure 16a).

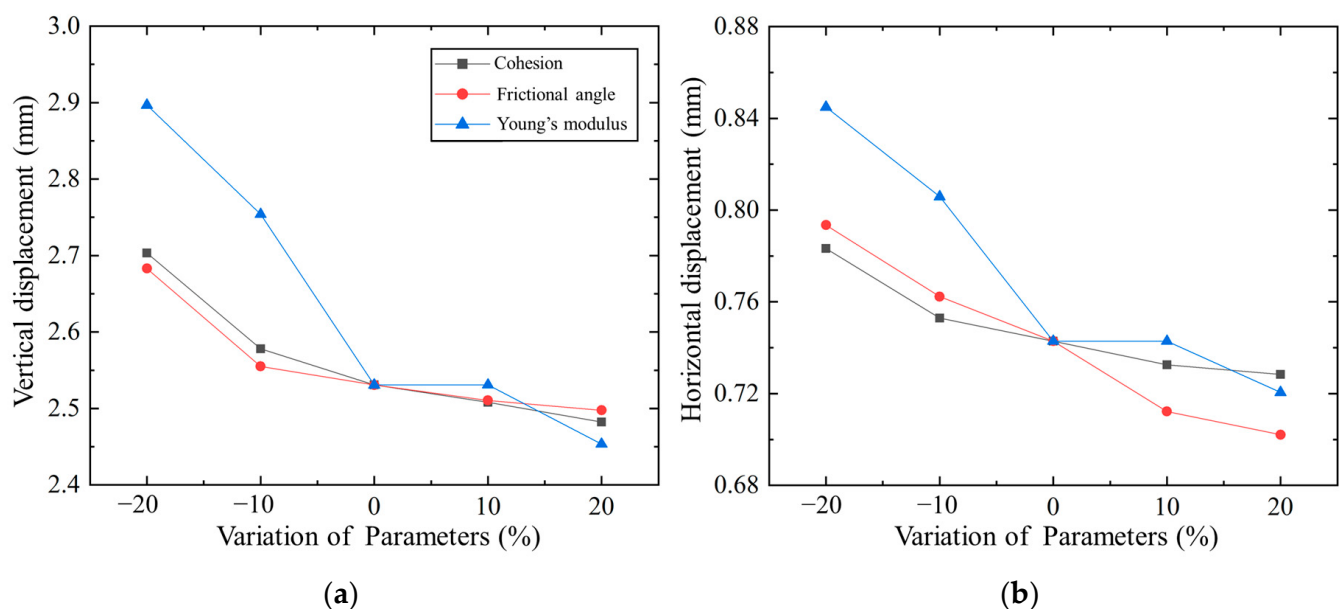


Figure 16. Sensitivity of pipe-jacking-induced displacements to key soil parameters. (a) Vertical displacement and (b) horizontal displacement as functions of relative variation (%) in cohesion c , frictional angle φ and Young's modulus E .

Despite these differences in displacement magnitude, the overall deformation mode, spatial distribution of displacement, and relative response trends remain essentially unchanged for all parameter cases, as confirmed by the displacement contour plots. The

limited sensitivity of the displacement response can be partly attributed to the grouting reinforcement applied within the construction influence zone, which enhances ground stiffness and strength and stabilizes the local mechanical response. Consequently, moderate variations in soil parameters result in only minor changes in the computed displacements.

Overall, the sensitivity analysis confirms that the main conclusions of this study are robust with respect to reasonable variations in key soil parameters, supporting the reliability of the numerical results.

5.4. Methodological Limitations

Despite the good agreement between numerical results and field monitoring data of this study, several methodological limitations should be acknowledged. First, the soil behavior was simulated using the modified Mohr–Coulomb constitutive model, which, although widely applied in engineering practice, provides a simplified representation of soil mechanical responses and may not fully capture complex behaviors such as strain softening or stress-path dependency. Second, the pipe jacking process was modeled using an equivalent advancement scheme with a fixed excavation length of 1.2 m per cycle, which idealizes the actual construction process and does not explicitly account for operational uncertainties. In addition, the numerical model assumes relatively homogeneous soil conditions and idealized boundary settings, which may differ from the inherent spatial variability of in situ ground conditions.

From a methodological perspective, physical model tests are often used to investigate soil–structure interaction mechanisms. However, for large-section rectangular pipe jacking, physical modeling also faces several inherent challenges. These include scaling effects that complicate the simultaneous satisfaction of geometric, kinematic, and stress similarity, difficulties in reproducing realistic in situ stress states and construction sequences, and limitations in representing complex reinforcement systems such as the portal-type ground improvement adopted in this study. As a result, numerical modeling combined with field monitoring offers a practical and flexible framework for engineering-scale analysis. Future studies may further enhance this framework by integrating advanced constitutive models, refined construction process simulations, or complementary experimental investigations.

6. Conclusions

This study investigated the deformation response of an existing metro tunnel induced by the construction of an overlying large-section rectangular pipe jacking tunnel using a combined approach of field monitoring and three-dimensional numerical simulation. The main conclusions can be summarized as follows:

- (1) Surface settlement characteristics. The surface settlement induced by pipe jacking exhibits a typical settlement trough pattern, with settlement magnitude increasing and gradually stabilizing as jacking progresses. Both numerical simulation and field monitoring show that the maximum settlement occurs above the jacking axis, reaching approximately 10–11 mm at the end of construction, with a lateral influence range of about 60 m. The simulated and measured values differ by less than 0.5 mm, indicating good accuracy of the numerical model in predicting surface deformation.
- (2) Tunnel deformation response. The metro tunnels exhibit clear spatiotemporal deformation characteristics during pipe jacking. The left tunnel responds earlier due to its proximity to the launching shaft, while the settlements of the two tunnels gradually converge as jacking advances, with a maximum vertical displacement of approximately 2.5 mm. Horizontal displacement evolves differently at various stages, but both tunnels eventually stabilize with values of 0.4–0.6 mm. The numerical simula-

tion effectively captures the evolution trend and magnitude of tunnel deformation, showing good agreement with the field monitoring results.

- (3) Effect of ground reinforcement. Ground reinforcement significantly reduces construction-induced deformation. Without reinforcement, the maximum surface settlement and tunnel vertical and horizontal displacements reach 45.5 mm, 3.93 mm, and 1.97 mm, respectively. After applying the DCM + MJS composite reinforcement scheme, these values decrease to 11.14 mm, 2.46 mm, and 0.71 mm, respectively, demonstrating that the reinforcement system effectively enhances ground stability and controls tunnel deformation.
- (4) Sensitivity of soil parameters. The parameter sensitivity analysis indicates that tunnel displacements decrease with increasing soil stiffness and strength (E , c , φ), with E having the strongest influence. Despite moderate variations in soil parameters, the overall deformation patterns and spatial distribution remain essentially unchanged, confirming the robustness of the numerical results and the reliability of the adopted modeling assumptions.

Overall, the “numerical simulation–monitoring verification–measure evaluation” integrated research framework established in this study successfully reproduces the mechanical behavior of the entire pipe jacking process. It clarifies the staged development of settlement, the mechanism of disturbance propagation, and the control effectiveness of reinforcement, providing a valuable analytical and practical reference for similar large-section pipe jacking projects crossing operational metro tunnels at close distances.

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