

DOI: 10.11835/j.issn.2096-6717.2024.123



开放科学(资源服务)标识码OSID:



局部渗漏对富水砂层基坑挡墙变形和地表沉降的影响

邱明明¹, 李晓敏¹, 赵海峰¹, 段君义², 杨果林³

(1. 延安大学 建筑工程学院, 陕西 延安 716000; 2. 南昌大学 工程建设学院, 南昌 330031;
3. 中南大学 土木工程学院, 长沙 410075)

摘要:富水砂层深基坑变形稳定可控是地下工程施工安全的基础保障。针对富水砂层深基坑渗漏灾害问题,以某地铁车站端头井深基坑渗漏事故为背景,通过现场调查和实测数据分析,总结基坑漏水漏砂灾害发展过程及事故原因,系统研究基坑渗漏水期间挡墙水平位移、墙后地表沉降、坑外地下水位的变化特征,并结合数值模拟方法揭示局部渗漏对基坑变形性状的影响规律。结果表明,局部渗漏引起挡墙水平位移显著增加,渗漏后挡墙最大水平位移约为渗漏前的1.29~1.47倍;局部渗漏引起敏感影响区沉降槽加深变宽,渗漏后最大地表沉降约为渗漏前的1.16~2.67倍;坑外地下水位“跳跃式”波动变化特征可作为基坑发生渗漏灾害前兆信号的重要判据;采取“注浆+高压旋喷桩”联合处治措施能有效应对富水砂层基坑渗漏灾害控制难题。

关键词:富水砂层;深基坑;局部渗漏;变形性状;数值模拟

中图分类号: TU751 文献标志码: A 文章编号: 2096-6717(XXXX)XX-0001-16

Effect of partial leakage during deep excavation in water-rich sandy strata on retaining wall deformation and ground settlement

QIU Mingming¹, LI Xiaomin¹, ZHAO Haifeng¹, DUAN Junyi², YANG Guolin³

(1. School of Architectural Engineering, Yan'an University, Yan'an 716000, Shaanxi, P. R. China; 2. School of Infrastructure Engineering, Nanchang University, Nanchang 330031, P. R. China; 3. School of Civil Engineering, Central South University, Changsha 410075, P. R. China)

Abstract: The stable and controllable deformation of deep foundation pits in water-rich sandy strata is the basic guarantee of underground engineering construction safety. Focusing on the leakage disaster of deep foundation pits in water-rich sandy strata, this paper takes the leakage incidents of launch shaft pits of a subway station as the research background. We then summarize and analyze the development process and accidental causes of water and sand leakage disasters through field investigation and survey data review, as well as the characteristics of horizontal displacement of the retaining wall and ground settlement behind the wall. The change of groundwater level outside the pit during partial leakage was systematically studied, and the effect rules of partial leakage on the deformation behavior of the foundation pit were revealed using a numerical simulation method. The results show that the retaining wall's horizontal displacement increases significantly due

Received: 2024-09-24

Foundation items: Natural Science Basic Research Program of Shaanxi Province (No. 2019JQ-834); Natural Science Basic Research Program of Shaanxi Provincial Education Department (No. 21JC035); Key Research Project of Yan'an Universities (Nos. 2021ZCQ011; 2023KXJ-010).

Author brief: QIU Mingming (1985-), PhD, associate professor, main research interest: geotechnical and underground engineering, E-mail: sxdfqiuming@163.com.

to partial leakage. Additionally, the maximum horizontal displacement of the retaining wall after leakage is 1.29 to 1.47 times that before leakage. The ground settlement trough of the sensitive zone deepens and widens due to partial leakage, and the maximum ground settlement after leakage is 1.16 to 2.67 times that before leakage. Thus, the characteristics of "jump" fluctuation of groundwater level can be used as an essential criterion for the precursor of leakage disaster in foundation pit. Importantly, the combined treatment measures of grouting and high-pressure jet spouting piles can effectively deal with the problem of leakage disaster control of foundation pits in water-rich sandy strata.

Keywords: water-rich sandy strata; deep foundation pit; partial leakage; deformation behavior; numerical simulation

1 Introduction

The high-quality development and sustainable utilization of urban underground space have entered a new stage. Additionally, the deep foundation pit projects with deep, large, difficult, and dangerous characteristics have increased rapidly. As deep underground excavation inevitably disturbs the surrounding environment and induces large strata deformation or structural damage, the concern of foundation pit leakage disaster has become a prominent problem in underground engineering^[1-3]. Engineering practice shows that leakage disasters of foundation pits are mainly induced by soil and water loss caused by groundwater seepage^[4]. Moreover, its occurrence process was characterized by suddenness, concealment, complexity, and high harm, threatening the safety of underground structures and the surrounding environment^[5-7]. Therefore, it is necessary to systematically study the problem of foundation pit leakage disasters to achieve the purpose of disaster prediction and prevention. Furthermore, these efforts can provide a theoretical basis and technical support for large-scale construction of urban underground engineering.

Domestic and foreign scholars have researched the evolution laws of deep foundation pit leakage disasters using case analysis, model tests, and numerical simulation. Based on analyzing the development process and deformation failure characteristics of leaking water and sand leakage in the foundation pit in regards to case analysis, it is pointed out that the partial leakage of the foundation pit causes soil loss, surface collapse, and other disasters^[8-10]. Jo et al.^[11] analyzed the leakage accident of a deep foundation pit in alluvial soil with an excavation depth of

38m using geological exploration and other means. Through statistical analysis, Liu et al.^[12] and Fan et al.^[13] pointed out that exposed steel bars, joint defects, partial wall bulges, and complex hydrogeological conditions are the leading causes of foundation pit leakage. Jiang et al.^[14] also analyzed the ground settlement characteristics caused by the leakage of the underground diaphragm wall (UDW). They believed that improper construction of the UDW and joint treatment were essential causes of the leakage of the retaining wall. Based on the measured data, Zhang et al.^[15] found that increased active earth pressure caused by foundation pit leakage was the main reason for the retaining wall's sudden deformation, with the surrounding soil lagging. Tan et al.^[16] analyzed the leakage accident in the foundation pit of the Shanghai subway and found that the ground settlement outside the pit caused by leakage could reach 42 mm. They proposed that the high-pressure rotating spouting and mixing piles could seal the leakage channel. In model testing, Zheng et al.^[17] designed a new visualization experimental device to investigate the leaking process of sand and water, revealing the catastrophe laws of water and sand leakage in underground engineering. Li et al.^[18] showed through centrifugal model tests that the leakage of the UDW resulted in a significant increase in the amount and range of ground settlement. Cao et al.^[19] analyzed the influence laws of soil leakage on sandy soil foundation pits with the help of model tests. As such, they pointed out that soil leakage caused serious harm to the surrounding environment of the foundation pit. They suggested that soil pile reverse pressure measures should be taken to prevent the deterioration of foundation pit deformation. In the aspect of numerical simulation, FEM^[7, 20-21] and DEM-

CFD^[22-23] coupling methods were widely used in the numerical simulation analysis of foundation pit leakage. Hu et al.^[20] established a plane strain model of foundation pit seepage failure by FEM software, and the permeability failure mode of foundation pit with silty sand foundation was analyzed. Dai et al.^[23] used the DEM-CFD coupled simulation method to reveal the ground movement laws induced by leakage of groundwater and sand in excavations. Wu et al.^[24-25] established a three-dimensional fluid-structure coupling analysis model to simulate the process of foundation pit leakage, and the research results showed that the change of ground settlement caused by water leakage was closely related to the location of the water leakage point, the amount of water seepage, soil permeability, and other factors. Chao et al.^[26] further established a three-dimensional DEM-CFD coupling model of foundation pit leakage. They pointed out that the leakage of UDW will cause the active water and soil pressure near the leakage point to increase by 5.7 ~ 8.5 times, and the soil above the leakage position will undergo significant settlement deformation.

This report investigates the accident causes, deformation characteristics, and foundation pit leakage control measures under different soil properties. In doing so, these can support structure types and excavation depths in other regions, providing helpful guidance for researching foundation pit leakage and its prevention. However, due to the complexity of the foundation pit leakage process and the difference in geological conditions, the characteristics and disaster mechanism of foundation pit leakage under different support types and geological conditions have not been fully grasped. Relevant case data and treatment experience must be further accumulated, especially for deep foundation pit engineering with water-rich sandy strata. Given this, this paper takes the leakage incidents of launch shaft pits of subway stations as the research background, summarizes and analyzes the development process and accident causes of water and sand leakage disaster through field investigation and survey data review, the characteristics of horizontal displacement of retaining wall, ground settlement behind wall and change of groundwater level outside

the pit during partial leakage were systematically studied. The numerical simulation method revealed the effect of the rules of partial leakage on the foundation pit's deformation behavior. Notably, the research results can provide a reference for the design and disaster prevention of deep foundation pits in water-rich sandy strata.

2 Engineering background

2.1 Soil Conditions

The length $\times$ width $\times$ depth of the deep foundation pit of a city subway station is about 470.0 m $\times$ 23.1 m $\times$ 17.9 m, which is constructed using an open-cut method. The strata of the proposed site are mainly Quaternary Holocene artificial fill (Qh₄^{ml}) and alluvial layer (Qh₄^{al}), as well as Paleogene Xinyu Group (Exn) bedrock. The soil layers of the site from top to bottom are plain fill, silty clay, silt, medium sand, coarse sand, gravel sand, strongly weathered argillaceous siltstone, moderately weathered argillaceous siltstone, and slightly weathered argillaceous siltstone. The physical and mechanical parameters of the rock and soil layers revealed by the site exploration are shown in Table 1. The excavation depth of the foundation pit is mainly a sandy soil layer. The particle gradation curve of sandy soil is shown in Fig. 1, and the average permeability coefficient is about 10^{-3} - 10^{-2} cm/s. The groundwater is mainly the pore phreatic water in the gravel layer, and the buried depth is about 4.1-6.5 m. Notably, rainfall and adjacent river water significantly affect the groundwater level change.

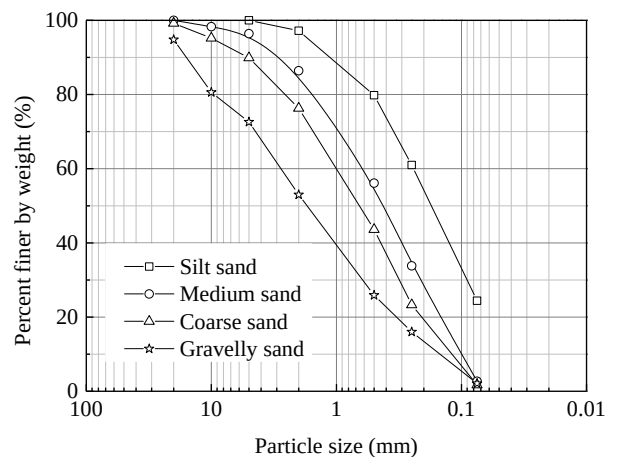


Fig. 1 Particle gradation curves of sandy soil

Table 1 Physical and mechanical parameters of soil layers at the site

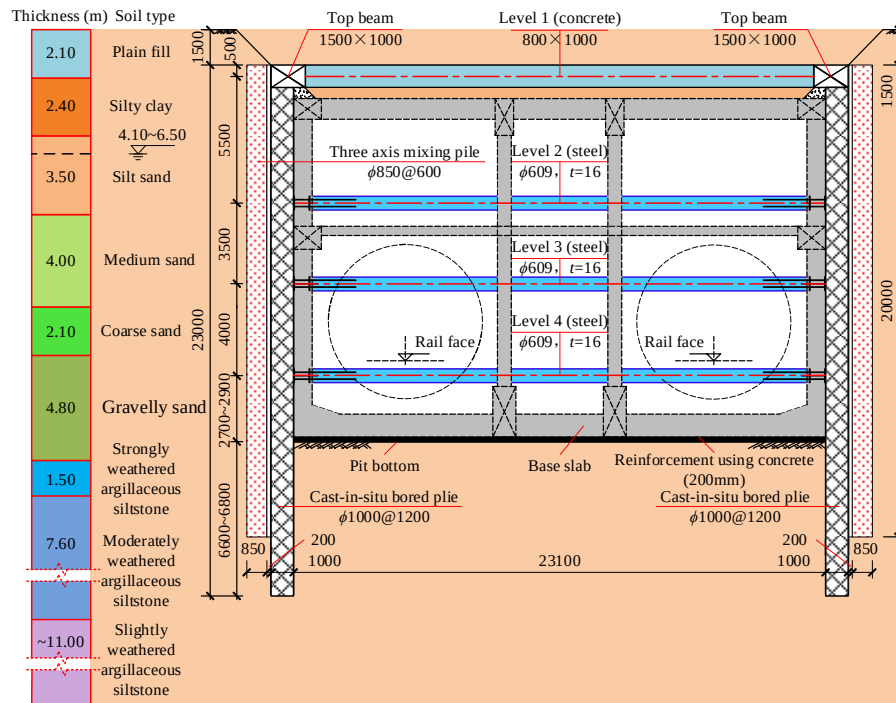
No.	Soil layer	h/m	$\gamma/(kN/m^3)$	c/kPa	$\varphi/(^\circ)$	$k/(cm/s)$	E_s/MPa	ν
1-2	Plain fill	1.0-4.8	17.0	3.0	10.0	2.00×10^{-4}	5.00	0.30
2-1	Silty clay	0.3-3.0	18.7	19.3	18.0	4.00×10^{-5}	9.56	0.34
2-2	Silt sand	0.5-9.0	17.9	10.8	30.0	0.80×10^{-3}	11.30	0.34
2-4	Medium sand	0.5-5.0	18.5	0	35.0	1.20×10^{-2}	14.40	0.31
2-5	Coarse sand	1.0-7.4	19.0	0	38.0	6.00×10^{-2}	15.80	0.31
2-6	Gravelly sand	2.5-6.0	19.5	0	40.0	8.00×10^{-2}	19.70	0.30
5-1	Strongly weathered argillaceous siltstone	0.4-2.3	21.0	50.0	23.0	2.48×10^{-6}	600	0.34
5-2	Moderately weathered argillaceous siltstone	6.4-10.6	24.3	600.0	35.0	4.50×10^{-7}	1500	0.34
5-3	Slightly weathered argillaceous siltstone		24.7	800.0	40.0	3.00×10^{-8}	2500	0.28

Note: h is thickness; γ is unit weight of soil; c is effective cohesion; φ is internal friction angle; k is vertical permeability coefficient; E_s is compression modulus; ν is Poisson ratio.

2.2 Retaining structure

The excavation depth of the launch shaft is approximately 16.7 to 17.9 m (Fig. 2). The retaining structure of the foundation pit is composed of a cast-in-situ bored pile (CSBP) and a three-axis mixing pile (TAMP). The size of CSBP is 23.0 m in depth, 1.0 m in diameter, and 1.2 m in spacing, and the size of TAMP is 20.0 m in depth, 0.85 m in diameter, and 0.6 m in spacing. The bi-slurry grouting reinforced the gap between the CSBP and

the TAMP to enhance the preventing water function of the retaining structure. A combination of concrete and section steel supported the foundation pit. The first structure (Level 1) is supported by a reinforced concrete beam, and its size is 0.8 m in wide, 1.0 m in height, and 9.0 m in spacing. The second, third, and fourth structures (Levels 2 to 4) are supported by steel pipes; their size is 0.69 m in diameter, 0.016 m in thickness, and 3.0 m in spacing.

**Fig. 2** Profile of the excavation pit retaining structure (unit: mm)

2.3 Field monitoring program

The supporting structure and surrounding environment are continuously monitored during excavation, including horizontal displacement of the retaining pile, ground settlement behind the wall, and

groundwater level outside the pit. The plan layout of monitoring points of the excavation pit is shown in Fig. 3. Among them, the 40 inclinometer points are arranged for horizontal displacement of the retaining pile, designated as CX-1 to CX-40, respectively.

The cumulative displacement alarm value of the horizontal displacement of the retaining pile does not exceed $0.20\%h_e$ (h_e , excavation depth). It is not greater than 20.0 mm, and the change rate of horizontal displacement does not exceed 3.0mm/d. There are 40 monitoring sections of ground settlement behind the wall. Each section has 5 monitoring points, designated as DS1- i and DS40- i (i is the number of measuring points). The cumulative displacement alarm value of ground settlement does not exceed 30.0 mm, and the change rate of ground settlement does not exceed 3.0 mm/d. The 24 monitoring points are arranged for groundwater level outside the pit, designated as SW-1 to SW-24, respectively. Furthermore, the cumulative displacement alarm value of the groundwater level does not exceed ± 1.0 m, and the change rate of the groundwater level does not exceed ± 0.5 mm/d.

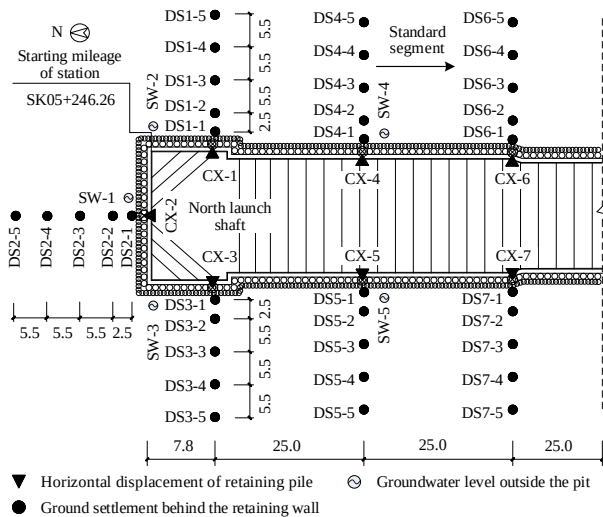


Fig. 3 Layout of the excavation pit monitoring points (unit: m)

3 Analysis of measured results of leakage incident during excavation

3.1 Development process of leakage during excavation

When the north launch shaft was excavated to the bottom of the pit 15.0 to 17.9 m, groundwater, and sand leakage occurred between the enclosing piles (Fig. 4 and 5). As such, this presented the characteristics of fast leakage speed and large flow rate, and instantaneous surface collapse occurred at the corresponding section of the leakage location. Subsequently, the groundwater and sand leakage

rate slowed, and the surface subsidence gradually slowed until it stopped. Among them, the first leakage occurred on the west side of the north launch shaft. About 255 min after excavating the fifth layer of soil, there was rapid leakage of groundwater lasting 3 to 5 min between CSBP WA-05 to WA-07. The leakage location was about 16.0 m depth from the surface, and the volume of the flowing sand was about 126.0 m³. Simultaneously, surface collapse occurred about 3.55 m away from the edge of the foundation pit. After the first large leakage of groundwater and sand, a small amount of sand continued to flow out, and the surface collapsed and slowly sank with the flowing sand. After about 195 min of leakage, the leaked groundwater of the retaining wall and surface collapse was not obvious. Moreover, a collapse pit of 8.0 m in length, 5.0 m in width, and 3.5 m in depth was finally formed. The second leakage occurred at the east side of the north launch shaft. About 80 min after the soil excavation of the fifth layer, there was a phenomenon of rapid leakage of groundwater and sand between the CSBP EA-06 and EA-07. The leakage location was about 17.0 m deep from the surface, and the volume of flowing sand was about 75.0 m³. The surface collapse occurred about 2.35 m away from the edge of the foundation pit, forming a collapse pit of 8.0 m in length, 4.0 m in width, and 2.5 m in depth. The retaining wall's groundwater leakage and surface collapse were not apparent after the leakage for about 107 min.

By comparing the two leakage conditions, it was found that the leakage location occurred in the long side direction of the north launch shaft, and the depth was between Level 4 and the bottom of the pit. The leakage existed in the form of a single point, but there was no continuity, and the location of the leakage point was in the same section. The symptoms were not obvious before the leakage of groundwater and sand; it showed the characteristics of being instantaneous; the leakage process lasted about 3 to 5 min, and the surface collapses with the leakage of groundwater and sand in the pit. Combined with the field observation phenomenon and measured data, the development process of foundation pit leakage disaster could be described as a

change in the underground hydraulic environment, visible wet spots on the retaining wall surface, partial leakage of the retaining wall, leakage spot expansion, forming through leakage channels, short dura-

tion water and sand gushing in pit, soil and water loss in the strata behind the wall, and large deformation of retaining wall and surface collapse.

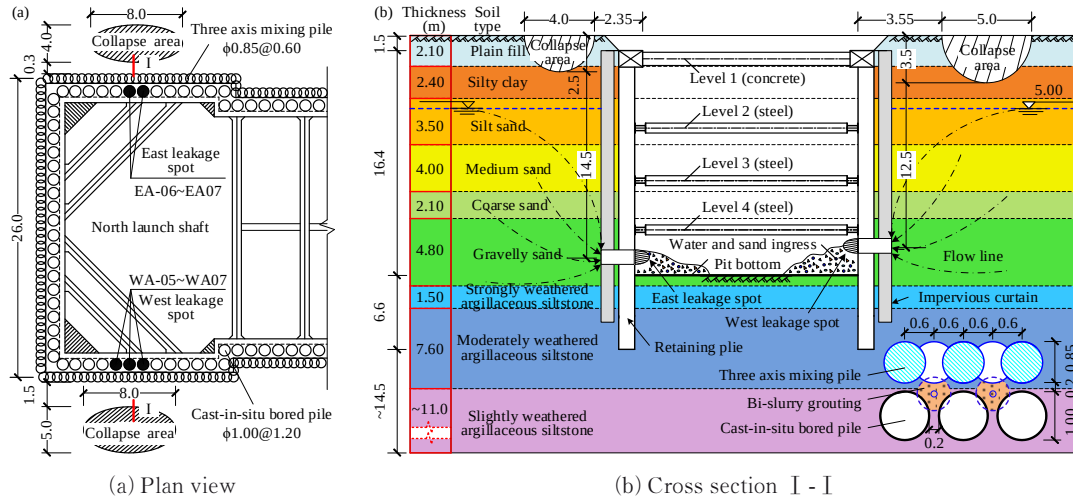


Fig. 4 The partial leakage of groundwater and the sand location at the north launch shaft (unit: m):



(a) Leakage of groundwater and sand

(b) Leakage spot

(c) Ground collapse

Fig. 5 Field observation of leakage groundwater and sand of north launch shaft during excavation

According to the development characteristics and induced factors of leakage disaster in this project, the combined treatment measures of grouting and high-pressure jet spouting pile were adopted to seal the leakage area, and the field measurement showed that the horizontal displacement of the retaining wall and the ground settlement behind the wall were effectively suppressed. At the same time, this method was adopted to pre-strengthen the retaining wall of the standard section and the south launch shaft of the pit in subsequent construction. Long-term monitoring showed that the lateral deformation of the retaining wall, ground settlement, and groundwater level change was kept within the control value range (Fig. 6 to 8), and the foundation pit's overall anti-seepage effect and stability were good.

3.2 Observed excavation behaviors during partial leakage

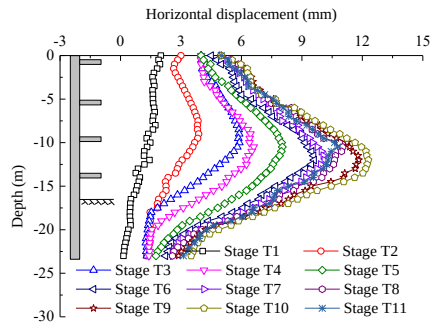
3.2.1 Variation characteristics of horizontal dis-

placement of retaining pile

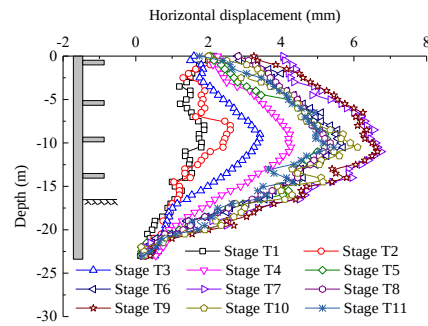
Fig. 6 shows the distribution curves of horizontal displacement of the retaining pile under different conditions. It can be seen that during shallow layer excavation, the horizontal displacement of the retaining pile to the pit is approximately "oblique", and the displacement value of the pile top is the largest. With increased excavation depth, the retaining pile's horizontal displacement keeps increasing and shows an inward convex trend. Due to the constraint effect of internal support and soil at the bottom of the pile, the horizontal displacement in the middle of retaining the pile is larger, and the horizontal displacement at the top and bottom of the retaining pile is smaller, which is approximately in an "inverted V" shape distribution. After the foundation pit leaks, the retaining pile's horizontal displacement increases. The horizontal displacement of the retaining pile tends to be stable with the pouring of the cushion layer and bottom

plate. When the foundation pit is excavated to the bottom of the pit, the maximum horizontal displacement of retaining pile of the measuring points CX-1 (the long side direction of the north launch shaft), CX-2 (the short side direction of the north launch shaft), CX-3 (the long side direction of the north launch shaft) and CX-38 (the long side direction of the south launch shaft, without leakage) are 8.03, 5.25, 11.92 and 4.36 mm, respectively. When partial leakage occurs in the foundation pit, the maximum horizontal displacements of the retaining pile of measuring points CX-1, CX-2, and CX-3 are

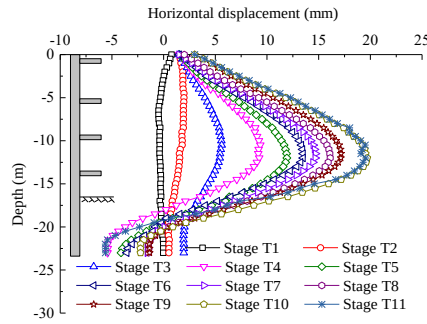
11.83, 6.75, and 17.16mm, respectively. Notably, these are about 1.47, 1.29, and 1.44 times before leakage. When the base slab is finished pouring, the maximum horizontal displacements of the retaining pile at CX-1, CX-2, CX-3, and CX-38 are respectively 10.77, 5.45, 19.55 and 6.26, which increased by 34.12%, 3.81%, 64.01%, and 43.58% compared with before leakage. Compared with the north launch shaft, the maximum horizontal displacement of the south launch shaft retaining pile without leakage is about 32.02% of that of the north launch shaft.



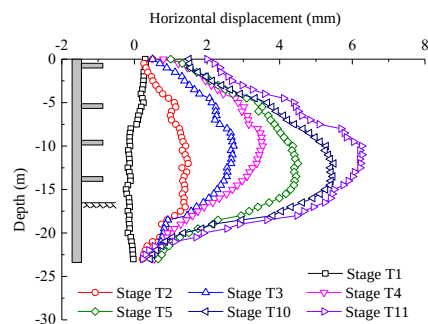
(a) Location: point CX-1 of north launch shaft



(b) Location: point CX-2 of north launch shaft



(c) Location: point CX-3 of north launch shaft



(d) Location: point CX-38 of south launch shaft

Note: Stage T1 represents level 1 (concrete), excavation depth is 2.0 m; Stage T2 represents level 2 (steel), excavation depth is 7.5 m; Stage T3 represents level 3 (steel), excavation depth is 11.0 m; Stage T4 represents level 4 (steel), excavation depth is 15.0m; Stage T5 represents excavation to bottom, excavation depth is 17.9m; Stage T6 represents leakage groundwater and sand on the west of north end shaft; Stage T7 represents seepage reinforcement on the west of north end shaft; Stage T8 represents leakage groundwater and sand on the east of north end shaft; Stage T9 represents seepage reinforcement on the east of north end shaft; Stage T10 represents construction of foundation cushion; Stage T11 represents construction of base slab.

Fig. 6 Distribution curves of horizontal displacement of retaining pile under different conditions

The above results demonstrated that the foundation pit's partial leakage would aggravate the retaining pile's horizontal displacement and was affected by the size of the foundation pit and the corner effect^[27-28]. As such, the retaining pile's horizontal deformation in the launch shaft's long side direction under partial leakage was greater than that in the short side direction. The reasons for this mainly

included the soil and water loss behind the wall under the action of high water pressure leakage, resulting in the impact of additional dynamic water load on the pile^[7, 26], causing a significant increase in soil and water pressure behind the wall. Moreover, the leakage of the retaining wall caused a delay in the construction period, which increased the exposure time of the foundation pit without a closed supporting

structure, and the time effect of retaining wall deformation was prominent. During the reinforcement of foundation pit leakage, post-pile grouting reinforcement, surface bulk plain concrete backfill, and the groundwater level after grouting reinforcement rebounded due to continuous rainfall. As a result, this increased the soil and water pressure behind the retaining pile. Thus, the complex multi-field coupling dynamic process caused by partial leakage was unfavorable to the deformation and stability of the foundation pit.

Combined with the monitoring results of supporting axial force during excavation, the supporting axial forces of the north launch shaft are approximately 208-2124 kN at Level 1, 793-1118 kN at Level 2, 257-508 kN at Level 3 and 104-215 kN at Level 4. Additionally, the supporting axial forces of the south launch shaft are approximately 132-672 kN at Level 1, 258-415 kN at Level 2, 236-519 kN at Level 3, and 199-415 kN at Level 4. The ratio of the maximum supporting axial force of the north launch shaft to that of the south launch shaft is 3.2 at Level 1 and 2.7 at Level 2. The supporting axial force of the north launch shaft is significantly higher than that of the south launch shaft. This indicates that the retaining wall of the north launch shaft suffered greater soil and water pressure during excavation, which eventually lead to greater horizontal displacement of the retaining wall of the north launch shaft. Therefore, it was further demonstrated that the sudden increase of soil and water pressure behind the retaining wall caused by foundation pit leakage was an essential factor in the rise of horizontal displacement of the retaining wall.

3.2.2 Variation characteristics of ground settlement behind the retaining wall

Fig. 7 shows the distribution curves of ground settlement behind the wall under different conditions. It can be seen that the ground settlement continues to increase and presents a downward concave trend with the increase of excavation depth. The larger the excavation depth is, the more significant the distribution characteristics of "scoop" grooves are; that is, the deeper the excavation depth of the foundation pit, the deeper the "scoop bottom" (the greater the settlement amount), and the longer the "scoop

handle" (the greater the influence range). The ground settlement trough caused by partial leakage is further deepened and widened, and the maximum ground settlement in the long side direction of the launch shaft is larger than that in the short side direction. When the excavation reaches the bottom of the pit, the maximum ground settlement behind the wall of DS1-*i* (the long side direction of the north launch shaft), DS2-*i* (the short side direction of the north launch shaft), DS3-*i* (the long side direction of the north launch shaft) and DS38-*i* (the long side direction of the south launch shaft, without leakage) are -3.90, -5.10, -9.80 and -4.90mm respectively. When partial leakage occurs, the maximum ground settlement behind the wall of CX-1, CX-2, and CX-3 are -10.40, -5.90, and -16.20mm, respectively, which is about 2.67, 1.16, and 1.65 times of that before leakage. After the base slab-pouring is completed, the maximum ground settlement behind the wall of measuring points CX-1, CX-2, CX-3, and CX-38 are -14.40, -6.70, -17.70, and -7.40 mm, respectively. Compared with the north launch shaft, the maximum ground settlement behind the south launch shaft wall is about 41.81% of that of the north launch shaft. Thus, the ground settlement of the north launch shaft increases significantly after leakage.

3.2.3 Variation characteristics of groundwater level outside the pit

Time history curves of the groundwater level outside the pit and the change rate during excavation are shown in Fig. 8 and Fig. 9, respectively. During the construction of the north launch shaft, the groundwater level out of the pit first decreases with the excavation time. It then gradually stabilizes, and the groundwater level rises rapidly, affected by continuous rainfall. Following foundation pit leakage, the groundwater level outside the pit drops sharply. The characteristics of the change rate of groundwater level show a "jump"; the maximum difference of water head between SW-1 and SW-3 before and after leakage are 0.90, 0.26, and 1.62 m, respectively, and the maximum change rate of water level are -0.90, -0.24, 1.59 m/d, respectively. The groundwater level and its change rate at SW-3 exceeded the alarm value. The

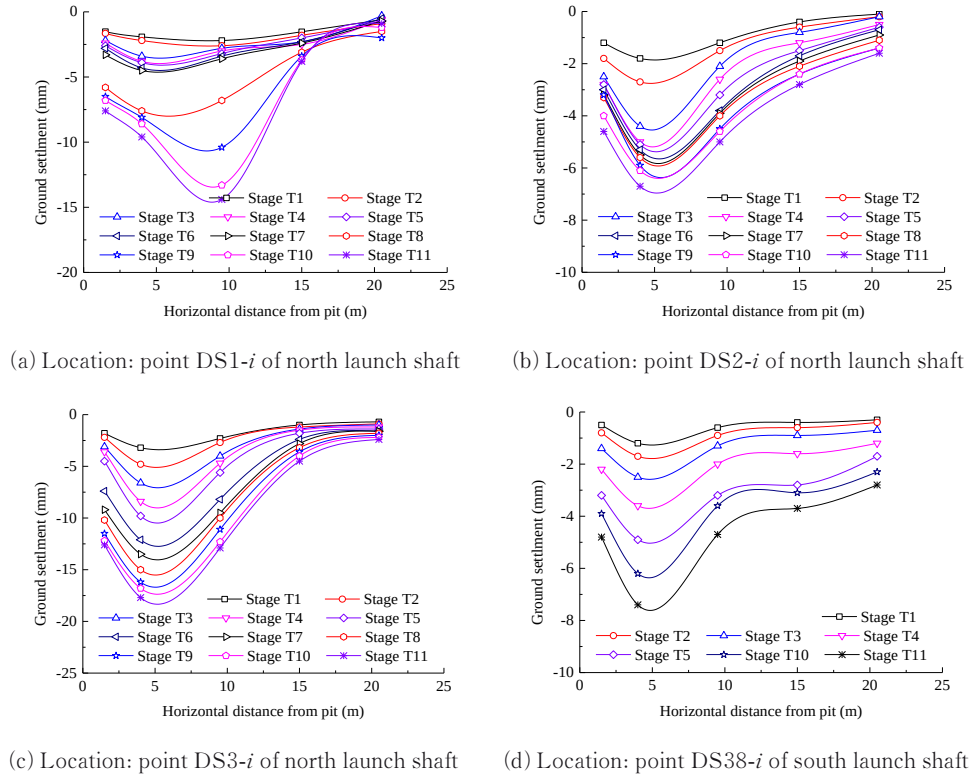


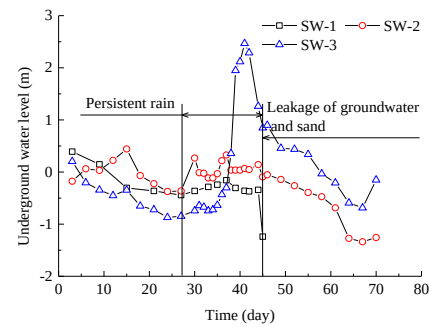
Fig. 7 Distribution curves of ground settlement under different conditions

groundwater level gradually recovered to stability after the reinforcement of partial leakage. Compared with the north launch shaft, the groundwater level change in the south launch shaft with enhanced waterproofing measures is relatively stable. Although the groundwater level outside the pit exceeds the monitoring alarm value of ± 1.0 m, its change rate does not exceed the alarm value of ± 0.5 m/d. Field monitoring showed no large deformation or other signs of instability during the construction of the south launch shaft.

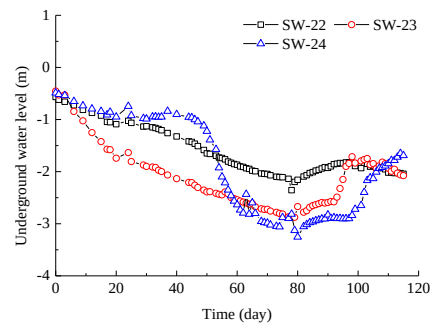
4 Numerical simulation of leakage groundwater of the pit

4.1 The establishment of the finite element model

According to the actual depth of the launch shaft, the excavation sizes are 17.9 m in depth and 23.0 m in width. The numerical analysis model of a deep foundation pit with an impervious curtain, retaining structure, and sand foundation is established by the finite element software PLAXIS2 D considering the seepage effect (Fig. 10). The process of dewatering, excavation, and retaining wall leakage (sand loss was not considered) is simulated based on the practical work. The



(a) North launch shaft



(b) South launch shaft

Fig. 8 Time history curves of underground water level outside the pit

deformation rules of the foundation pit caused by them are analyzed. Considering the influence range of elastic-plastic disturbance of deep excavation, the model width is 3 times the excavation depth. The

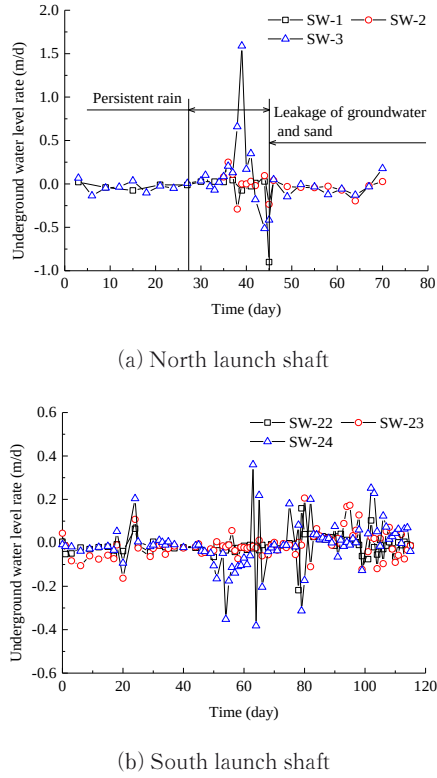


Fig. 9 Time history curves of underground water level outside the pit

size of the established calculation model is 36.0 m in height and 120.0 m in width, and the model is divided into 2745 units and 22227 nodes. The model's boundary is the displacement boundary condition; the surface is a free boundary, the side is restricted to horizontal movement, and the bottom is limited to horizontal and vertical movement. The groundwater level is 5.0 m below the surface, and the ground overload is 20.0 kPa.

The solid element simulation is used to simulate the soil layer in numerical simulation. It is assumed that the thickness of the foundation soil layer is evenly distributed along the horizontal direction, and

the soil is homogeneous and an isotropic material. The hardening soil model (HS model) is adopted to simulate the influence of excavation unloading and soil hardening^[29-31], and this relationship can be described by Formula (1). The HS model has 11 basic parameters, including 3 failure parameters as in the Mohr-Coulomb model (c , φ and ψ), 3 basic parameters for soil stiffness (E_{50}^{ref} , $E_{\text{oed}}^{\text{ref}}$ and m), and 5 advanced parameters ($E_{\text{ur}}^{\text{ref}}$, ν_{ur} , p^{ref} , R_f and K_0). Where c is the cohesion, φ is the internal friction angle, ψ is the dilatancy angle, E_{50}^{ref} is the secant stiffness in standard drained triaxial test, $E_{\text{oed}}^{\text{ref}}$ is the tangent stiffness for primary oedometer loading, m is the power for stress-level dependency of stiffness, $E_{\text{ur}}^{\text{ref}}$ is the reference Young's modulus for unloading and reloading, ν_{ur} is the Poisson's ratio for unloading and reloading, p^{ref} is reference stress for stiffness, R_f is the failure ratio, K_0 is the lateral pressure coefficient for normal consolidation (default $K_0=1-\sin\varphi$). The soil around the foundation pit is in a state of flow slip due to the seepage process. The calculation parameter of the sand soil layer was taken as the index of peak total stress intensity in this calculation^[32-34], and the values of each calculation parameter are shown in Table 1 and Table 2. The interface element simulates the interaction between the soil and the retaining structure, which is reflected by setting the appropriate strength reduction coefficient (R_{int}) of the interface element. The structural elements are used to simulate a retaining pile and internal supporting, the solid elements are used to simulate an impervious curtain, and these materials are subject to a linear elastic constitutive model. The calculated parameters of the established

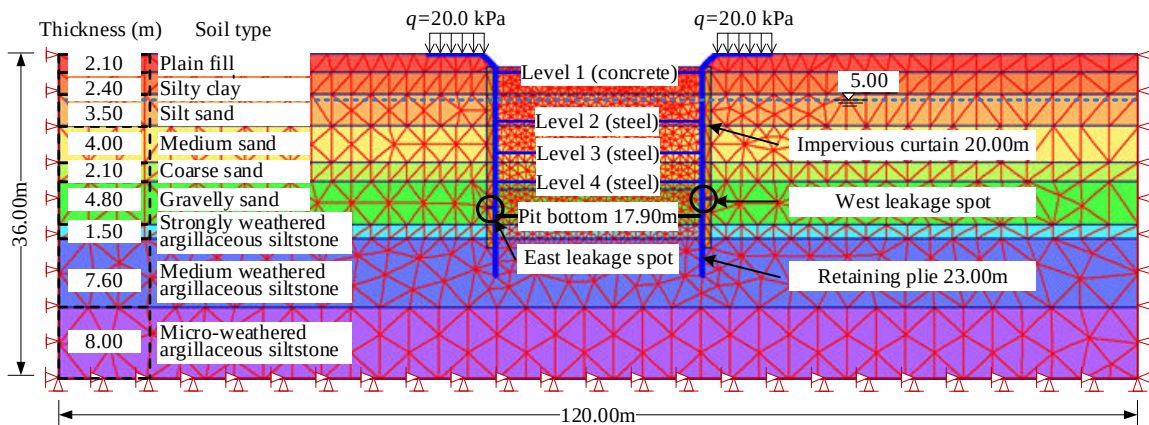


Fig. 10 Finite element model

numerical model are as follows: the weight of the impervious curtain is 20.0 kN/m³, the cohesion is 120.0 kPa, the internal friction angle is 40.0°, the elastic modulus is 0.4 GPa, and the Poisson's ratio is 0.25. The weight of the retaining pile is 25.0 kN/m³, the elastic modulus is 31.5 GPa, and the Poisson's ratio is 0.2. The weight of concrete support is 25.0 kN/m³, the elastic modulus is 30.0 GPa, and the Poisson's ratio is 0.2. The weight of steel support is 78.0 kN/m³, the elastic modulus is 210.0 GPa, and the Poisson's ratio is 0.3.

$$-\epsilon_1 = \frac{1}{2E_{50}} \frac{q}{1 - q/q_a} \quad (1)$$

Where ϵ_1 is the vertical strain, q is the deviatoric stress, q_a is the asymptotic value of the shear strength, and E_{50} is the confining pressure-dependent stiffness modulus for primary loading. It is given by

the equation (2).

$$E_{50} = E_{50}^{\text{ref}} \left(\frac{c \cos \varphi - \sigma'_3 \sin \varphi}{c \cos \varphi + p^{\text{ref}} \sin \varphi} \right)^m \quad (2)$$

Where is σ'_3 is the minor principal stress, p^{ref} is the reference pressure, a default setting $p^{\text{ref}}=100$ kPa is used, c is the cohesion, and φ is the internal friction angle.

The stress-dependent stiffness modulus E_{ur} for unloading and reloading paths can be calculated by equation (3), and the oedometer stiffness modulus E_{od} for one-dimensional compression can be calculated by equation (4).

$$E_{\text{ur}} = E_{\text{ur}}^{\text{ref}} \left(\frac{c \cos \varphi - \sigma'_3 \sin \varphi}{c \cos \varphi + p^{\text{ref}} \sin \varphi} \right)^m \quad (3)$$

$$E_{\text{od}} = E_{\text{od}}^{\text{ref}} \left(\frac{c \cos \varphi - \sigma'_3 \sin \varphi}{c \cos \varphi + p^{\text{ref}} \sin \varphi} \right)^m \quad (4)$$

Table 2 Calculation parameters

Soil layer	$E_{50}^{\text{ref}}/\text{MPa}$	$E_{\text{od}}^{\text{ref}}/\text{MPa}$	$E_{\text{ur}}^{\text{ref}}/\text{MPa}$	ν_{ur}	$p^{\text{ref}}/(\text{kN}/\text{m}^2)$	R_f	m	$\psi/(^{\circ})$	R_{int}
Plain fill	5.0	5.0	25.0	0.30	100	0.9	0.5	0.0	0.65
Silty clay	9.6	9.6	47.8	0.34	100	0.9	0.5	0.0	0.65
Silt sand	11.3	11.3	33.9	0.34	100	0.9	0.5	3.0	0.70
Medium sand	14.4	14.4	43.2	0.31	100	0.9	0.5	3.0	0.70
Coarse sand	15.8	15.8	47.4	0.31	100	0.9	0.5	5.0	0.70
Gravelly sand	19.7	19.7	59.1	0.30	100	0.9	0.5	5.0	0.70

4.2 Simulation of dewatering excavation and leakage process

Considering the influence of dewatering seepage and leakage on the deformation of the retaining structure and surrounding environment during excavation, the finite element analysis adopts staged dewatering and excavation path simulation, and the supporting structure is first applied before the soil excavation in the pit. The steps of simulation were as follows: Stage C0, the retaining pile and impervious curtain were constructed, balancing the initial ground stress, and clearing the initial displacement; Stage C1, the first layer of soil is excavated to -2.0 m, and the first concrete support (Level 1) is installed; Stage C2, the groundwater level is reduced to -8.0 m, the second layer of soil is excavated to -7.5 m, and the second steel support (Level 2) is installed; Stage C3, the groundwater level is reduced to -11.5m, the third layer of soil is excavated to -11.0 m, and the third steel support (Level 3) is installed; Stage C4, the groundwater

level is reduced to -15.5 m, the fourth layer of soil is excavated to -15.0 m, and the fourth steel support (Level 4) is installed; Stage C5, the groundwater level is reduced to -18.4m, and the fifth layer of soil is excavated to the bottom of the pit -17.9 m; Stage C6, on the west side of north shaft, the retaining wall at 1.9 m from the bottom of the pit has a partial leakage, and the size of leakage hole is 0.2 m length and 0.2 m width; Stage C7, on the east side of north shaft, the retaining wall at 0.9 m from the bottom of the pit has a partial leakage, and the size of leakage hole is 0.2 m length and 0.2 m width.

When partial leakage occurs inside the retaining wall, the leakage point provides a seepage channel for the groundwater outside the pit to enter the pit. In this model, the leakage area was preset on the retaining structure, and the leakage area of the retaining wall and impervious curtain were simulated by the drainage line and solid unit, respectively. The partial leakage water of the foundation pit was simulated by opening the drainage line and adjusting

the permeability coefficient of the solid unit (permeable boundary condition). The permeability coefficient of the solid unit for the leakage zone determined by trial calculation is 8.00×10^{-2} cm/s. The distribution of seepage velocity (v_w) field under partial leakage groundwater is shown in Figure 11. It can be seen from Fig. 11 that the difference in groundwater head inside and outside the pit was changed due to the dewatering and excavation. The groundwater outside the pit flows around the permeable soil layer at the bottom of the impervious curtain; the phenomenon of "seepage concentration" appears in the soil around the bottom. The seepage in the soil is mainly horizontal, and the maximum seepage velocity is 1.44×10^{-3} m/d. The seepage velocity in the sandy soil layer is higher than in the silty soil and argillaceous siltstone layers. The impervious curtain's waterproof effect changes the foundation pit's seepage path and then affects the foundation pit seepage field distribution. The partial leakage of the retaining wall causes the seepage effect of soil to increase, and the maximum seepage velocity is 350.3 m/d.

4.3 Calculation results and analysis

4.3.1 Effect of partial leakage on the horizontal displacement of retaining pile

Fig. 12 shows the horizontal displacement contour of surrounding soil under different construction stages. It can be seen from Fig. 13 that after the foundation pit was excavated to the bottom, the soil outside the pit was displaced toward the pit. The horizontal displacement of soil strata between the Level 3 and the Level 4 is the largest (Fig. 12(a)). With the increase of the distance to the edge of the pit, the horizontal displacement of the surrounding strata de-

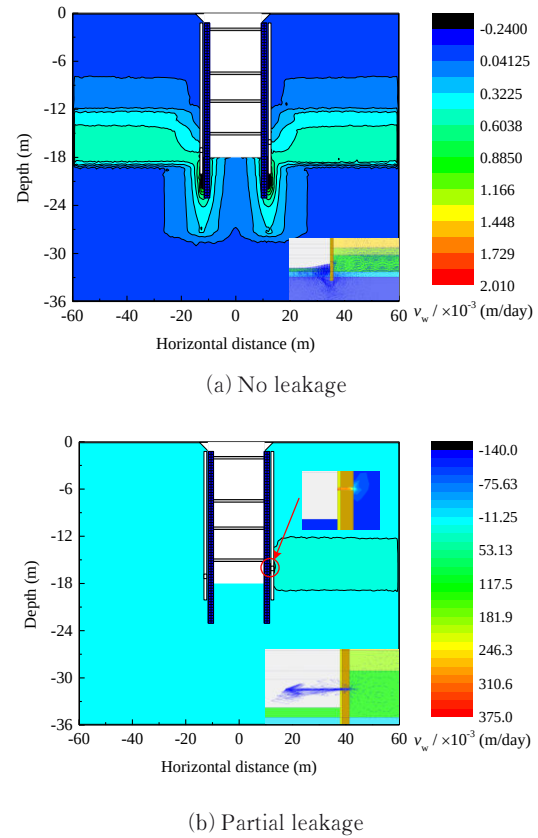


Fig. 11 Comparison between seepage velocity field under partial leakage groundwater

creased, and the horizontal displacement of soil was not significant beyond about $2.0h_c$. When the foundation pit's leakage occurred, the stratum's horizontal displacement and influence range at the leakage side increased significantly compared to before the leakage (Fig. 12(b)). Under the condition that the left and right sides of partial leakage successively, the horizontal displacement distribution of surrounding strata changes from the symmetric distribution before the leakage to the asymmetric distribution and then develops into an approximate symmetric distribution (Fig. 12(c)).

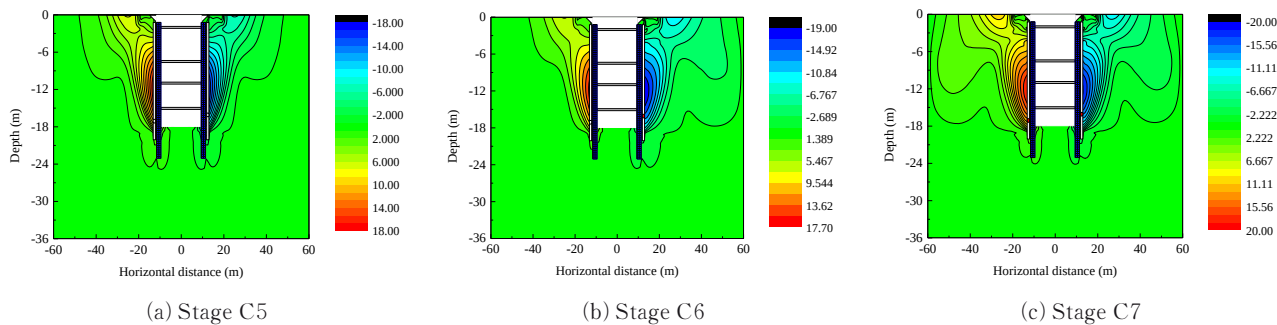


Fig. 12 Horizontal displacement contour of surrounding soil under different construction stages (unit: mm)

Fig. 13 compares the calculated value of horizontal displacement of the retaining pile and the measured value. As can be seen, the development trend of the horizontal displacement curves of the retaining wall is consistent with the measured results. The horizontal displacement curves of the retaining wall also show a progressive development process with the excavation depth, which can be described as form "oblique" shape, "bottom of pot" shape, shallow "inverted V" shape, and then deep "inverted V" shape. The calculated values of maximum horizontal displacement of retaining pile under different construction stages are respectively 0.92, 9.34, 13.92, 16.69, 17.51, and 18.93 mm, and the corresponding measured values are respectively 0.77, 1.97, 5.62, 9.36, 11.92 and 17.16 mm. The calculated values of horizontal displacement of the retaining pile after leakage increased by 8.11%. The calculated and measured values of the maximum horizontal displacement of the retaining pile under partial leakage are respectively $0.11\%h_e$ and $0.10\%h_e$, and the calculated and measured values of the maximum horizontal displacement location of the retaining pile are respectively $0.68h_e$ and $0.73h_e$. Compared with the measured values, the horizontal displacement of the retaining pile after leakage tends to increase, but the calculated values of horizontal displacement of the retaining pile after leakage are small because the impact effect of dynamic water loads on the retaining pile under leakage conditions is not reflected in the numerical calculation.

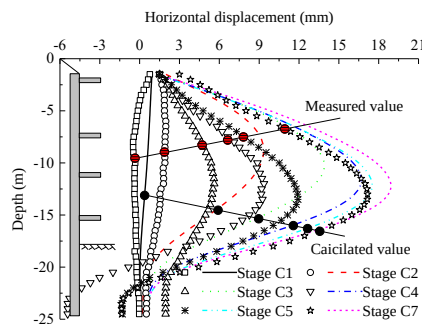


Fig. 13 Comparison of calculated value and measured value of horizontal displacement of retaining pile

4.3.2 Effect of partial leakage on ground settlement behind the retaining wall

Fig. 14 shows the vertical displacement contour of surrounding soil under different construction stag-

es. It can be seen from Fig. 14 that after the excavation unloading of the foundation pit, the soil on both sides of the pit occurs settlement deformation, and the soil at the bottom of the pit occurs uplift deformation. The sensitive zone of ground settlement ranges from $1.00h_e$ to $2.00h_e$, and the location of maximum ground settlement is about $0.42h_e$ (Fig. 14(a)). When the leakage of the right retaining wall occurs, the vertical displacement and its influence range of the stratum around the foundation pit's right side increase. At the same time, the stratum's vertical displacement around the foundation pit's left side does not increase significantly, resulting in an asymmetric distribution of the vertical displacement around the left and right sides of the foundation pit (Fig. 14(b)). When the leakage of the left retaining wall occurs, the stratum's vertical displacement around the foundation pit's left side increases significantly, and the vertical displacement around both sides of the foundation pit presents an approximate symmetrical distribution after two leaks (Fig. 14(c)). The comparisons in Fig. 13 indicated that the influence of leakage on the vertical displacement of the surrounding stratum is much greater than that of the horizontal displacement, resulting in settlement or collapse of the ground after partial leakage. It is suggested that the adverse impact on the adjacent buildings and the surrounding environment should be paid attention to in engineering practice.

Fig. 15 compares the calculated and measured value of ground settlement behind the wall. As can be seen, the ground settlement curves behind the wall successively transformed from a "flat ladle bottom" shape to a "shallow ladle bottom" shape and then to a "deep ladle bottom" shape with excavation depth. According to the distribution characteristics of the ground settlement trough behind the wall, the influence range of ground settlement can be divided into sensitive zone I, transitional zone II, and slightly into zone III. The ground settlement is the largest in the sensitive zone I, ranging from $1.00h_e$ to $1.20h_e$. The calculated values of maximum ground settlement behind the wall under different construction stages are respectively 1.46, -7.56, -11.21, -13.57, -14.36, and -33.17 mm, and the corresponding measured values are respectively -3.20,

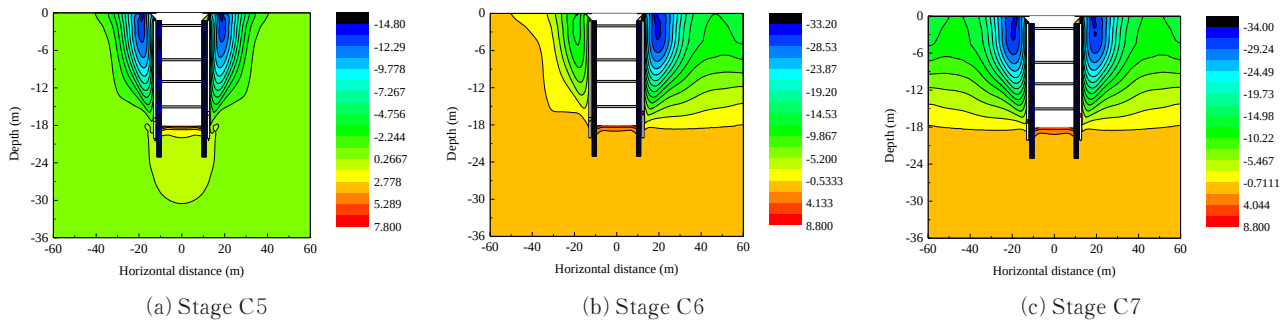


Fig. 14 Vertical displacements contour of surrounding soil under different construction stages (unit: mm):

—4.80, —6.60, —8.40, —9.80 and —16.20 mm. The calculated value of maximum ground settlement after leakage is 2.31 times that before leakage. This shows that foundation pit leakage will cause a significant increase in the ground settlement behind the wall; when partial leakage occurs in the foundation pit, the key is to quickly block the leakage channel and reduce the soil and water loss to prevent the disaster of surrounding strata.

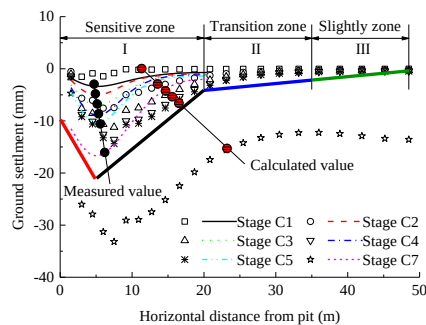


Fig. 15 Comparison of calculated value and the measured value of the ground settlement

5 Conclusion

(1) The foundation pit's leakage disaster shows sudden, instantaneous, and progressive characteristics. Additionally, its development process can be described as a change in the underground hydraulic environment, visible wet spots on the retaining wall surface, partial leakage of the retaining wall, leakage spot expansion, forming through leakage channels, short duration water and sand gushing in pit, soil and water loss in the strata behind the wall, and large deformation of retaining wall, surface collapse and other disasters.

(2) The horizontal displacement curve of the retaining wall successively transformed from an "oblique" shape to a "bottom of pot" shape, a shallow "inverted V" shape, and then to a deep "inverted V"

shape with excavation depth. The horizontal displacement of the retaining wall increases significantly due to partial leakage. Furthermore, the maximum horizontal displacement of the retaining wall after leakage is 1.29 to 1.47 times that before leakage.

(3) The ground settlement curve behind the wall is distributed in a "spoon shape" groove; the ground settlement mainly occurs in the sensitive zone, ranging from $1.00h_e$ to $2.00h_e$. The ground settlement trough deepens and widens due to partial leakage, and the maximum ground settlement after leakage is 1.16 to 2.67 times that before leakage. The influence of water leakage on the vertical displacement around the foundation pit is much greater than that of the horizontal displacement.

(4) The characteristics of "jump" fluctuation of groundwater level can be used as an essential criterion for the precursor of leakage disaster in foundation pit. It is suggested that attaching importance to and giving full play to the feedback and prediction function of construction monitoring is conducive to preventing and controlling leakage disasters for deep foundation pits in water-rich sandy strata. Importantly, this example proves that the combined treatment measures of grouting and high-pressure jet spouting piles can effectively deal with the problem of leakage disaster control of foundation pits in water-rich sandy strata.

References

- [1] ZHENG G, DAI X, DIAO Y, et al. Experimental and simplified model study of the development of ground settlement under hazards induced by loss of groundwater and sand [J]. *Natural Hazards*, 2016, 82(3): 1869-1893.
- [2] ZHENG G, CHENG X S, ZHOU H Z, et al. Resilient evaluation and control in geotechnical and underground engineering [J]. *China Civil Engineering Journal*, 2022,

- 55(7): 1-38. (in Chinese)
- [3] XIANG Z F, XU J M. Deformation characteristics of deep foundation pit with suspended waterproof curtain during excavation [J]. *Hydrogeology and Engineering Geology*, 2023, 50(5): 96-106. (in Chinese)
- [4] SUN Q, LI H E, QIN S Q, et al. Analysis of foundation pit damage caused by groundwater [J]. *Chinese Journal of Geotechnical Engineering*, 2006, 28 (Sup1): 1428-1432. (in Chinese)
- [5] ZHENG G, ZHU H H, LIU X R, et al. Control of safety of deep excavations and underground engineering and its impact on surrounding environment [J]. *China Civil Engineering Journal*, 2016, 49(6): 1-24. (in Chinese)
- [6] WANG J X, LIU X T, LIU J X, et al. Dewatering of a 32.55 m deep foundation pit in MAMA under leakage risk conditions [J]. *KSCE Journal of Civil Engineering*, 2018, 22(8): 2784-2801.
- [7] LIU J C, TAN Y, SONG X H, et al. Effects of through-wall leaking during excavation in water-rich sand on lateral wall deflections and surrounding environment [J]. *Journal of Zhejiang University (Engineering Science)*, 2023, 57(3): 530-541. (in Chinese)
- [8] NI J C, CHENG W C. Characterising the failure pattern of a station box of Taipei Rapid Transit System (TRTS) and its rehabilitation [J]. *Tunnelling and Underground Space Technology*, 2012, 32: 260-272.
- [9] KORFF M, MAIR R J, VAN TOL A F, et al. Building damage and repair due to leakage in a deep excavation [J]. *Proceedings of the Institution of Civil Engineers - Forensic Engineering*, 2011, 164(4): 165-177.
- [10] WU C S, GONG L, SHEN W D. Monitoring and treatment during the water leakage process of a peep subway station pit [C]//*International Conference on Civil Engineering and Transportation (ICCET 2011)*, Jinan, P. R. China, Oct 14-16, 2011. *Applied Mechanics and Materials*, 2011, 94-96: 1951-1956.
- [11] JO Y S, CHO S H, JANG Y S. Field investigation and analysis of ground sinking development in a metropolitan city, Seoul, Korea [J]. *Environmental Earth Sciences*, 2016, 75(20): 1353.
- [12] LIU J C, TAN Y. Review of through-wall leaking incidents during excavation of the subway stations of Nantong metro line 1 in thick water-rich sandy strata [J]. *Tunnelling and Underground Space Technology*, 2023, 135: 105056.
- [13] FAN D D, LIU X Y, JING X C, et al. Causes analysis on wall leaking of metro foundation pit in Nantong water-rich sandy stratum [J]. *Tunnel Construction*, 2020, 40 (Sup1): 225-231. (in Chinese)
- [14] JIANG F P, LIU G B. Surface subsidence caused by water seeping through diaphragm wall of deep foundation pit [J]. *Chinese Journal of Geotechnical Engineering*, 2010, 32(Sup2): 574-577. (in Chinese)
- [15] ZHANG J, YE S. Leakage risk identification of foundation pit retaining structure based on measured data [J]. *Chinese Journal of Geotechnical Engineering*, 2008, 30(Sup1): 667-671. (in Chinese)
- [16] TAN Y, LU Y. Forensic diagnosis of a leaking accident during excavation [J]. *Journal of Performance of Constructed Facilities*, 2017, 31(5): 04017061.
- [17] ZHENG G, DAI X, ZHANG X S. Experimental study and numerical simulation of leaking process of sand and water in underground engineering [J]. *Chinese Journal of Rock Mechanics and Engineering*, 2014, 33(12): 2458-2471. (in Chinese)
- [18] LI B, YU Z Q, LI M, et al. Centrifugal model test on influence of leakage on foundation pit diaphragm wall in the first-level terrace of Yangtze River in Wuhan [J]. *Journal of Yangtze River Scientific Research Institute*, 2020, 37(3): 78-81, 89. (in Chinese)
- [19] CAO W P, XI M Y, ZHAO C, et al. Model test of influence of local failure on internal braced row pile support [J]. *Journal of Water Resources and Water Engineering*, 2023, 34(5): 190-197. (in Chinese)
- [20] HU Q, LING D S, CHEN R P, et al. Research on deep foundation pit excavation due to seepage failure of soil [J]. *Rock and Soil Mechanics*, 2008, 29(11): 2967-2972. (in Chinese)
- [21] KHOSRAVI M, KHOSRAVI M H, GHOREISHI NAJAFABADI S H. Determining the portion of dewatering-induced settlement in excavation pit projects [J]. *International Journal of Geotechnical Engineering*, 2021, 15(5): 563-573.
- [22] CLIMENT N, ARROYO M, O'SULLIVAN C, et al. Sand production simulation coupling DEM with CFD [J]. *European Journal of Environmental and Civil Engineering*, 2014, 18(9): 983-1008.
- [23] DAI X, ZHENG G, CHENG X S, et al. Numerical simulation of ground movement induced by leakage of groundwater and sand in excavations based on the DEM-CFD method [J]. *Chinese Journal of Rock Mechanics and Engineering*, 2019, 38(2): 396-408. (in Chinese)
- [24] WU Y X, SHEN S L, LYU H M, et al. Analyses of leakage effect of waterproof curtain during excavation dewatering [J]. *Journal of Hydrology*, 2020, 583: 124582.
- [25] WU Y X, LYU H M, SHEN S L, et al. A three-dimensional fluid-solid coupled numerical modeling of the barrier leakage below the excavation surface due to dewatering [J]. *Hydrogeology Journal*, 2020, 28(4):

- 1449-1463.
- [26] CHAO H, TAN Y, LIU T R. Meso-mechanism of ground collapse induced by leaking of underground diaphragm wall in water-rich sandy strata [J]. Tunnel Construction, 2023, 43(7): 1180-1189. (in Chinese)
- [27] ZHANG K Y, ZHANG M, SUN B, et al. A calculation method for deformation of diaphragm wall of narrow deep foundation pit in soft soil considering spatio-temporal effect [J]. Rock and Soil Mechanics, 2023, 44(8): 2389-2399. (in Chinese)
- [28] CHENG K, XU R Q, YING H W, et al. Observed performance of a 30.2 m deep-large basement excavation in Hangzhou soft clay [J]. Tunnelling and Underground Space Technology, 2021, 111: 103872.
- [29] FINNO R J, CALVELLO M. Supported excavations: Observational method and inverse modeling [J]. Journal of Geotechnical and Geoenvironmental Engineering, 2005, 131(7): 826-836.
- [30] WANG W D, WANG H R, XU Z H. Experimental study of parameters of hardening soil model for numerical analysis of excavations of foundation pits [J]. Rock and Soil Mechanics, 2012, 33(8): 2283-2290. (in Chinese)
- [31] ZHU N, ZHOU Y, LIU W, et al. Study of silty soil behavior disturbed for installation of diaphragm wall in Suzhou [J]. Rock and Soil Mechanics, 2018, 39(Sup1): 529-536. (in Chinese)
- [32] ZHANG J S, PENG Z P, ZHOU J X. Flow-sliding property of sand deposit in the cofferdam of the Three Gorges Project and the determination of its shear strength parameters [J]. Journal of Hydraulic Engineering, 1998(6):14-18. (in Chinese)
- [33] POULOS S J. The steady state of deformation [J]. Journal of the Geotechnical Engineering Division, 1981, 107(5): 553-562.
- [34] ZHU J Q, KONG L W, GAO W H, et al. Steady-state properties of Nanjing sand [J]. Chinese Journal of Geotechnical Engineering, 2012, 34(5): 931-935. (in Chinese)
- (编辑 XXX)