In-Situ Monitoring and Numerical Analysis of Deformation in Deep Foundation Pit Support: A Case Study in Taizhou

Junding Liu 1, Jiangnan Ye 2, Xinyu Shen 1, Jing Yu 1, Tao Wu 3, Ji Yuan 1, Qian Ye 1,*, Shifang Wang 1,*, and Haijie He 1,4,*

Abstract: With the increasing prevalence of deep foundation pit projects in modern urban construction, the deformation control of the associated support structures is crucial to ensuring construction safety. Despite the growing importance of this issue, research on deep foundation pit excavation in soft soil areas near the sea is limited. This study investigates the deformation characteristics and impact of a geometrically complex deep foundation pit project in Taizhou, utilizing both long-term in-situ monitoring and numerical simulation. The results of this study indicated that the maximum horizontal displacement of the foundation pit was approximately within the depth range of 5 m, with a value of 34.5 mm. The maximum settlement of both the top of the pit and the surrounding structures were determined to be less than 25 mm, confirming the safety of the excavation process. The excavation process of the foundation pit was simulated and analyzed by the Midas GTS finite element software. The results showed a similar variation pattern to the measured results, but the numerical results were generally lower than the measured values. This study provides valuable insights into the deformation characteristics of deep foundation pit excavation in soft soil areas and serves as a reference for future projects in similar conditions.

Keywords: foundation pit; deformation law; numerical simulation; construction monitoring

1. Introduction

With the rapid pace of urbanization construction in China, there is an increasing demand for ensuring the safety of underground projects such as high-rise buildings and subways [1]. The construction of deep foundation pits requires careful deformation management and an adequate bearing capacity of the foundation pit support system to avoid excessive deformation of the support structure and surrounding soil layer. Failure to do so could result in the deformation exceeding the acceptable limits [2]. Thus, it would affect the normal use of the neighboring buildings and may cause the foundation pit’s destabilization and collapse, resulting in huge economic losses. Therefore, it is crucial to ensure the safety of the construction process and the normal operation of neighboring buildings by controlling the deformation of deep foundation pit support systems in constructing deep foundation pits.

Many researchers have employed various methods, including empirical methods, geotechnical centrifugal simulation tests, in-situ monitoring, and numerical simulations, to study the deformation characteristics of deep foundation pits [3–5]. The empirical method involves the analysis of data from numerous pit projects to derive the deformation law of deep foundation pits. Xiao et al. [6] analyzed data from 92 foundation pit excavations in soft
soil areas in China, and found that the sidewall deformation and surface settlement of the pit varied regularly as the pit width increased. Chen et al. [7] conducted a series of centrifuge model tests to examine the interaction between foundation pits constructed successively. The results indicated that the stress and deformation of the foundation pit support structure that was built earlier were greater than those of the later-constructed foundation pit, and that the foundation pit with higher protection requirements should be constructed last. Liu et al. [8] conducted on-site measurement of the deformation of support structure during the construction process of foundation pit engineering, and obtained empirical estimation methods for the deformation of support structures and surface settlement. Wang et al. [9] conducted field tests to study the deformation characteristics of deep foundation pits in high water table areas of saturated soft loess soils, and revealed the deformation law of foundation settlement, enclosure structure, and support axial force. They found that reasonable axial force was an effective method to control the deformation of diaphragm walls and the axial force of steel support. In-situ monitoring tests on composite structural foundation pits in soft soil areas revealed that the excavation of pits had significant temporal and spatial effects on surrounding buildings [10,11]. Zhao et al. [12] employed Midas GTS software to perform finite element modeling in order to assess the effects of excavation at different sections of pits on tunnel deformation. The results of the numerical simulation were compared with monitoring data to verify the accuracy of the calculations. Numerical simulation is a valuable tool for evaluating the impact of various pit excavation methods and support parameters on pit deformation and provides a basis for the design and construction of foundation pits. With the rapid advancement of computer technology, the time required for large-scale finite element numerical calculations has been significantly reduced, making numerical simulation methods increasingly popular in foundation pit engineering [13]. Yang G. [14] conducted theoretical research on the stress and deformation characteristics of support systems based on linear elasticity theory.

In constructing deep foundation pits in complex environments, the primary control conditions of foundation pit engineering have shifted from strength control to deformation control. The deformation of the foundation pit encompasses the surface settlement, horizontal and vertical displacements of the enclosure structure, and uplift of the bottom of the foundation pit. These deformations are influenced by the stress release, groundwater seepage, and soil rheology that occur after excavation of the foundation pit [15,16]. Due to the coupling relationship between the displacement of the pit enclosure, the surface settlement, and the pit bottom uplift, conventional research methods are limited in reflecting the influence of multiple factors. Therefore, the analysis of pit deformation is mostly conducted through numerical simulation [17]. Lin et al. [18] utilized the Midas GTS finite element software to establish a three-dimensional spatial model to simulate the excavation process of a deep foundation pit. The results indicated that the calculated values of pile deformation, pile top displacement, and axial force of internal support during construction were in agreement with the observed values. In a separate study, Zhao et al. [12] evaluated the impact of excavation in different sections of deep foundation pits on the deformation of adjacent subway tunnels using finite element models. The results indicated the accuracy of the finite element calculations. Yang et al. [19] found, through numerical simulation, that considering the spatial effect during the excavation of ultra-long foundation pits can effectively reduce the deformation of the support structure and the surrounding strata. The numerical simulation method can better simulate the foundation pit excavation process under the premise of reasonable parameter selection. As such, it is important to conduct real-time construction monitoring of the foundation pit excavation process and obtain relevant parameters. Yu J. and Gong X. [20] and Liu Ji. and Zeng Y. et al. [21] used software such as a self-developed three-dimensional finite element analysis program and FLAC3D to analyze the stress deformation characteristics of the support structure during the excavation process of foundation pits.

Most prior studies have focused on foundation pits with straightforward geometries and single excavation events, and there is limited research on saturated soft soil in the
Taizhou area near the ocean. However, the coastal areas in China are mainly saturated soft soil, which limits local engineering construction and development. Therefore, studying the excavation of foundation pits in saturated soft soil areas will be beneficial for proposing reasonable construction suggestions for the area, which can effectively alleviate the limitations of saturated soft soil on engineering construction. This study focuses on a geometrically complex deep foundation pit project located in Taizhou, China, characterized by its large span, irregular shape, and extremely deep muddy clay layer. Long-term in-situ monitoring and numerical simulation were implemented to study the deformation characteristics during the excavation of the deep foundation pit, and the horizontal and vertical displacements of the enclosure wall during the excavation of the pit, as well as its impact on surrounding buildings, were analyzed. The design of the foundation pit support structure was verified and evaluated, with the goal of offering guidance and recommendations for the design of comparable foundation pit support systems in deep foundation pit construction projects in Taizhou.

2. Project Background
2.1. Construction Site

The Engineering Procurement Construction (EPC) of the Jiaojiang Xinhai Urban Shared Space Project is located in the core area of trade and commerce in Jiaojiang District, Taizhou City. It is bounded by Modern Avenue to the north, Central Avenue to the east at a distance of approximately 100 m, Hongjiachangpu to the south at a distance of approximately 200 m, and is relatively open to the west. The Hongjiachangpu River, which is approximately from 50 to 80 m wide, is located to the south of the site and has a water surface elevation of 2.8 m and a water depth of 1.5 to 2.0 m. The geomorphological unit of the site is a marine plain with convenient transportation. The west side of the site was formerly occupied by a plant which has been demolished, and the original building foundation has been excavated. The rest of the site is characterized by arable land. On the east side, a significant amount of construction waste has been accumulated, and the terrain is generally flat with slight local elevations.

The total planned land area of the Jiaojiang Xinhai Urban Shared Space Project is approximately 79,022 m², and the total construction area is approximately 118,533 m². The above-ground part is divided into five plots, which are primarily composed of a number of two- to three-story commercial buildings with category C seismic protection. With the exceptions of Plot II (buildings 2-1# and 2-3#), Plot IV, and Plot V, the entire site features an integral basement with a basement floor elevation of -0.25 m. Table 1 provides a basic overview of the proposed building structure.

<table>
<thead>
<tr>
<th>Plot</th>
<th>Building</th>
<th>Story</th>
<th>Building Height (m)</th>
<th>Building Ground Elevation (m)</th>
<th>Outdoor Ground Elevation (m)</th>
<th>Structure</th>
<th>Column Load (kN)</th>
<th>Proposed Foundation</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>1-1#–1-5#</td>
<td>2F</td>
<td>9.9</td>
<td></td>
<td></td>
<td></td>
<td>5000</td>
<td>Pile foundation</td>
</tr>
<tr>
<td>II</td>
<td>2-1#, 2-4#, and 2-5#</td>
<td>2F</td>
<td>11.9</td>
<td></td>
<td></td>
<td></td>
<td>5000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2-1#, 2-3#</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3000</td>
<td></td>
</tr>
<tr>
<td>III</td>
<td>3-1#–3-5#</td>
<td>2F</td>
<td>11.6</td>
<td></td>
<td></td>
<td></td>
<td>5000</td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>4-1#</td>
<td>2F</td>
<td>10.87</td>
<td></td>
<td></td>
<td></td>
<td>3000</td>
<td></td>
</tr>
<tr>
<td>V</td>
<td>5-1#, 5-3#</td>
<td>1–3F</td>
<td>7.15–11.9</td>
<td></td>
<td></td>
<td></td>
<td>5000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5-2#</td>
<td>1–3F</td>
<td>7.15–11.9</td>
<td></td>
<td></td>
<td></td>
<td>3000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Basement</td>
<td>−1F</td>
<td>/</td>
<td>/</td>
<td>/</td>
<td></td>
<td>2100</td>
<td></td>
</tr>
</tbody>
</table>

Table 1. Basic overview of the proposed construction list.
The underground part of the Jiaojiang Xinhai Urban Shared Space Project features a pile foundation structure with an excavation size of 110 m by 700 m and a depth ranging from 3.8 to 6.05 m. The excavation constitutes a large-span deep foundation pit and employs the zoned excavation method supported by bored piles. The primary support system of the pit consists of a combination of bored piles and a water curtain of cement mixing piles.

2.2. Geological and Hydrogeological Data

The proposed regional site is located in the coastal area of southeastern Zhejiang Province, which is characterized by a mid-subtropical maritime monsoon climate. The climate is mild, with four distinct seasons, abundant rainfall, ample sunshine, and a lack of severe cold in winter or extreme heat in summer. According to meteorological records, the average annual temperature in the region is 17 °C, with an extreme maximum temperature of 38.1 °C and an extreme minimum temperature of −6.8 °C. Precipitation is irregularly distributed both between and within years, with a concentration of rainfall during the plum rain season from April to June and the typhoon season from July to September. The multi-year average precipitation is 1558.47 mm, with a maximum annual precipitation of 2375.1 mm and a minimum annual precipitation of 912.8 mm.

The area belongs to the Huangyan–Xiangshan Fault Zone of the Southeastern Zhejiang Uplift of the South China Fold System, which is dominated by fracture structures which spread mostly in the north-northeast and north–east direction. The base comprises Late Paleozoic strata of light metamorphic rocks, and the upper part consists of Mesozoic volcanic rocks with significant thickness. There is no regional large fracture passing through, the fold structure is not developed, and the north–east-oriented Taishan–Huangyan Large Fracture and Wenzhou–Zhenhai Large Fracture pass through from the west side of the site. The thickness of the Quaternary strata at the project site is large, and the regional fractures are far away. There is no active fault passing through the site and its vicinity, and the regional geological structure has no influence on the project.

The site’s geomorphological type is a Quaternary Marine Plain landform with river ponds located in the northeast corner and an original ditch connected to an external river. Most of the ground sections are composed of construction waste from demolition and recent artificial landfill residue, and the topography is slightly undulating. The soil layer in the depth range of 94 m below the ground level of the site can be divided into eight engineering geological layers. Table 2 presents the physical and mechanical parameters of each soil layer. By conducting triaxial tests on soil, the cohesion and internal friction angle can be obtained. Through compression tests, the compressive modulus can be obtained. The parameters in this paper come from a water content test, density test, specific gravity test, consolidation test, and triaxial compression test, in the “Standard for geotechnical testing method”.

<table>
<thead>
<tr>
<th>Geological Layer</th>
<th>Layer Thickness (m)</th>
<th>Unit Weight γ(kN m(^{-3}))</th>
<th>Cohesion c (kPa)</th>
<th>Internal Friction Angle φ (°)</th>
<th>Compressive Modulus (E_s) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>0.6–5.8</td>
<td>18.3</td>
<td>27.5</td>
<td>13.6</td>
<td>3.3</td>
</tr>
<tr>
<td>Muddy clay</td>
<td>14.5–30.4</td>
<td>16.5</td>
<td>10.0</td>
<td>8.0</td>
<td>2.1</td>
</tr>
<tr>
<td>Clay</td>
<td>0.0–7.9</td>
<td>16.8</td>
<td>20.2</td>
<td>11.6</td>
<td>2.5</td>
</tr>
<tr>
<td>Clay</td>
<td>3.0–17.2</td>
<td>18.2</td>
<td>27.5</td>
<td>15.9</td>
<td>4.0</td>
</tr>
<tr>
<td>Silt</td>
<td>1.7–20.0</td>
<td>18.0</td>
<td>27.6</td>
<td>14.0</td>
<td>3.6</td>
</tr>
<tr>
<td>Silty clay</td>
<td>0.0–20.4</td>
<td>18.5</td>
<td>31.9</td>
<td>15.8</td>
<td>5.5</td>
</tr>
<tr>
<td>Silty clay</td>
<td>0.0–4.1</td>
<td>19.5</td>
<td>37.0</td>
<td>18.5</td>
<td>7.25</td>
</tr>
<tr>
<td>Fine sand</td>
<td>Unpenetrated</td>
<td>19.5</td>
<td>-</td>
<td>-</td>
<td>10.5</td>
</tr>
</tbody>
</table>

The groundwater at the site occurs mainly as pore phreatic water and confined water. The pore phreatic water is mainly stored in the upper clay layer, and the groundwater
recharge is primarily from atmospheric precipitation and surface water runoff. Runoff and evaporation are the primary modes of discharge, and the water table depth ranges from 0.20 to 6.30 m. The annual dynamic change in water level is about 1.50 m. The pore confined water is mainly distributed in the lower fine sand layer, and the burial depth of the confined water table is about 15.0 m according to regional information, with an annual dynamic change of about 1.00 m. As the groundwater depth of the proposed site is shallow, the water in the pit during excavation is directly drained by digging a drainage trench and using water collection wells (pits).

2.3. Design of the Support Structure and Excavation

The miscellaneous fill on the site surface is unsuitable for use due to its poor quality. The silty clay layer below it has average mechanical properties, but it is thin and can only be used as a natural foundation support layer for light constructions (structures). Beneath this layer lies a thick layer of highly compressible muddy clay, which means that the natural foundation of the site is shallow and the basic conditions are poor.

According to the engineering geological conditions and the characteristics of the proposed building, the natural foundation conditions of the proposed site were deemed poor. The entire site, except for Plot II (buildings 2-1# and 2-3#), Plot IV, and Plot V, features an integral basement. Excavation of the basement pit resulted in the bottom of the pit mostly being located on the muddy clay, and partially on the silty clay. The section of the foundation pit support structure is shown in Figure 1. In consideration of the large loads of the proposed building and poor natural foundation conditions, the pile foundation was deemed more suitable than the shallow foundation. The shallow surface layer is composed of miscellaneous fill, silty clay, muddy clay, and silty clay, in that order. The top beam and horizontal internal support were applied at a depth of 0.6 m from the ground surface, and vertical support was added at the node of horizontal internal support.

Figure 1. Sectional drawing of the foundation pit support structure (unit: m).

Prior to excavation, the site was leveled to ensure that the site elevation was not higher than the design elevation. Subsequently, bored piles were installed along the contour line of the pit to create an underground diaphragm wall. Although the depth of excavation for this project is shallow, the length of the pit is approximately 700 m. Therefore, for the safety of construction, the excavation method was divided into zones, mainly from the two ends...
of the pit to the middle of the pit, as shown in Figure 2. The pit was excavated in six layers along the depth direction, and the specific excavation steps are shown in Table 3.

![Figure 2. Schematic diagram of foundation pits using zoned excavation.]

<table>
<thead>
<tr>
<th>Step</th>
<th>Construction Stage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Level the site, and the ground elevation is 3.6 m</td>
</tr>
<tr>
<td>2</td>
<td>Bored piles are installed along the contour line of the pit to create an underground diaphragm wall</td>
</tr>
<tr>
<td>3</td>
<td>Excavate from both ends of the foundation pit to the middle of the foundation pit at a depth of 0.7 m, and the top beam, horizontal support column, and horizontal support are applied</td>
</tr>
<tr>
<td>4</td>
<td>Excavate from both ends of the foundation pit to the middle of the foundation pit at a depth of 2.2 m</td>
</tr>
<tr>
<td>5</td>
<td>Excavate from both ends of the foundation pit to the middle of the foundation pit at a depth of 2.8 m</td>
</tr>
<tr>
<td>6</td>
<td>Excavate from both ends of the foundation pit to the middle of the foundation pit at a depth of 3.8 m</td>
</tr>
<tr>
<td>7</td>
<td>Stop the excavation on the right side of the foundation pit, and excavate the left side of the foundation pit at a depth of 5.4 m</td>
</tr>
<tr>
<td>8</td>
<td>Area ② and area ③ on the left side of the foundation pit are excavated at a depth of 6.05 m</td>
</tr>
</tbody>
</table>

3. In-Situ Monitoring

3.1. Monitoring Scheme

The excavation for the foundation pit spanned 700 m in length and 110 m in width. The maximum excavation depth on the left side was 6.05 m, while on the right side it was 3.8 m. A diaphragm wall was formed using bored piles along the perimeter of the foundation pit. The left side of the pit was supported by horizontal cross bracing, while the right side was partially supported by corner bracing. A total of 80 monitoring points were set up around the foundation pit, with a spacing of approximately 20 m between each point.

In order to ensure the safety of the foundation pit project, surrounding buildings, and underground pipelines, and to guide the construction, the project monitored various items such as the underground diaphragm walls, surrounding ground surface, buildings, and supporting axial force from the beginning to the completion of construction. The datum point is 50 m outside the foundation pit, and high-precision total station is used to measure the observation point. The project monitored the horizontal displacement of the enclosure wall, as well as the horizontal and vertical displacements of the top of the enclosure wall and the settlement of buildings around the foundation pit at each monitoring point.

In this paper, the measured results of monitoring points W1, W2, W3, W4, and W5 around the foundation pit were selected for study, and the layout of the monitoring points is illustrated in Figure 3. The control values for monitoring parameters such as the horizontal
displacement of the enclosure wall, horizontal displacement of the top of the enclosure wall, vertical displacement of the top of the enclosure wall, and settlement of the surrounding buildings were set at 30 mm to ensure safety during construction.

Figure 3. Layout of monitoring points (unit: m).

3.2. Monitoring Results

3.2.1. Horizontal Displacement of the Enclosure Wall along Its Depth

The horizontal displacement of the enclosure wall along its depth direction was evaluated at the monitoring points W1, W2, W3, W4, and W5. These points were located at depths ranging from 1 m to 22 m, as depicted in Figure 4. The construction of the pit followed the procedure outlined in Table 3, and the first monitoring data were recorded as C0 curve when the excavation depth reached 0.7 m in Step 3. The second data monitoring was performed and recorded as C1 curve at the end of Step 3. The third data monitoring was performed and recorded as C2 curve at the end of Step 4, and so on until the seventh data monitoring was performed and recorded as C6 curve at the end of Step 8. Ten days after the completion of Step 8, the eighth data monitoring was conducted for monitoring points W1, W2, and W4, recorded as C7 curve. The maximum horizontal displacements of the monitoring points W1, W2, W3, W4, and W5 were 27.19 mm, 31.64 mm, 34.16 mm, 34.5 mm, and 19.76 mm, respectively.

Figure 4. Cont.
Given the shallow depth of the pit, the maximum horizontal displacements of the monitoring points were all relatively close to the ground surface. Monitoring points W2, W3, and W4 were located in the middle part of the pit edge and had less cross bracing, resulting in larger horizontal displacements. Monitoring point W1 benefited from more cross bracing, leading to a slightly smaller horizontal displacement. Monitoring point W5, situated at the corner of the pit, had a surplus of cross bracing, explaining the smallest horizontal displacement.

3.2.2. Horizontal Displacement of the TOP of the Enclosure Wall

During foundation pit construction, as the soil pressure in the pit is relieved, the soil pressure behind the enclosure wall will cause lateral displacement. The value of the horizontal displacement of the top of the enclosure wall reflects the magnitude of the soil pressure behind the wall, and the change in this value reflects the deformation of the pit, which is related to the stability of the pit. Therefore, the horizontal displacement of the top of the enclosure wall is an important indicator for measuring the safety of the pit support structure. The variation curves of the horizontal displacement of the top of the enclosure wall at the monitoring points W1, W2, W3, W4, and W5, with the same time interval of observations, were recorded as AW1, AW2, AW3, AW4, and AW5 curves, respectively. The observation time was consistent with Figure 4. Monitoring point W2 was also monitored for the ninth time at an interval of 10 days after the eighth monitoring, as shown in Figure 5.

Figure 4. Horizontal displacement curves of piles under different working conditions: (a) W1, (b) W2, (c) W3, (d) W4, and (e) W5.

Figure 5. Variation curves of horizontal displacement of the top of the enclosure wall and the number of observations.
horizontal displacement of the top of the enclosure wall reflects the magnitude of the soil pressure behind the wall, and the change in this value reflects the deformation of the pit, which is related to the stability of the pit. Therefore, the horizontal displacement of the top of the enclosure wall is an important indicator for measuring the safety of the pit support structure. The variation curves of the horizontal displacement of the top of the enclosure wall at the monitoring points W1, W2, W3, W4, and W5, with the same time interval of observations, were recorded as AW1, AW2, AW3, AW4, and AW5 curves, respectively. The observation time was consistent with Figure 4. Monitoring point W2 was also monitored for the ninth time at an interval of 10 days after the eighth monitoring, as shown in Figure 5.

**Figure 5.** Variation curves of horizontal displacement of the top of the enclosure wall and the number of observations.

The results presented in Figure 5 show a significant increase in the horizontal displacement of the top of the enclosure wall during the excavation process. However, after the completion of excavation, the horizontal cross bracing at each observation point effectively suppressed the horizontal deformation of the foundation pit, leading to a smooth and stable change in the lateral displacement of the top of the enclosure wall. The maximum horizontal displacements of the top of the enclosure wall at the monitoring points W1, W2, W3, W4, and W5 were 24 mm, 28 mm, 28 mm, 27 mm, and 24 mm, respectively, with the displacement of the monitoring points W2, W3, and W4 being the largest. The results suggest that the horizontal displacement of soil around the foundation pit increased, but the distribution was not uniform, with the highest displacement occurring in the middle of the foundation pit.

### 3.2.3. Vertical Displacement of the Top of the Enclosure Wall

The variation curves of the vertical displacement of the top of the enclosure wall at the monitoring points W1, W2, W3, W4, and W5, with the same time interval of observations, were recorded as AW1, AW2, AW3, AW4, and AW5 curves, respectively. The observation time was consistent with Figure 4, as shown in Figure 6. As the figure shows, the vertical displacement of the top of the enclosure wall increased significantly during the excavation process, and finally tended to be basically stable. The maximum vertical displacements of the top of the enclosure wall at the monitoring points W1, W2, W3, W4, and W5 were 15 mm, 20 mm, 24 mm, 20 mm, and 18 mm, respectively, with the displacement of monitoring point W3 being the largest. The results indicated that the vertical displacement of soil around the foundation pit increased, but the distribution was not uniform, with the highest displacement occurring in the middle of the foundation pit. Hence, vertical displacement of the top of the enclosure wall is a critical parameter that requires careful monitoring, as excessive displacement can potentially compromise the safety of the top beam and the entire internal support system.
The excavation process of the foundation pit can cause significant soil deformation and ground settlement under long-term large drawdown conditions. This settlement can lead to cracks and deformations in the surrounding buildings, affecting their stability during subsequent excavation and precipitation construction. Therefore, it is particularly important to control the settlement of the buildings around the pit. Each observation point had completed buildings located about 30–50 m away.

The variation curves of the buildings around the foundation pit at the monitoring points W1, W2, W3, W4, and W5, with the same time interval of observations, were recorded as AW1, AW2, AW3, AW4, and AW5 curves, respectively (see Figure 7). The observation time was consistent with Figure 4. The surrounding buildings outside observation point W3 were monitored every 10 days after the end of observation at other observation points.

The settlement displacements of the surrounding buildings had evident differences at the beginning, gradually increased, and then entered a stable stage with smaller and smaller differences, as seen from Figure 7. This is attributed to the unloading caused by soil excavation and precipitation in the initial stage of construction, leading to a sharp increase in the settlement around the foundation pit. In the later construction stage, the horizontal
cross bracing came into play, and the deformation around the foundation pit reached its maximum, after which the deformation entered the stable period.

4. Numerical Simulation
4.1. Numerical Model and Boundary Conditions

The modeling of the foundation pit area encompasses the entire 700 m length and 110 m width, with an excavation depth of 6.05 m on the left side and 3.8 m on the right side, and an enclosure pile length of 14.5–21 m. The model extends to more than 100 m around and two–three times the pile length in the depth direction. As such, the model’s plane dimension is 950 m × 347 m × 60 m, as shown in Figure 8.

Figure 8. Meshing of the computational model.

To conduct a numerical simulation of the foundation pit, the survey report and design drawing of the main support structure were used as inputs. The software Midas GTS NX 2022 R1 is widely used in geotechnical finite element analysis such as tunnel and foundation pit calculations, municipal subway stations, mining roadway excavation, dam stress analysis, blasting analysis, train movement load analysis, geotechnical seepage calculation, geotechnical dynamic analysis, load structure calculation, etc.; therefore, the software Midas GTS NX was utilized for the simulation, and the soil layer was simplified into four layers. The modified Mohr–Coulomb model was selected as the constitutive model of the rock and soil mass, with the unloading elastic modulus value of the soil taken as three times the compressive modulus value. The Poisson’s ratios of clay, muddy clay, and silty clay were 0.38, 0.45, and 0.42, respectively. The unit weight of concrete was 23 kN/m³, Poisson’s ratio was 0.22, and the elastic modulus was 34.5 GPa. Other parameters are shown in Table 2.

The internal support and underground diaphragm wall were modeled using the elastic constitutive model. The model was established using 2D plate elements for the underground diaphragm wall and 1D beam elements for the reinforced concrete horizontal and vertical supports and top beams. The model included 55,622 elements and 32,301 nodes. The boundary conditions of the model were automatically constrained foundation boundaries, i.e., four side normal constraints and fixed constraints at the bottom. The load was self-weight, and it was assumed that the foundation soil was all in a saturated state; the underground diaphragm wall and the bottom of the foundation were considered impermeable boundaries.
Before the excavation of the foundation pit, the initial ground stress balance and site displacement clearing of the working conditions were set. Then, the excavation was conducted according to the construction steps, as shown in Table 3.

4.2. Simulation Results

4.2.1. Horizontal Displacement Analysis of the Enclosure Wall

The monitoring point W1 was selected in the constructed model for the horizontal displacement analysis of the enclosure wall along the depth direction, and its simulated values are shown in Figure 9. Step 0 in the figure is the deformation value after ground stress balance in the simulation, and Step 2–Step 14 are the whole excavation process simulated by zoned excavation steps.

Figure 9. Horizontal displacement of the diaphragm wall along the depth direction.

Upon comparing Figure 9 with Figure 4a, it can be observed that the trend of the measured horizontal displacement values along the depth direction initially increased rapidly and then decreased gradually. However, the simulated values of the horizontal displacement along the depth direction increased gradually, stabilized, and then decreased gradually. This discrepancy can be attributed to the difference between the formation of the ground diaphragm wall in the actual construction using bored piles, and its direct imposition in the numerical simulation, leading to a higher stiffness of the simulated enclosure wall. Thus, the simulated results show a continuous equal phase of horizontal displacement, unlike the actual construction where the displacement increased rapidly and then gradually became smaller.

It was observed both in the actual construction and the numerical simulation that the horizontal displacement is relatively small near the top of the enclosure wall, as there are top beam and horizontal cross bracing in that region. Additionally, as the excavation depth of the foundation pit is small, but the height of the enclosure wall is larger than the excavation depth, the bottom deformation of the enclosure wall is also small.

The measured and simulated values both show an increasing trend in horizontal deformation of the enclosure wall as the excavation proceeds, but the measured values are significantly larger than the simulated values. This is likely due to ideal modeling conditions in the numerical simulation. In the construction process, the measured horizontal displacement gradually increased in each step, whereas the simulated values increased differently. Specifically, the simulated values show a clear increase when excavating the
area where the observation point W1 is located, but do not show an obvious increase when excavating areas ②, ③, or ④.

4.2.2. Horizontal Displacement Analysis of the Top of the Enclosure Wall

The monitoring point W1 was selected in the constructed model for the horizontal displacement analysis of the top of the enclosure wall, and its simulated and measured values were compared and analyzed under different construction stages, as shown in Figure 10. The curve representing the measured values of the monitoring point W1 is denoted as AW1, and the curve representing the simulated values is denoted as BW1.

![Figure 10. Comparison of the curves between simulated and measured values of horizontal displacement of the top of the enclosure wall.](image)

As shown in Figure 10, the simulated values do not precisely match the measured values due to the ideal modeling assumptions used in the numerical simulation. In order to improve the calculation efficiency, the soil layer was simplified by assuming that it was uniformly distributed horizontally, while the actual soil layer distribution is not exactly horizontal. The machine tools and other temporary loads that may be stacked around the pit during the construction process are not considered.

The curves of the simulated and measured results have a similar trend, and the top of the diaphragm wall at the observation point W1 moves in the horizontal direction as the construction proceeds and the displacement increases slowly and finally stabilizes. The simulated values are generally smaller than the measured values, because the modeling process simplifies the actual project and the calculated results are ideal values. However, the maximum values of both do not exceed the limit value of 30 mm, and the excavation process of the foundation pit is in a safe state.

4.2.3. Vertical Displacement Analysis of the Top of the Enclosure Wall

The monitoring point W1 was selected in the constructed model for the vertical displacement analysis of the top of the enclosure wall, and its simulated and measured values were compared and analyzed under different construction stages, as shown in Figure 11. The curve representing the measured values of the monitoring point W1 is denoted as AW1, and the curve representing the simulated values is denoted as BW1.
working conditions were selected and compared with the simulated results to verify the
simulation was conducted using Midas GTS software, and the measured values of typical
horizontal and vertical displacements of the top of the enclosure wall and settlement of
the surrounding buildings during the construction of the foundation pit. The numerical
model was assumed to be impermeable, and the difference between the water and soil pressure inside and outside the enclosure wall. The horizontal cross bracing plays a significant role in reducing the pressure inside and outside the enclosure wall, and the bottom of the foundation pit springs back, driving the enclosure wall to move upward, thus reducing the settlement of the enclosure wall. Therefore, the top of the enclosure wall will produce vertical displacement.

5. Conclusions

This paper took the foundation pit project of the Jiaojiang Xinhai Urban Shared Space Project in Taizhou, Zhejiang as its research object, and analyzed the monitoring data of the horizontal and vertical displacements of the top of the enclosure wall and settlement of the surrounding buildings during the construction of the foundation pit. The numerical simulation was conducted using Midas GTS software, and the measured values of typical working conditions were selected and compared with the simulated results to verify the scientific design of the foundation pit support structure. Three main conclusions were obtained as follows:

- The horizontal displacement of the enclosure wall in the middle of the pit is the largest, which requires extra support. The density of cross bracing impacts the horizontal displacement of the enclosure wall, with denser cross bracing resulting in smaller horizontal displacement of the enclosure wall, indicating a reasonable support configuration.

Figure 11. Comparison of the curves between simulated and measured values of vertical displacement at the top of the enclosure wall.

Figure 11 shows a noticeable difference between the trends of the simulated and measured values. The measured value stabilizes and then gradually increases, while the simulated value slowly increases, suddenly increases, and then gradually decreases. In actual construction, the top of the enclosure wall is subject to vertical displacement during initial excavation, and this displacement tends to stabilize after horizontal cross bracing is installed. However, with the gradual increase in excavation depth, the unbalanced force on the two measurements of the enclosure wall increases, the soil drainage behind the enclosure wall also increases, and the soil near the enclosure wall reconsolidates; therefore, the top of the enclosure wall will produce vertical displacement.

When establishing the numerical model, it was assumed that the soil is impermeable and that the difference between water and soil pressure inside and outside the enclosure wall during the initial process of excavation would be maintained at a large level, resulting in the settlement of the enclosure wall. As the excavation progresses, the horizontal cross bracing plays a significant role in reducing the pressure inside and outside the enclosure wall, and the bottom of the foundation pit springs back, driving the enclosure wall to move upward, thus reducing the settlement of the enclosure wall.

The maximum vertical displacements of the top of the enclosure wall, both measured and simulated, were within the limit value of 30 mm, indicating that the support structure remained stable and in a safe state throughout the excavation of the foundation pit.
The horizontal and vertical displacements of the top of the enclosure wall and the settlement of the buildings around the pit all increase gradually with the excavation process of the pit. Although they are all within the allowable limit, they approach it, indicating that the design of the support structure is at its optimal solution.

The comparison and analysis of the measured value of the horizontal displacement of the enclosure wall and the numerical simulation results indicate that the two results cannot be completely consistent, but both show a trend of increasing and then decreasing, with the numerical simulation results being smaller. Furthermore, the measured horizontal displacement of the top of the enclosure wall follows a similar trend to that of the numerical simulation, albeit with the numerical simulation value being smaller. The measured value of the vertical displacement of the top of the enclosure wall has a large difference from the numerical simulation result. The results suggest that finite element numerical software is a feasible tool for simulating pit excavation, but the accuracy could be improved with real-time monitoring points for calibration.

Comparing the in-site data of the top of the enclosure wall with the numerical simulation results, it can be seen that numerical simulation can effectively reflect the variation pattern of displacement, but the horizontal displacement in the numerical simulation needs to be corrected. It is recommended to use in-site data for correction.

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