

Performances and environmental responses of a foundation pit with composite supporting mode in water-rich soft soil adjacent to the sea

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Abstract: Foundation pits in coastal, water-rich soft soils face risks and challenges such as excessive deformation, severe seepage, and tight construction schedules. Considering the complex geological and hydrological conditions and the need to balance safety and economy, A composite support scheme that combines pile anchors with graded excavation was proposed. Numerical simulations were conducted to predict deformation of the retaining structure and associated environmental responses, thereby preliminarily verifying the feasibility of the proposed scheme. Finally, a real-time monitoring system was installed to record horizontal and vertical displacements of retaining piles, axial forces in anchor cables, ground settlement, and groundwater levels outside the pit. The temporal evolution and spatial distribution of these responses were analyzed and the effectiveness of the adopted design and construction schemes was validated. The results indicate that most deformation and environmental responses occurred during the dewatering-excavation stage. Anchor cables remained in tension, and tensile force correlated strongly with the deflection of retaining piles. Upper-level anchors developed greater tensile forces than lower-level anchors, reflecting the cantilever deformation mode of the retaining piles. After adopting the composite pile-anchor and grading excavation approach, the maximum retaining wall deflection, maximum ground settlement, and maximum groundwater level variation outside the pit were 28.0, 26.36, and 699 mm, respectively, all of which are within allowable safety thresholds.

Key words: foundation pit; soft soil; pile-anchor; grading excavation; numerical simulation; tested performance

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With the rapid development of underground space, geological conditions encountered during foundation pit excavation for building basements, metro stations, and other underground facilities have become increasingly complex^[1]. Foundation pits in deep, thick, water-rich soft soils are increasingly common. Because of the poor engineering properties of soft soil, excavation in such deposits poses a high construction risk, and failures and accidents sometimes occur^[2-4]. Therefore, reducing deformation and ensuring safety are major concerns for these foundation pit projects. Recent studies have addressed design methods^[5-6], mechanical and deformation behavior^[7-10], seepage behavior^[11-13], and construction-induced environmental responses^[14-16] for foundation pits through theoretical analysis, field monitoring, and numerical simulation etc., and various mitigation measures such as zoning excavation^[17-19], ground improvement^[20-22], and cut-off walls^[23-24] have been proposed. These studies improved the understanding of the performance and failure mechanisms of foundation pits in soft soils. However, documented cases of foundation pits in reclamation-formed soft soils adjacent to the sea are extremely limited. Most existing support structures rely on retaining piles (or walls) combined with internal horizontal struts, while alternative design schemes and construction experiences remain relatively scarce.

In this work, we presented a case study of a hotel basement foundation pit adjacent to the East China Sea, underlain by thick, water-rich, soft soil resulting from land reclamation. We developed and implemented an integrated mitigation approach. First, considering safety and economy, we proposed a composite supporting mode of pile anchors combined with grading excavation, challenging the traditional concept that “anchor cable is unsuitable for foundation pit supporting structure in soft soil.” Second, we used numerical simulation to predict performance and environmental response and to preliminarily verify the proposed design feasibility. Finally, we installed a monitoring system to record the horizontal and vertical displacements of the retaining pile, axial force of anchor cable, ground settlement, and groundwater levels outside the pit. We analyzed their temporal evolution

and spatial distribution and used the results to evaluate the effectiveness of the design and construction schemes.

1 Site Condition

1.1 Project introduction

Fig. 1 shows the location of the foundation pit. The foundation pit is in Chenghai District, Shantou City, and it is a single-story underground basement of a hotel. The plane area of the foundation pit is approximately 15 000 m²; the perimeter is about 570 m; and the maximum excavation depth is 7.6 m. Xuehai Road borders the site to the north, Donghai Avenue to the east, and a planned exhibition center bounds the site to the south and west. These surrounding elements place high demands on foundation pit safety.



Fig. 1 Location of the foundation pit

1.2 Geological conditions

The site is adjacent to the East China Sea and lies in

the alluvial plain of the lower Hanjiang River. The original landform was a coastal tidal flat; the present ground consists primarily of artificial reclamation fill. Geological investigation divides the stratigraphy into nine layers (top to bottom): ① miscellaneous fill; ② dredger fill; ③ muck; ④ medium sand; ⑤ sandy clay; ⑥ fully weathered granite; ⑦ strongly weathered granite (soil shape); ⑧ strongly weathered granite (blocky shape); ⑨-1 moderately weathered granite (fragmental shape); ⑨-2 moderately weathered granite (columnar shape).

Table 1 presents statistical results for the top and bottom depths and the thickness of each soil layer. Excavation of the foundation pit primarily involves Layers ①, ②, and ③, and Layer ④ (medium sand) lies beneath the foundation pit base.

Table 2 reports the physical and mechanical parameters of the soil layers. The water content (w), unit weight (γ), saturation degree (S_r), void ratio (e), plasticity index (I_p), liquidity index (I_L), and permeability coefficients (K_v and K_h) were obtained through physical properties test of soil. The compression modulus (E_s) was obtained from consolidation tests. The friction angle (φ) and cohesion (c) were obtained from consolidated-drained triaxial shear tests. The standard penetration test blow count (N) was obtained from in-situ SPT testing. Because Layers ① and ④ have loose soil structure and no complete core samples were recovered, some parameters are unavailable for those layers. Layers ①, ②, and ③ exhibit high water content, large void ratios, and low strength. Their engineering properties are poor.

Table 1 Statistical results of top depth, bottom depth and thickness of each soil layer

m

Soil layer No.	Layer top depth			Layer bottom depth			Thickness		
	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
①	0.00	0.00	0.00	0.30	2.90	1.58	0.30	2.90	1.58
②	0.00	2.90	1.29	2.80	7.40	4.54	1.00	5.20	3.25
③	1.10	7.40	4.23	4.30	12.20	8.28	1.40	6.20	4.05
④	4.30	12.20	8.08	6.80	14.00	10.43	0.60	5.20	2.35
⑤	7.20	13.40	10.22	8.50	17.20	13.13	0.70	6.50	2.91
⑥	8.50	17.20	12.36	11.20	23.50	15.59	0.70	8.70	3.22
⑦	6.80	23.50	15.04	10.00	27.20	17.08	0.80	4.70	2.05
⑧	7.20	27.20	14.95	9.10	54.60	18.61	0.30	27.40	3.65
⑨-1	10.60	10.60	10.60	12.20	12.20	12.20	1.60	1.60	1.60
⑨-2	9.80	30.10	17.37	15.30	36.70	22.69	1.40	12.40	5.32

Table 2 Physical and mechanical parameters of soil layers in the site

Soil layer No.	$w/\%$	$\gamma/(\text{kN} \cdot \text{m}^{-3})$	$S_r/\%$	e	I_p	I_L	c/kPa	$\varphi/^\circ$	E_s/MPa	N
②	62.10	15.9	97.16	1.70	20.44	1.80	6.9	1.5	1.95	1.1
③	63.72	15.8	96.93	1.74	20.06	1.91	7.9	1.7	1.90	1.2
④		20.0					0	25.0		11.5
⑤	21.30	19.5	85.90	0.66	14.40	0.25	17.7	20.6	4.67	17.2
⑥	20.60	19.6	86.60	0.63	13.90	0.18	21.8	26.2	4.08	35.2

1.3 Hydrogeological conditions

Groundwater at the site comprises phreatic (unconfined) and confined aquifers. Phreatic water is mainly present in shallow Layers ① and ②, which are thick, highly permeable, and richly saturated. Geological investigation indicates that the groundwater level is 0.65 to 1.10 m below the ground surface, with an annual variation range of 0.5 to 1.0 m. Phreatic conditions can promote water accumulation at the pit bottom and destabilize the pit wall.

Confined water is present mainly in Layer ④ and in fractures within the weathered granite layers below it. Layer ④ contains substantial water storage, and the weathered granite layers have limited water storage. Investigation indicates that the potentiometric head in Layer ④ corresponds to a pressure-head level of 3.10 to 3.80 m below the ground surface. Confined conditions raise the risk of pile-hole collapse, leakage at pile joints, and bottom inrush.

1.4 Challenges of foundation pit

The site's coastal location and reclaimed, thick soft soil foundation with high water content and a shallow groundwater table present significant construction challenges. A confined aquifer with an elevated head lies beneath the excavation base. During construction, risks include pit wall leakage, water inrush, borehole collapse, and large deformations of the retaining structure. Furthermore, the foundation pit is a single-floor underground basement for a 5-star hotel scheduled to open in October 2024. The construction time left for the underground structure is less than 6 months. Therefore, there is an urgent need for a reasonable design scheme of a retaining structure that allows rapid construction and ensures safety.

2 Design and Construction of Foundation Pit

2.1 Design of retaining structure

Given the complex geological and hydrological conditions and the tight construction schedule for the foundation pit, we adopted a composite supporting system. As shown in Figs. 2 and 3, single-row bored piles combined with double-row prestressed anchor cables are used at the east and north sides (adjacent to municipal roads). Grading with slope protection (steel mesh and shotcrete) and cement-soil mixing piles are used on the south and west sides (adjacent to open spaces). Cement-soil mixing piles are constructed around the foundation pit as a waterproof curtain. In the figures, H_0 is the excavation depth of the foundation pit; H_1 is the depth of the bored pile; α_1 and α_2 are the inclination angles of the 1st and 2nd levels of the anchor cable, respectively; L_1 and L_2 are the full length of the 1st and 2nd levels of the anchor cable, respectively; F_1 and F_2 are the jacking forces of the 1st and

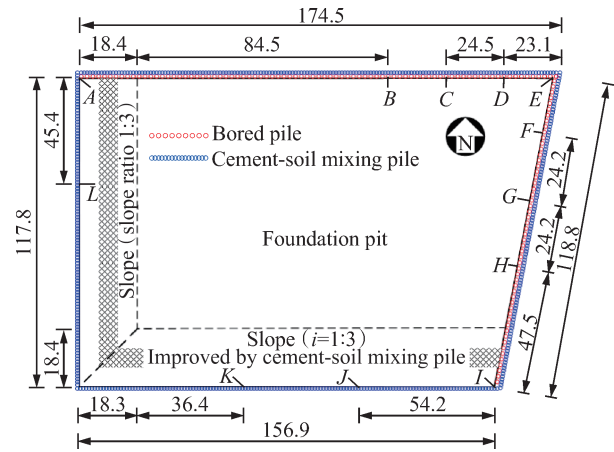


Fig. 2 Plane layout of retaining structure of the foundation pit (unit: m)

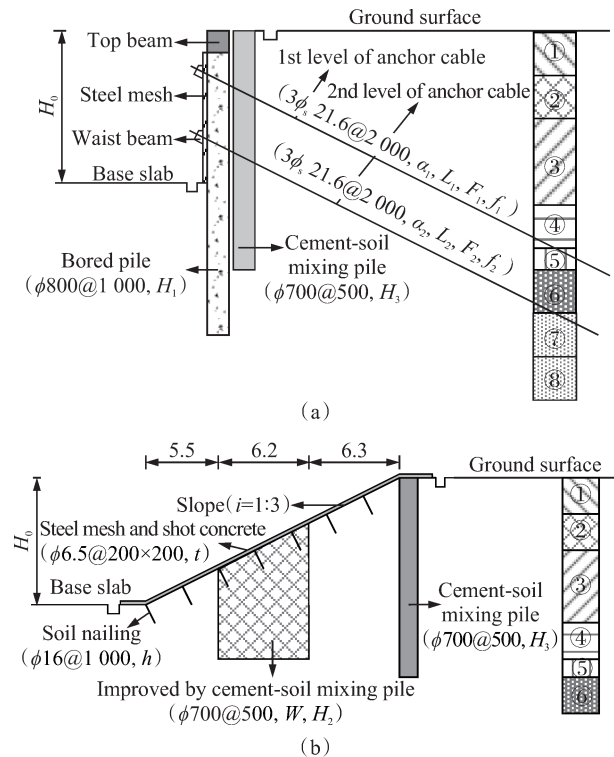


Fig. 3 Cross section of retaining structure of foundation pit (unit: mm). (a) Pile-anchor structure on the north and east sides; (b) Grading excavation on the south and west sides

2nd levels of the anchor cable, respectively; f_1 and f_2 are the locking forces of the 1st and 2nd levels of the anchor cable, respectively; H_3 is the depth of the cement-soil mixing pile; i is the slope ratio; h is the insertion depth of soil nailing; t is the thickness of shot concrete on the slope; W and H_2 are the width and depth of improved soil by the cement-soil mixing pile, respectively.

The retaining structure was divided into sections (AB, BC, CD, DE, EF, FG, GH, HI, IJ, JK, KL, LA), and section-specific design parameters are given in Tables 3 and 4. The anchor cable lengths are designed so the anchor heads extend to weathered rock layers where feasible.

Table 3 Design parameters of retaining structures on the north and east sides

Section	H_0/m	H_1/m	$\alpha_1/(\circ)$	L_1/m	F_1/kN	f_1/kN	$\alpha_2/(\circ)$	L_2/m	F_2/kN	f_2/kN	H_3/m
AB	7.0	13	30	34.5	520	390	30	18.0	250	190	11.0
BC	7.0	14	30	28.5	540	410	30	21.0	250	190	12.0
CD	7.0	13	30	23.0	520	390	30	15.0	250	190	11.5
DE	7.0	15	30	32.5	520	390	30	23.5	270	210	12.5
EF	7.6	16	30	34.0	380	280	30	22.0	230	165	12.5
FG	7.6	15	30	41.5	375	270	30	29.5	245	180	13.0
GH	7.6	15	30	35.5	390	285	30	22.5	220	160	11.5
HI	7.6	16	30	29.5	370	275	30	18.0	200	150	13.0

Table 4 Design parameters of retaining structures on the south and west sides

Section	H_0/m	i	h/m	t/m	W/m	H_2/m	H_3/m
IJ	6.0	1:3	1.0	0.1	6.2	10.0	14.0
JK	6.0	1:3	1.0	0.1	6.2	10.0	14.0
KL	6.0	1:3	1.0	0.1	6.2	10.0	14.0
LA	6.0	1:3	1.0	0.1	6.2	10.0	14.0

2.2 Dewatering of groundwater

To ensure safe excavation, well point dewatering was used in the pit to lower the groundwater level below the base slab. Ten ZBL-9-60 vacuum well point systems were installed; each system contains 30 well-point pipes (300 well points total).

The main header pipe is 600 m long with a 60 mm diameter. Each well point pipe lateral is 4 to 6 m long with an outer diameter of 48 mm; the filter length in each lateral is 80 mm. Well point pipe spacing is 1.5 to 1.8 m, and the lateral pipe bottoms are embedded 0.5 m below the base slab. The well point pipe and the main pipe are connected by rubber hoses.

3 Numerical Simulation of Foundation Pit

3.1 Numerical model establishment

A three-dimensional numerical model of the foundation pit was developed based on the site's geological and hydrological conditions and the design and construction parameters.

To minimize boundary effects, the model's horizontal extent was set to 5 times the excavation depth from the foundation pit edge, and its vertical extent was set to 4 times the excavation depth below the base slab. The model dimensions are 250 m $\times$ 188 m $\times$ 35 m, as shown in Fig. 4 (a). In the model, soil was represented by solid elements. Bored piles were modeled as plates to represent equivalent stiffness principle of a retaining wall, as shown in Fig. 4 (b). The cement-soil mixing-pile waterproof curtain was modeled with solid elements, as shown in Fig. 4 (c). Waist beams and prestressed anchor cables were modeled using beam elements, as shown in Fig. 4 (d).

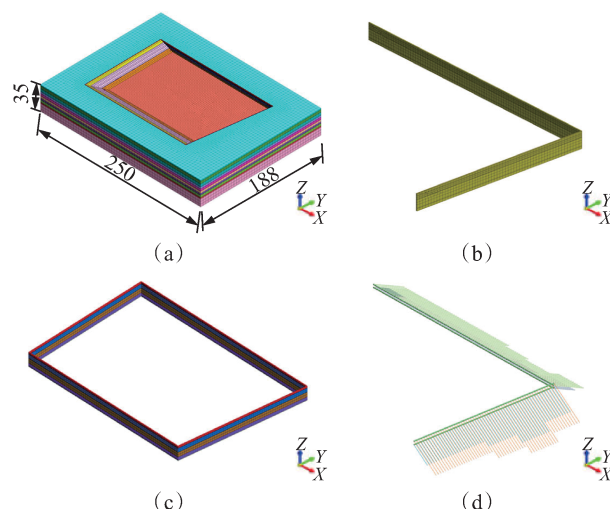


Fig. 4 FEM numerical model for the foundation pit (unit: m). (a) 3D model; (b) Retaining wall model; (c) Cement-soil mixing pile model; (d) Prestressed anchor cable model

Displacement boundary conditions are as follows: Lateral (left and right) boundaries were constrained in the horizontal displacements. The bottom boundary was constrained in both horizontal and vertical displacements. The top boundary was kept free. Load boundary conditions considered only self-weight (gravity). No additional external loads are considered. Hydraulic boundary conditions used an initial groundwater table at 0.5 m below ground surface. Because the project construction period is less than 6 months and the annual variation in the initial groundwater level is only 0.5 to 1.0 m, the groundwater level was assumed constant for the simulation.

3.2 Constitutive model and parameters

3.2.1 Constitutive model and parameters of soil layers

Ref. [25] indicates that the hardened soil (HS) model

is an advanced constitutive model suitable for representing the mechanical and deformation behavior of a wide range of soils, including soft and dense soils. Consequently, the HS model was adopted in this study.

The HS model requires 11 parameters, including cohesion (c), effective friction angle (φ), dilatancy angle (ψ), secant stiffness in a standard drained triaxial test (E_{50}^{ref}), tangent stiffness for primary oedometer loading ($E_{\text{od}}^{\text{ref}}$), unloading-reloading triaxial stiffness ($E_{\text{ur}}^{\text{ref}}$), stress-level dependency exponent (m), unloading-reloading Poisson's ratio (ν_{ur}), reference stress for stiffness (P^{ref}), failure ratio (R_f), and the at-rest stress ratio for normally consolidated conditions (K_0). Following

Plaxis recommendations^[26], m may be 0.5 to 1.0 for clay soils, and 0.5 for sandy soils; ν_{ur} may be taken as 0.2; $P^{\text{ref}} = 100$ kPa; ψ may be set as $\psi = \varphi - 30^\circ$ for sandy soils, and $\psi = 0^\circ$ for clay soils; K_0 can be estimated by $K_0 = 1 - \sin \varphi$ for normally consolidated soils. The values of c , φ , E_{50}^{ref} and R_f can be determined through triaxial consolidated-drained shear test; $E_{\text{ur}}^{\text{ref}}$ can be determined through triaxial consolidated-drained unloading-reloading tests; and $E_{\text{od}}^{\text{ref}}$ can be determined through consolidation tests. Based on laboratory results, recommended values, and empirical relations^[27], HS parameters for each soil layer were determined and are listed in Table 5.

Table 5 Parameters of the HS model for each soil layer

Soil layer No.	$\gamma/(\text{kN} \cdot \text{m}^{-3})$	c'/kPa	$\varphi'/(^{\circ})$	$\psi/(^{\circ})$	$E_{50}^{\text{ref}}/\text{MPa}$	$E_{\text{od}}^{\text{ref}}/\text{MPa}$	$E_{\text{ur}}^{\text{ref}}/\text{MPa}$	m	ν_{ur}	$P^{\text{ref}}/\text{kPa}$	R_f	K_0
①	15.5	6.5	1.5	0	2.5	2.0	10	0.8	0.2	100	0.9	0.97
②	15.9	6.9	1.5	0	2.5	2.2	11	0.8	0.2	100	0.9	0.97
③	15.8	7.9	1.7	0	3.0	2.5	12.5	0.8	0.2	100	0.9	0.97
④	20.0	0	25.0	0	6.0	4.0	20	0.8	0.2	100	0.9	0.58
⑤	19.5	17.7	20.6	0	8.0	6.4	32	0.8	0.2	100	0.9	0.65
⑥	19.6	21.8	26.2	0	15.0	13.4	67	0.7	0.2	100	0.9	0.56
⑦	19.2	25.0	26.5	0	22.4	18.6	93	0.7	0.2	100	0.9	0.55
⑧	22.0	30.0	27.0	0	28.6	25.0	125	0.7	0.2	100	0.9	0.55
⑨-1	22.5	40.0	28.0	0	40.0	36.0	180	0.6	0.2	100	0.9	0.53
⑨-2	23.0	55.0	28.5	0	46.5	42.0	210	0.6	0.2	100	0.9	0.52

3.2.2 Constitutive model and parameters of re-aining wall, waterproof curtain, waist beam, and prestressed anchor cable

Retaining walls and waist beams were assigned linear elastic behavior corresponding to C30 concrete. For C30 concrete, unit weight $\gamma = 24.5$ kN/m³, elastic modulus $E = 30$ GPa, and Poisson's ratio $\nu = 0.2$.

The waterproof curtain (cement-improved soil) was modeled using the HS constitutive model with the following parameters: $\gamma = 20.0$ kN/m³, $c = 28$ kPa, $\varphi = 26^\circ$, $\psi = 0^\circ$, $E_{50}^{\text{ref}} = 25$ MPa, $E_{\text{od}}^{\text{ref}} = 20$ MPa, $E_{\text{ur}}^{\text{ref}} = 100$ MPa, $m = 0.6$, $\nu_{\text{ur}} = 0.2$, $P^{\text{ref}} = 100$ kPa, $R_f = 0.9$, $K_0 = 0.56$.

Prestressed anchor cables were modeled as linear elastic steel elements. For steel, $\gamma = 78.5$ kN/m³, $E = 200$ GPa, $\nu = 0.3$.

3.2.3 Constitutive model and parameters of interface

A contact-interface model was applied at wall-soil contacts. The interface is characterized by a normal stiffness modulus K_n and a shear stiffness modulus K_t , computed as

$$K_n = E_{\text{od},i}/t_v \quad (1)$$

$$K_t = G_i/t_v \quad (2)$$

where

$$\begin{aligned} E_{\text{od},i} &= 2G_i(1 - \nu_i)/(1 - 2\nu_i) \\ G_i &= RG_{\text{soil}} \\ G_{\text{soil}} &= E/2(1 + \nu_{\text{soil}}) \end{aligned}$$

where ν_{soil} is the Poisson's ratio of soil (set to be 0.35); ν_i is the Poisson's ratio of the interface (set to be 0.45); t_v is the virtual thickness coefficient (typically 0.01-0.10), $t_v = 0.1$ was used for the concrete wall-soil interface; R is the strength reduction coefficient (typically 0.7-1.0), $R = 0.7$ was used for the concrete wall-soil interface.

3.3 Simulation steps and typical stages

Refs. [28-31] have shown that dewatering can cause significant movement of retaining walls. This study modeled both dewatering and excavation. The analysis procedure comprised the following steps:

- ① Activate displacement, load, and hydraulic boundary conditions to obtain the initial seepage and stress fields.
- ② Construct the retaining wall and waterproof curtain.
- ③ Sequentially lower the groundwater in the pit to 1.0 m below each expected excavation elevation (by adjusting pressure water head), remove the corresponding soil layers, and install prestressed anchor cables.
- ④ Repeat Step ③ until the excavation reached the base slab.

In Steps ① and ②, any deformations generated by gravity construction were allowed to stabilize and were omitted in subsequent excavation dewatering-induced deformations. For the parametric analysis presented here,

we selected the stage at which excavation has reached the base slab for detailed evaluation.

3.4 Analysis of simulation results

3.4.1 Horizontal displacements of retaining wall and slope

Fig. 5 presents horizontal displacement contours for the retaining walls and slopes. T_x and T_y denote displacements in the X and Y directions, respectively. The maximum horizontal displacements for the east retaining wall and the west slope are 21.97 and 27.39 mm, respectively (Fig. 5 (a)). For the north retaining wall and the south slope, the maxima are 28.91 and 27.33 mm, respectively (Fig. 5 (b)). The north retaining wall displacement slightly exceeds that of the east wall, whereas the west and the south slopes exhibit comparable horizontal displacements. Maximum displacements are located near the midspan of each side.

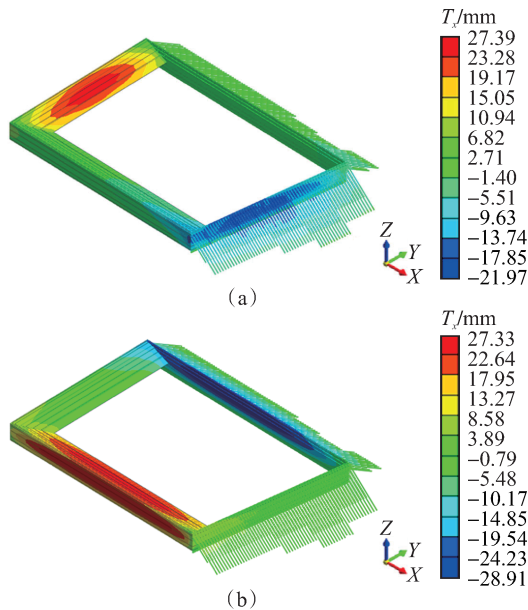


Fig. 5 Horizontal displacement contour map of retaining structure and slope. (a) West and east sides; (b) North and south sides

3.4.2 Axial force of prestressed anchor cable

Fig. 6 shows the axial force contour for prestressed anchor cables. The anchor cables were in tension, and the axial-force distribution along their length approximated a right trapezoid, decreasing gradually from the anchor head to the anchor end. Because the anchor cable bears the load and deformation together with the retaining pile, the axial force of the anchor cable has a good corresponding relationship with the displacement of the retaining wall. Given the larger horizontal displacement on the north side than on the east side, the maximum axial force of the anchor cable was located on the north side of the foundation pit. Vertically, the upper (1st) anchor-level axial force exceeds that of the lower (2nd) level; the

maximum axial forces for the 1st and 2nd anchor levels are 528.68 and 475.44 kN, respectively.

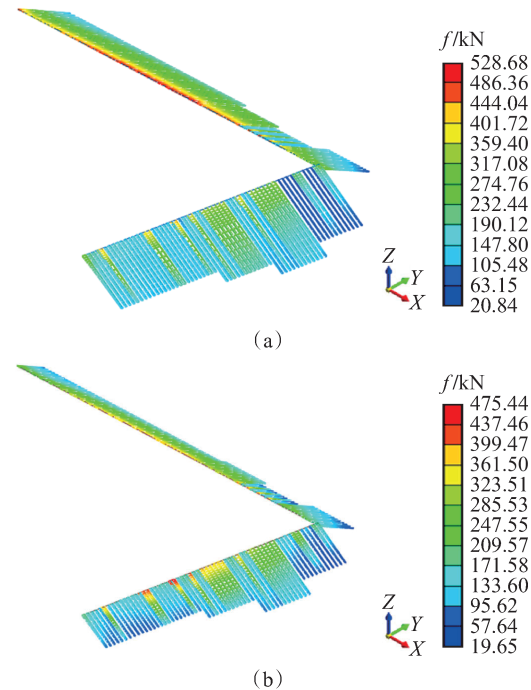


Fig. 6 Axial force contour map of prestressed anchor cable. (a) 1st level of anchor cable; (b) 2nd level of anchor cable

3.4.3 Ground settlement

Fig. 7 shows ground settlement contours outside the pit. Settlement on the north and east sides of the pit (retaining wall support) exceeds settlement on the west and south sides (grading excavation). Consistent with the displacements and anchor axial force distributions, the maximum ground settlement is 25.43 mm on the north side. This finding indicates that construction attention should focus on ground settlement along the north and east sides of the foundation pit.

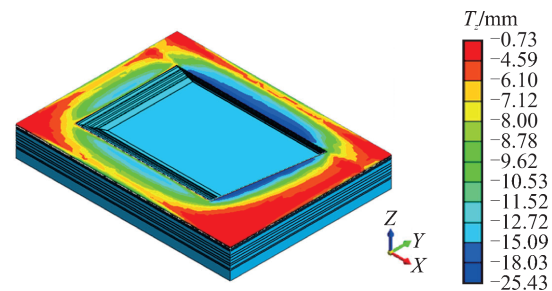


Fig. 7 Contour map of ground settlement outside the pit

3.4.4 Groundwater level

Fig. 8 presents the groundwater pressure water head contours. Within the pit, the groundwater level was lowered from approximately 0.5 m below the ground surface to the pit bottom, resulting in a pressure head of 0 m at the pit base. Outside the pit, the groundwater level remained at the original level (pressure water head -0.5 m relative to ground surface datum). These results indicate

that the cement-soil mixing-pile waterproof curtain effectively interrupted hydraulic connectivity between the pit interior and the surrounding ground, so pit dewatering did not appreciably alter groundwater levels outside the curtain.

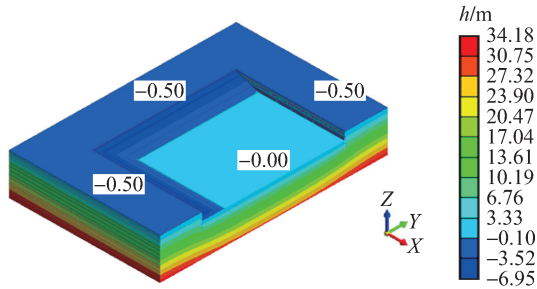


Fig. 8 Pressure water head contour map of groundwater

3.5 Evaluation of simulation results

Following the Chinese technical standard for monitoring of building excavation engineering (GB 50497—2019) and considering local accumulated experience, control values for foundation-pit performance and environmental responses were adopted, as listed in Table 6.

The above analysis shows that after the soil is excavated to the base slab, the maximum horizontal displacement of retaining walls and slopes was 28.91 mm; the maximum ground settlement was 25.43 mm; and the groundwater level outside the pit remained unchanged. These values do not exceed the control limits in Table 6. Therefore, the adopted design and construction schemes are preliminarily judged feasible.

Table 6 Control values of foundation pit performance and environmental responses

No.	Monitoring items	Accumulated value /mm	Rate value/(mm·d ⁻¹)
1	Deflection of the retaining wall	60	5
2	Horizontal displacement of the top of the retaining wall	40	4
3	Horizontal displacement of the top of the slope	50	5
4	Vertical displacement of the top of the retaining wall	30	3
5	Vertical displacement of the top of the slope	40	4
6	Ground settlement	45	4
7	Groundwater level	1 000	500

4 Observed Performance of Foundation Pit

4.1 Monitoring items

To verify the rationality of the design and construction schemes and to guide construction through field feedback, the foundation pit performance and associated environmental responses were monitored. According to the Chinese technical standard (GB 50497—2019) for monitoring building excavation engineering, the project is classified as safety level II. The adopted monitoring program included deflection of retaining walls, horizontal and vertical displacements at the tops of retaining walls and slopes, axial force of prestressed anchor cables, ground settlement, and groundwater levels outside the pit.

Fig. 9 shows the plane layout of the monitoring net-

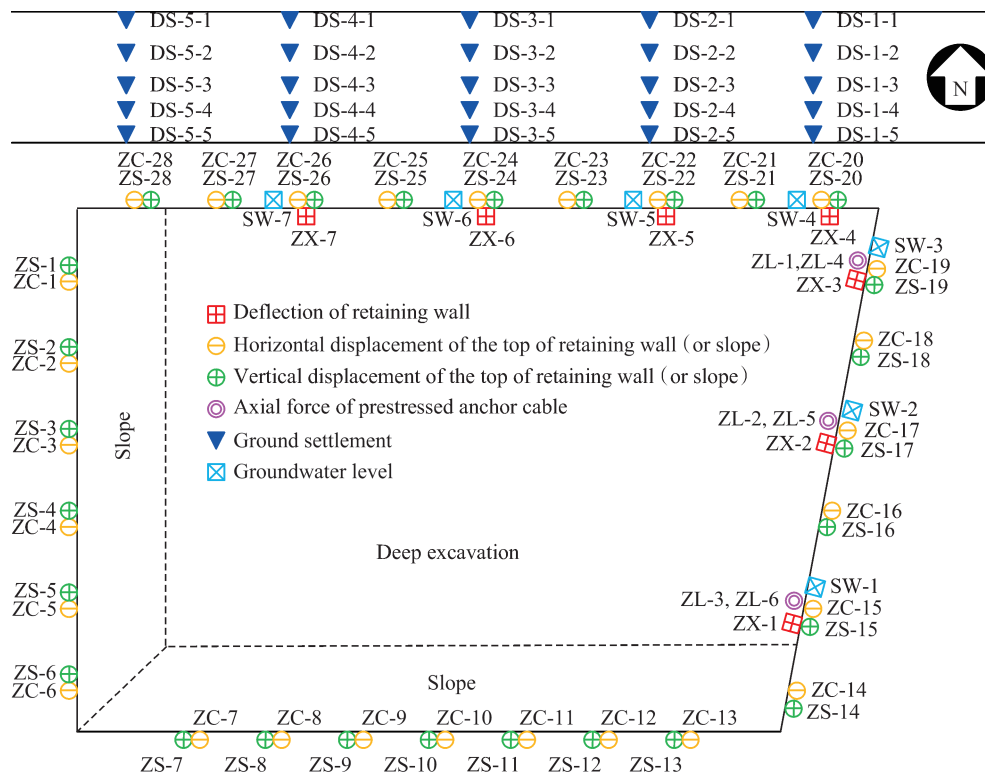


Fig. 9 Plane layout of the monitoring items

work. Seven deflection gauges (ZX- a , $a=1, 2, \dots, 7$) were installed on the retaining walls; 28 points (ZC- a , $a=1, 2, \dots, 28$) measured horizontal displacements at wall/slope tops; 28 points (ZS- a , $a=1, 2, \dots, 28$) measured vertical displacements at the same locations; six points (ZL- a , $a=1, 2, \dots, 6$) monitored axial forces of prestressed anchor cables; five settlement sections (DS- a - b , $a=1, 2, \dots, 5$; $b=1, 2, \dots, 5$) provided 25 ground settlement points; and seven observation wells (SW- a , $a=1, 2, \dots, 7$) monitored groundwater levels.

4.2 Monitoring frequency

Field monitoring commenced on 16 February, 2023 and concluded on 13 August, 2023, covering the entire construction period (about 6 months). The monitoring frequency varied with the construction stage and is summarized in Table 7. During dewatering and earthwork excavation (Stage 1), monitoring frequency gradually increased as excavation progressed. During base slab casting (Stage 2) and main structure construction (Stage 3), the monitoring frequency gradually decreased.

4.3 Analysis of monitoring results

4.3.1 Deflection of retaining wall

Fig. 10 shows the deflection development curves of the

Table 7 Monitoring frequency of foundation pit

Stage No.	Construction activity	Construction time	Monitoring frequency/d ⁻¹
1	Dewatering and earthwork excavation	2023-02-16—2023-03-18	1/3
		2023-03-19—2023-04-06	1/2
		2023-04-07—2023-05-09	1
2	Base slab casting	2023-05-10—2023-05-31	1/2
		2023-06-01—2023-07-06	1/3
3	Main structure construction	2023-07-07—2023-07-20	1/7
		2023-07-21—2023-08-13	1/10

retaining wall at each monitoring point. It is found that the retaining wall exhibited a cantilever deflection profile: lateral displacement decreases with depth, and the maximum lateral displacement occurs at the top of the wall. The deflection of the retaining wall increased with construction progress, particularly during Stage 1; growth during Stages 2 and 3 was very limited. This observation indicates that wall deflection mainly occurred during the dewatering and earthwork excavation stage, and was driven by excavation-induced lateral unloading of the pit wall. During base slab casting and main structure construction, lateral unloading of the pit wall did not increase, and subsequent deflection of the retaining wall was mainly time-dependent.

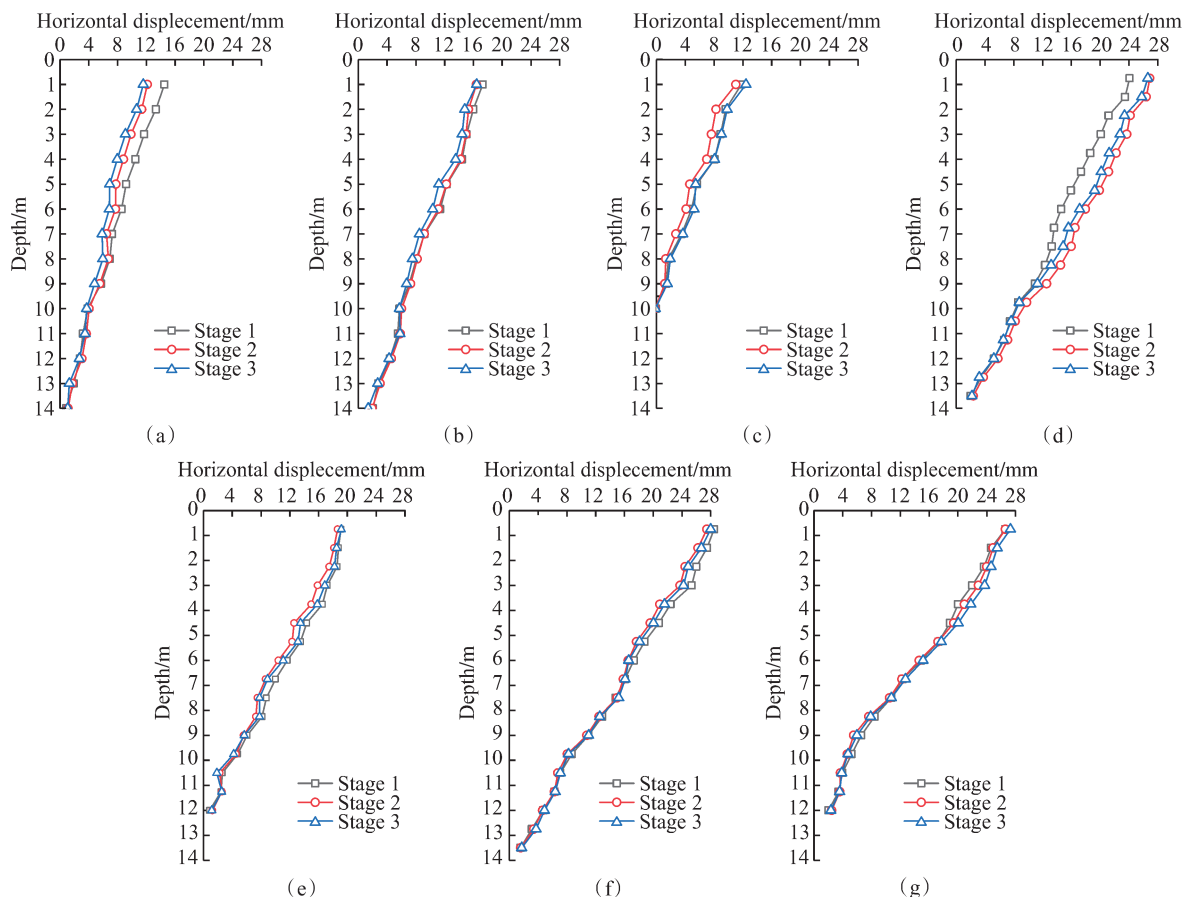


Fig. 10 Deflection development curves of retaining wall. (a) ZX-1; (b) ZX-2; (c) ZX-3; (d) ZX-4; (e) ZX-5; (f) ZX-6; (g) ZX-7

To clarify the spatial distribution, Stage 3 monitoring results were selected for analysis (Fig. 11). On the east side, ZX-2 recorded the greatest deflection, with a maximum of 16.46 mm. On the north side, apart from a small deflection at ZX-5, deflections at the other north-side points are similar, with a maximum of 28.0 mm at ZX-6. The maximum deflection of the retaining wall on both east and north sides does not exceed the warning value in Table 6. Therefore, the adopted design and construction schemes achieved satisfactory deformation control.

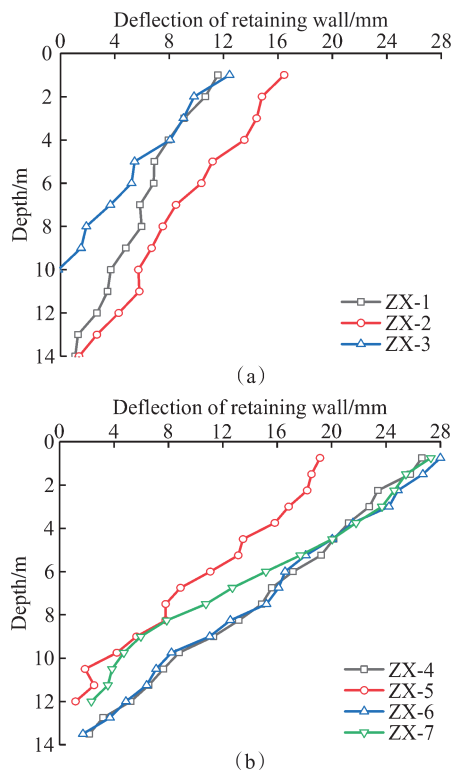


Fig. 11 Deflection distribution curves of retaining wall. (a) East side; (b) North side

Furthermore, the north side deflection is generally greater than the east side deflection, consistent with the numerical prediction. This difference may reflect: (1) longer anchor cable length on the east side, which increases resistance to deformation; (2) higher ground loads adjacent to the municipal road on the north side, which increase lateral pressure on the retaining wall.

4.3.2 Horizontal displacement of the top of the retaining wall

Fig. 12 shows the horizontal displacement development curves of the top of the retaining wall. The top of the retaining wall moved horizontally into the pit, and displacement gradually increased with time. The growth rate and magnitude depended on the construction stage. Specifically, Stage 1 produced rapid displacement, accounting for more than 70% of the final value. Stage 2 growth slowed and contributed less than 30%. In Stage 3, the horizontal displacement remained essentially stable.

Fig. 13 shows the horizontal displacement distribution

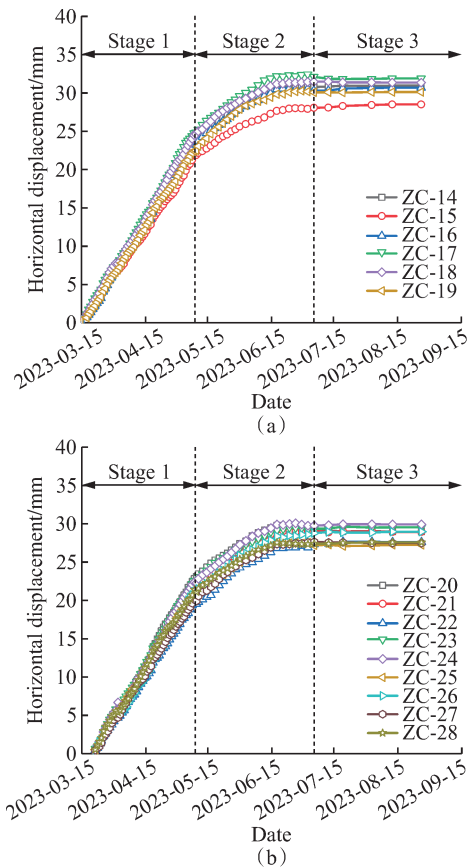


Fig. 12 Horizontal displacement development curves of the top of the retaining wall. (a) East side; (b) North side

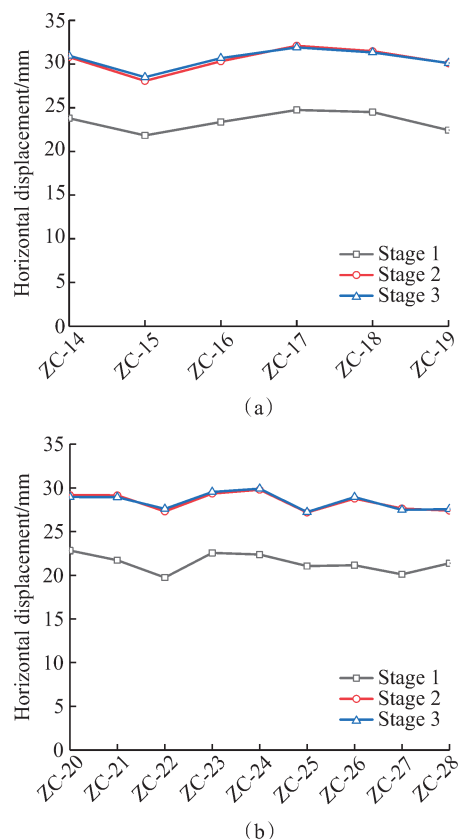


Fig. 13 Horizontal displacement distribution curves of the top of the retaining wall. (a) East side; (b) North side

curves at the top of the retaining wall. The spatial distribution patterns in Stages 1 to 3 are generally consistent. The maximum horizontal displacement on the east side of the foundation pit is 31.90 mm at ZC-17, while that on the north side is 29.92 mm at ZC-24. Both values are below the control values in Table 6, supporting the reliability of the adopted design and construction schemes.

4.3.3 Vertical displacement of the top of the retaining wall

Fig. 14 shows the vertical displacement development curves of the top of the retaining wall. The top of the retaining wall settled over time. The growth rate and magnitude of settlement are related to the construction stages. Specifically, in Stage 1, settlement increased rapidly (over 60% of the final settlement). In Stage 2, the growth slowed (contributing less than 40%). In Stage 3, settlement remained essentially stable.

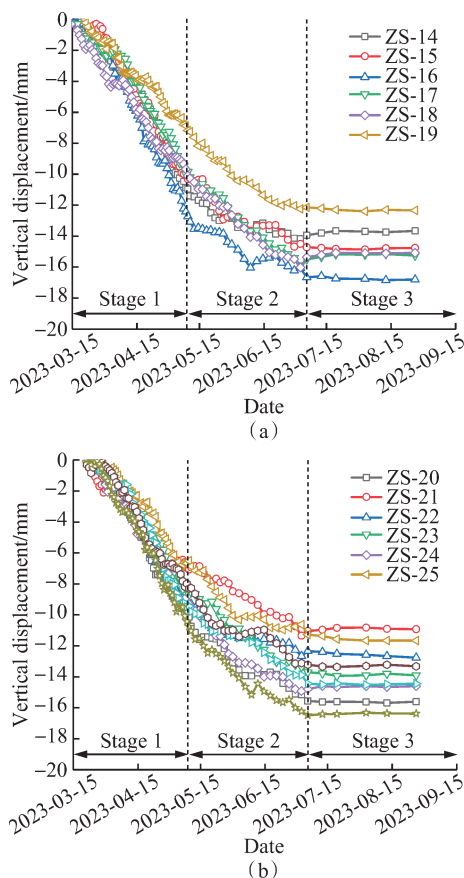


Fig. 14 Vertical displacement development curves of the top of the retaining wall. (a) East side; (b) North side

Fig. 15 shows the vertical displacement distribution curves at the top of the retaining wall. The spatial distribution patterns of vertical displacement in Stages 1 to 3 are generally consistent. The maximum settlement on the east side of the foundation pit is -16.8 mm, occurring at ZS-16; on the north side, the maximum is -14.6 mm, occurring at ZS-24. Both maxima are within the control limits of Table 6, supporting the reliability of the adopted design and construction schemes.

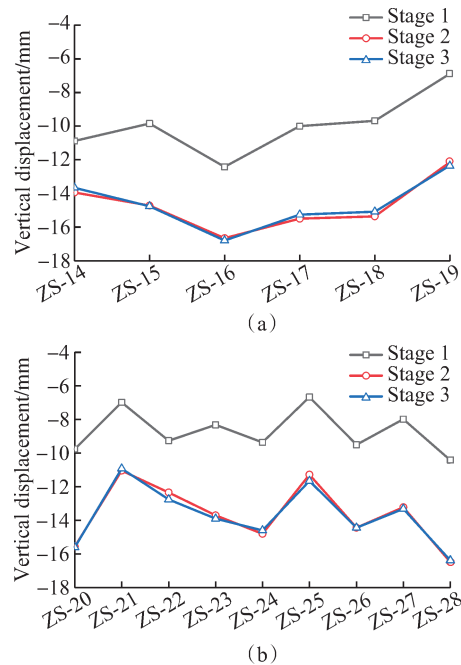


Fig. 15 Vertical displacement distribution curves of the top of the retaining wall. (a) East side; (b) North side

4.3.4 Axial force of prestressed anchor cable

Fig. 16 shows the development curves of the axial force of the prestressed anchor cable. In the early and middle periods of Stage 1, the axial force increased; in

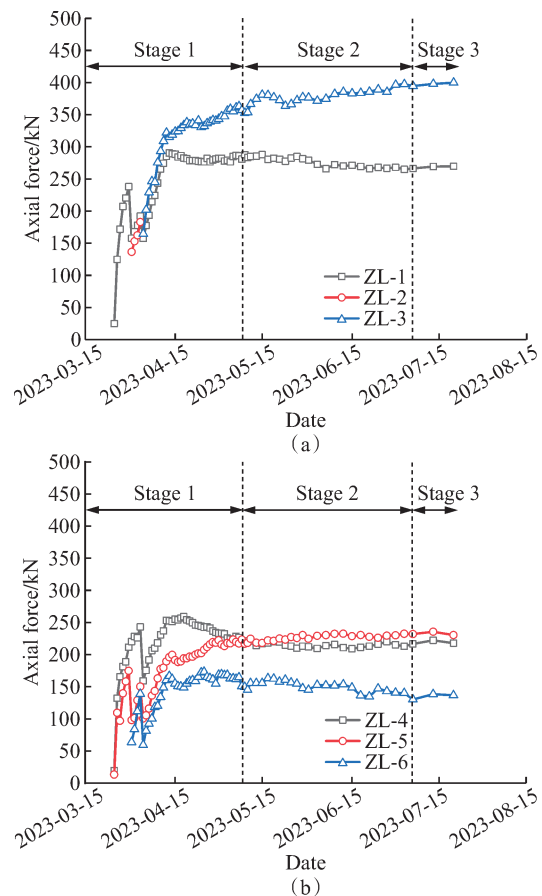


Fig. 16 Axial force development curves of prestressed anchor cable. (a) The 1st level of anchor cable; (b) The 2nd level of anchor cable

the late period of Stage 1 and in Stages 2 and 3, it remained relatively stable. This likely resulted from earthwork excavation above the anchor heads, which occurred primarily in the early and middle periods of Stage 1, accelerating anchor deformation and increasing stress during those periods. Also, the axial force of the upper (1st) anchor level exceeds that of the lower (2nd) level, consistent with the afore-mentioned cantilever deformation feature of the retaining wall.

Monitoring points for anchor forces are instrumented in sections *EF*, *FG*, and *GH* (Fig. 2). Design axial forces for the upper 1st level of anchor cables are 380,

375, and 390 kN, and those of the lower 2nd level of anchor cables are 230, 245, and 220 kN. The measured axial forces fall within the design ranges.

4.3.5 Ground settlement

Fig. 17 shows the development curve of ground settlement outside the pit. Settlement patterns across the monitoring sections were consistent and closely correlated with the construction sequence. In Stage 1, the ground settlement increased rapidly and accounted for more than 90% of the final settlement. In Stage 2, settlement growth slowed and contributed at most about 10%. In Stage 3, settlement remained essentially stable.

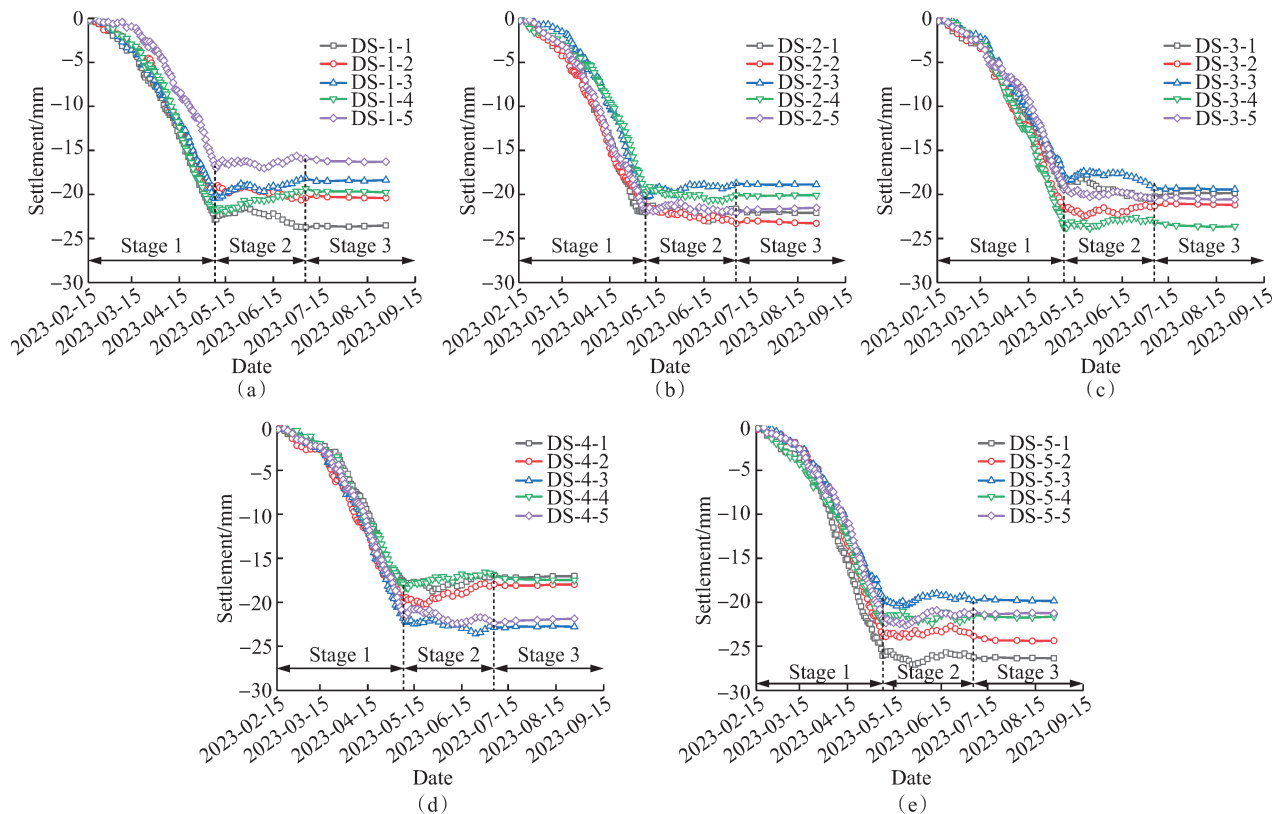


Fig. 17 Development curves of ground settlement. (a) The 1st section; (b) The 2nd section; (c) The 3rd section; (d) The 4th section; (e) The 5th section

Fig. 18 shows the spatial distribution of ground settlement outside the pit after monitoring is complete. The location of the maximum settlement within a given section varied without a simple pattern. The maximum ground settlement was 26.36 mm, occurring at DS-5-1, which

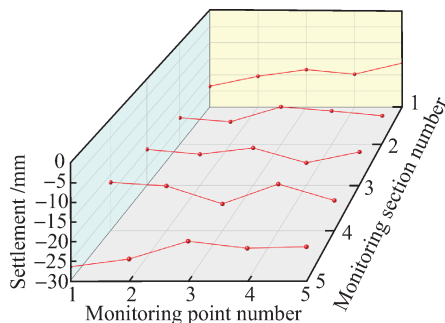


Fig. 18 Spatial distribution curves of ground settlement

does not exceed the control value in Table 6. Thus, the adopted design and construction schemes ensured environmental safety.

4.3.6 Groundwater level

Fig. 19 shows the development curve of the groundwater level outside the pit. In the early period of Stage 1, groundwater levels at some observation wells rose due to reinjection from pit dewatering. In late Stage 1 and in Stage 2, groundwater levels outside the pit fluctuated without a sustained increase. In Stage 3, dewatering is complete, and the groundwater level outside the pit remains stable. At the end of monitoring, the maximum groundwater level increase was 699 mm at SW-1, which did not exceed the control value in Table 6. This indicates that the waterproof curtain effectively isolated the groundwater inside and outside the pit.

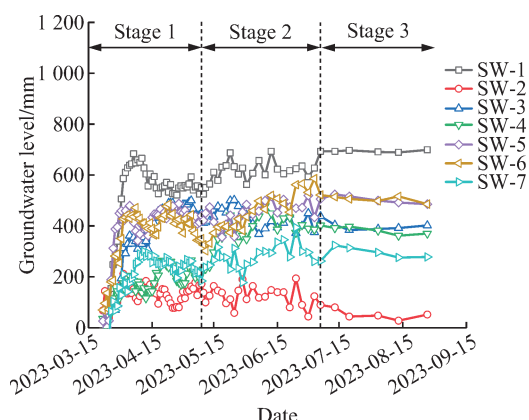


Fig. 19 Development curves of groundwater level outside the pit

5 Conclusions

(1) The deflection of the retaining wall, horizontal and vertical displacements at the retaining wall top, and ground settlement mainly occurred during the dewatering and earthwork excavation stage. Additional movement during base slab casting and main structure construction was limited. Maximum displacements occurred on the north side of the foundation pit due to the spatial size effect.

(2) During the construction of the foundation pit, the anchor cables remained in tension. Axial forces increased notably in the early and middle periods of the dewatering and earthwork excavation stage, then stabilized in the later stages. Axial force of anchor cable correlates closely with the horizontal displacement of retaining wall, and upper anchors developed greater axial forces than lower anchors.

(3) Groundwater levels outside the pit rose during the early earthwork excavation period (attributed to reinjection related to pit dewatering) but were stable during base slab casting and main structure construction. The waterproof curtain effectively seals the hydraulic connection both inside and outside the pit.

(4) Given the site's complex geological and hydrological conditions, the composite pile-anchor plus grading excavation form controlled construction-induced deformation and environmental responses within safe limits, providing a practicable and economic solution for water-rich reclaimed soft soil sites with tight construction schedules.

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临海富水软土地层复合式支护基坑开挖性能及环境响应

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摘要: 临海富水软土地层某深基坑面临变形大、渗漏严重和施工工期紧等风险和挑战。基于复杂的地质、水文条件, 综合考虑安全性和经济性, 提出了桩锚联合放坡开挖的复合支护模式; 然后, 通过数值模拟预测该方案下施工引起的围护结构变形和环境响应, 初步验证所提方案的可行性; 最后, 建立实时监测系统, 对施工过程中的围护桩水平和竖向位移、锚索轴力、地表沉降、坑外地下水位进行监测, 明确了其时间演化和空间分布特征, 验证了所用方案的有效性。结果表明: 基坑围护结构变形及环境响应主要发生在基坑降水开挖阶段; 锚索呈受拉状态, 且拉力与围护桩侧移相关性较强, 上层拉力大于下层拉力; 采用桩锚联合放坡开挖方案后, 围护桩最大侧移为 28.0 mm, 地表最大沉降为 26.36 mm, 坑外水位最大变化量为 699 mm, 均在安全控制值范围内。

关键词: 基坑; 软土; 桩锚; 放坡开挖; 数值模拟; 实测性能

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