Research Article

Monitoring and Prediction Analysis of Settlement for the Substation on Soft Clay Foundation

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The bearing capacity of soft clay foundations is low, which easily causes excessive cumulative and uneven settlement of the substation. This will seriously threaten the safe operation of equipment in the substation. Based on a 220-kV substation project in a coastal soft clay ground, the settlement of the substation building is monitored on the spot, and the settlement law of soft clay ground is analyzed. Furthermore, a three-dimensional finite element model is developed using finite element software named ZSOIL.PC. A numerical model for predicting the settlement of soft clay ground in a substation is proposed using the appropriate element type, material constitutive model, and calculation parameters. The rationality of the numerical method is validated by comparison with the measured data. After improving the soft clay ground with the cement-soil deep mixing method, there are still certain degrees of cumulative and differential settlements in the substation distribution device building, but both are within the specification requirements. The settlement of the soft soil ground in the substation increases with time, but the growth rate gradually decreases and ultimately tends to stabilize after a certain period of time, ensuring the uniformity of foundation treatment is the key to controlling the differential settlement. The research results can provide a reference for the subsequent installation and construction of electrical equipment and the design of substation engineering in soft soil ground.

1. Introduction

As an important place to transform and transmit electric energy in the power grid, the substation is the core and link of the entire power grid system and is very important for the safe and stable operation of the power system. Land resources have become increasingly scarce with the rapid development of the social economy. To ensure the normal power supply in high concentrated neutral power load areas, substations are sometimes forced to be built on coastal soft clay foundations. Soft clay foundations have a high water content, high compressibility, and usually low bearing capacity [1]. Coupled with the effects of temperature, rainfall, and environmental loads of the substation, large settlement deformation and uneven settlement of the substation can easily result and can seriously threaten the safe and stable operation of the equipment in the substation (typical examples are shown in Figure 1 below). To improve the quality of substation projects and ensure the safe and stable operation of the power grid, it is necessary to implement settlement monitoring for substation projects [2, 3].

A large number of studies have explored the problem of settlement of soft clay foundations, and many methods have been proposed to calculate the settlement of foundations. Considering the influence of lateral stress on foundation settlement, Skempton and Bjerrum [4] proposed a modified method to calculate the final main consolidation settlement using the three-dimensional pore pressure obtained under the undrained triaxial stress condition. Burland [5] studied the structural problem of soft clay compression and considered that the compression coefficient is small in the range of structural yield stress; it will rapidly increase when it is greater than the structural yield pressure, and the compression curve gradually approaches the compression curve of the remolded soil. Indraratna et al. [6] carried out a three-dimensional numerical simulation of excess pore water
pressure and consolidation settlement under vacuum pre-loading. Shen [7] described the mechanical behavior of structural soils using damage theory and established the evolution law of soil damage. Wang and Chen [8, 9] proposed an analytical expression to calculate the one-dimensional consolidation degree of a structural soft clay foundation by piecewise linearization of the consolidation curves. Feng et al. [10] presented a simplified method for calculating the multistage creep settlement of multilayer soft clays under one-dimensional strain conditions. Wang et al. [11] introduced the concept of the damage ratio to the Duncan-Zhang model to consider the structure of soft clay. Zhao et al. [12] calculated the settlement of soft clay foundations by developing a nonlinear elastic-viscoplastic creep constitutive model. Yu et al. [13] calculated the compressibility of the pile-soil system and studied the settlement characteristics of granular filler on saturated soft clay foundations. Rui et al. [14] calculated the settlement of soft clay foundations considering the effect of structural characteristics based on laboratory and in situ test data. Shi et al. [16] analyzed the settlement of an embankment on a soft foundation using the finite element method when considering the rheology and damage of the structural soil based on a soil elastoplastic model. Jin et al. [17] computed the viscoelastoplastic deformation of soft clay using the modified Nishihara model with a nonassociated flow rule. Hu et al. [18] presented a monitoring program of substation foundation settlement based on distributed optical sensing technology to solve the problem of foundation settlement monitoring.

In summary, there have been relatively few studies on the settlement deformation law of substations on coastal soft soil foundations, and there is also a lack of appropriate numerical methods to predict the settlement. In this study, settlement monitoring was carried out for a 220-kV substation project in coastal soft soil ground, and the settlement law of the substation ground was analyzed according to the monitoring results. Based on the finite element software ZSOIL.PC, a three-dimensional finite element model was developed to predict the long-term cumulative settlement of substation buildings. The numerical model for predicting settlement was validated by comparison with the measured data. The research results can provide a reference for the engineering design of substation projects in soft soil grounds.

2. Project Overview

2.1. Project Scale. The substation site is located 82 kilometers north of Binzhou city in Shandong province, China, near the Taoer River Estuary. The main buildings in the substation include two distribution device buildings and four main transformer foundations, of which the 220-kV power distribution building is on the south side of the station area with two floors above the ground. The 110-kV power distribution device building is on the north side of the station area, with one underground floor (cable interlayer) and two floors above the ground. The four main transformers are arranged in the middle of the station area, and a ring-shaped fire road is set around it. A 220-kV power distribution building was first built, and then, a 110-kV power distribution device building was constructed. The two power distribution buildings and the four transformers are shown in Figure 2, and their geometry can be approximated as a cube. Their layout is shown in Figure 6.

2.2. Engineering Geological Conditions. According to the geological survey results, combined with the regional geological data collected and the experience of existing nearby construction projects, the landform genetic type of the site is a sea-land interaction sedimentary plain, and the landform type is a coastal beach. The site is flat, with an elevation of 1.25–1.42 m. It was originally a shrimp pond and was always located below the water surface. The site is composed of Quaternary Holocene artificial fill (Q4a) and a Holocene sea-land interaction sedimentary layer (Q4mc), and the lithology is silt clay, silt, and fine sand. A topographic map of the station site during the geological survey is shown in Figure 3. A total of 19 static penetration test holes are arranged in the project. The exploration spacing in the station site area is 12.50–40.20 m, and the exploration depth is 17.00–25.00 m.
The number, spacing, and depth of exploration points of the project meet the relevant requirements of the “Code for Investigation of Geotechnical Engineering (GB 50021–2001)” [19] and the “Technical Code for Geotechnical Investigation of Substation (DL/T5170-2015)” [20]. In addition, to determine the physical property index of the soil, representative soil samples were selected to carry out laboratory soil tests, e.g., direct shear tests, compaction tests, and unconfined compression tests. Typical static cone penetration test [21] results are shown in Figure 4. The typical physical property index for each soil layer is shown in Table 1. The stratigraphic characteristics of the proposed site are described as follows from new to old and from top to bottom:

(i) A layer of muddy clay, saturated, soft plastic to flow plastic state, uneven soil, obvious bedding, with a thin layer of silt and fine sand, and local mixed shell debris
(ii) A layer of silty soil, slightly wet ~ very wet, medium-dense state, uneven soil, stratification is obvious, sandwiched viscous soil, sandy soil thin layer, and a large amount of local mixed shell debris
(iii) A layer of muddy clay, saturated, soft plastic to flow plastic state, uneven soil, obvious bedding, sandy silt and fine sand, and local mixed shell debris
(iv) A layer of fine sand, loose state, uneven soil, obvious bedding, sandwiched thin layer of silt and fine sand, and mixed with a large number of shell debris in a layered distribution
(v) A layer of silty soil, slightly wet ~ very wet, medium-dense state, uneven soil, stratification is obvious, sandwiched viscous soil, sandy soil thin layer, and a large amount of local mixed shell debris

2.3. Foundation Treatment Measures. According to the geological conditions of the project, the service environment, equipment operation, and load characteristics of the substation building, the cement-soil mixing method, a deep mixing method, was used in the project for foundation treatment. Construction and quality inspection were carried out in strict accordance with the requirements of the “Technical Code for Ground Treatment of Buildings” JGJ79-2012 [22] and the “Technical Specifications for Power Engineering Foundation Treatment” DL/T5024-2005 [23]. The curing agent of the mixing pile was ordinary Portland cement with a strength grade of 42.5, and the cement content was 55–65 kg/m. At the same time, concrete antisulfate corrosion preservatives were added according to 6–8.0% of the cement content.

Figure 5 shows the layout of the cement-soil mixing pile for foundation treatment. The cement-soil mixing pile is arranged in a rectangle, with a horizontal and vertical spacing of 1 m, a pile diameter of 0.5 m, and a pile length of 11 m. The characteristic value of the bearing capacity of the composite foundation is \( f_{spk} \geq 150 \text{kPa} \) for 110-kV and 220-kV power distribution buildings after treatment, and the value is \( f_{spk} = 120 \text{kPa} \) for the main transformer and firewall, lightning rod, water pump house, fire pool, etc. The characteristic value of the bearing capacity for a single pile is \( R_a \geq 123 \text{kN} \). The standard value of cubic compressive strength for the cement-soil mixing pile is \( f_{cu} \geq 2.5 \text{MPa} \). The compaction coefficient of soil after site leveling is 0.95.

3. Settlement Monitoring

3.1. Monitoring Scheme. On-site settlement observations are carried out by the geometric leveling method, and its accuracy is second-class leveling according to the provisions of the Code for Engineering Survey (GB50026-2007) [24] and
the Technical Code for Construction Survey of Electric Power Engineering (DL/T 5445–2010) [25]. A TOPCON DL501 automatic precision electronic level made in Japan with a TOPCON BIS20 bar code indium steel level was used for measurement because it has a very high accuracy for deformation observation. According to the two codes mentioned above, settlement monitoring points are mainly selected around the building and the important load-bearing parts, settlement joints, and both sides of the postpouring belt. Monitoring points N1, N4, N5, and N8 (or S1, S4, S5, and S8) are the four corners of the power distribution building in Figure 6, which are prone to stress concentration at special locations, so they are used as monitoring points. Monitoring points N2, N3, N6, and N7 (or S2, S3, S6, and S7) are close to the settlement joints and the postpouring belt of the power distribution building in Figure 6, so they are also selected as monitoring points. The layout of the settlement monitoring points is shown in Figure 6. After completing the construction, the settlement of the 220-kV and 110-kV power distribution buildings was monitored for 148 and 101 days, respectively.

3.2. Monitoring Results. Figure 7 shows the accumulative settlement versus the observation time for the 220-kV power distribution building after completing the construction. As seen in the figure, the settlement of each monitoring point gradually increases with increasing observation time, and until the observation deadline, the increase in settlement does not seem to stop. Among them, monitoring points S1, S4, S5, and S8 have similar trends, while monitoring points S2, S3, S6, and S7 have similar trends, as shown in Figure 7.

The average value of the settlement for monitoring points S1 and S8 is taken as the settlement amount on the left
side of the building, the average value of the settlement for monitoring points S4 and S5 is taken as the settlement amount on the right side of the building, and the average value of the settlement for monitoring points S2, S3, S6, and S7 is taken as the settlement amount on the middle part of the building. The calculation results are shown in Figure 8. As shown in the figure, there is an obvious uneven settlement on the left, middle, and right sides of the 220-kV power distribution building, and the settlement amounts on the left and right sides are less than that in the middle. There are two main reasons for this difference. One is that the properties of ground soil have some difference below the whole building, so the soil strength after reinforcement is not uniform. For this project, the soil on both sides has been better strengthened than that in the middle, so the corresponding subgrade bearing capacity is also comparatively higher. Another reason is that the additional stress caused by the deadweight of the superstructure in the middle of the foundation is larger than the stress levels on both sides.

According to the three average settlements shown in Figure 8, we can calculate the differential settlement between the left and middle sides, the middle and right sides, and the left and right sides. The calculation results are shown in Figure 9. The differential settlement amounts on the left and right sides are very small, with values close to 0. However, the difference between the left and the middle and the
difference between the right and the middle are relatively large, and the differences for the two are essentially the same and tend to converge after approximately 120 days.

The power distribution building of the substation belongs to the frame structure of industrial and civil buildings. The soils of the project site are high compressibility soil. According to the relevant requirements in the Code for the Design of Building Foundations (GB50007-2011) [26], the settlement difference between adjacent columns should be less than 0.003 L (L is the distance between any two measuring points). Table 2 shows the values of differential settlement among the monitoring points. The differential settlement between any two adjacent measuring points is within the allowable range, indicating that the settlement of the 220-kV power distribution building is safe.

Figure 10 shows the accumulated settlement of the 110-kV power distribution building. As shown in the figure, the settlement of each monitoring point continues to increase with increasing time. After 101 days of observation, the onsite settlement still shows a significant growth trend. In general, the settlement of the 110-kV power distribution building was smaller than that of the 220-kV power distribution building for the same observation days. This is mainly because the upper load of the 110-kV distribution building is less than that of the 220-kV distribution building. The maximum, minimum, and average values of the total settlement of the 8 measuring points were calculated, and the differential settlement was determined by calculating the difference between the maximum settlement and the minimum settlement. The calculation results are shown in Figure 11. The differential settlement tends to be stable after approximately 75 days, which is partly caused by the recent construction of the 220-kV power distribution building. The main reasons are explained as follows: the 220-kV power distribution building was still experiencing overall settlement and uneven settlement after construction was completed. However, the 110-kV power distribution building is close to the 220-kV building. Due to the additional stress caused by the deadweight of the 220-kV power distribution building, the side close to the 220-kV power distribution building may experience greater settlement compared with the other side, which will cause differential settlement of the 110-kV building. The maximum measured differential settlement between monitoring points is 4.11 mm, which is far less than the allowable value of 32 mm, indicating that the differential settlement of the 110-kV power distribution building is also safe.

It should be noted that the overall length of the two power distribution buildings is large, and the whole site is constructed by filling soil. The site in the station area is higher than the surrounding site. The water content of soil greatly and unevenly changes, and the properties of the foundation soil are different. As a result, the bearing capacity of the soil in different positions of the whole foundation is different to some extent. Although the cement-soil mixing
method is adopted for foundation treatment in this project, the treatment effect is still not ideal, and the strength of the foundation soil after reinforcement is not uniform. In general, the reinforcement effect of both sides of the power distribution building is better than that of the middle part. In addition, the additional soil stress caused by the dead weight of the superstructure in the central part of the building is larger than those on both sides. Therefore, the settlement in the middle of the two distribution buildings is larger than the settlement on both sides.

4. Numerical Simulation of the Foundation Settlement

Uneven settlement occurred in the construction process of the transformer substation, and the settlement did not tend to be stable at the observation deadline, which affected the installation and construction of subsequent electrical equipment. Therefore, based on the settlement observation data of the station buildings and the geotechnical engineering investigation report of the site, with the help of three-dimensional geological modeling technology, a three-dimensional geological model based on-site borehole data and engineering geological profile were established to simulate the settlement of the foundation. The development trend of the settlement and the final settlement value is predicted and analyzed using the numerical model, which can provide a reference for the installation of electrical equipment. In this study, a true three-dimensional finite element software program for geotechnical engineering, named ZSOIL.PC and developed by the Swiss Federal Institute of Technology, is used for numerical analysis.

4.1. Simulation of Stratigraphic Spatial Distribution. The accuracy of the spatial distribution of strata is the basis of the accuracy of geotechnical numerical analysis. Generally, the spatial distribution of strata is uneven, and the soil layer exhibits certain phenomena, such as thickness changes and pinching out in space. At present, the spatial distribution of strata is generally determined according to the experience of engineers and linear interpolation of borehole data. In this study, based on limited geological borehole data, the spatial distribution of strata is calculated using the three-dimensional geological borehole spatial interpolation technology owned by the ZSOIL.PC software. Stratum information can be automatically generated by inputting borehole data, such as coordinates, thickness of strata, and soil category, as shown in Figure 12.
4.2. Three-Dimensional Finite Element Model. Combined with the background of the project overview and the layout of buildings, the overall three-dimensional finite element model of the ground and the substation building is established, as shown in Figure 13. A sensitivity study was conducted to establish the length, width, and height of the soil domain to eliminate the boundary effects on the results. It was found that a mesh size of $121.5 \times 87.8 \times 34.65$ m (length $\times$ width $\times$ height) is sufficient to avoid boundary effects on the obtained results. It should be noted that under the action of self-weight, the structure and foundation mainly experience vertical settlement deformation with very small horizontal deformation. The height of 34.65 m for the soil domain is sufficient to avoid the effect of the boundary on the settlement results.

For the building components in the substation, the beam element is used to simulate the beam and column system, the shell element is used to simulate the floor, and the solid element is used to simulate the foundation. The load on the floor is simulated by a uniform load, and the infilled wall is replaced with a uniform load. The number of model nodes is 63,788, the number of hexahedral solid elements is 53,764, the number of one-layer shell elements is 6172, the number of beam elements is 3087, and the number of seepage elements is 1764.

It is assumed that there is no horizontal displacement at the vertical boundary of the model due to the constraint of lateral Earth pressure in the infinite soil domain; hence, normal horizontal constraints were applied to the vertical boundaries. Fixed constraints were applied to the bottom boundary since there was no displacement at the bottom of the model. Obviously, the top boundary was fully free. In addition, the top and vertical boundaries are permeable boundaries because soil seepage can occur at these boundaries, and the bottom boundary is an impermeable boundary since there is almost no seepage on the boundary with thick soil depth.

Based on the above three-dimensional finite element model, fluid-solid coupling analysis of the foundation settlement is performed using the ZSOIL.PC software based on Biot’s consolidation theory. The analysis includes 4 working conditions: working condition 1, initial geostatic balance, and displacement clearing; working condition 2, layered backfilling, construction of mixing piles, and foundation; working condition 3, construction of the 220-kV power distribution building; and working condition 4, construction of the 110-kV power distribution building. The three-dimensional finite element models for the four working conditions are shown in Figures 13(a), 13(b), 13(c), and 13(d).

4.3. Calculation Parameters. The constitutive model and related calculation parameters are the keys to finite element analysis. The ground has a certain settlement with the growth of time under the additional stress caused by the deadweight of the substation building. However, from the actual monitoring results, it is far from reaching the failure state of the foundation (i.e., a large deformation state). The importance of considering small strain behavior has been highlighted when addressing soil deformation issues caused by geotechnical engineering construction [27, 28]. However, traditional elastoplastic constitutive models (e.g., Mohr-Coulomb, Duncan-Chang, and modified Cam-Clay models) cannot capture small-strain behaviors, which could lead to computational inaccuracy when used to assess ground settlement. Therefore, the Hardening Soil model with small-strain stiffness (HSS model) proposed by Benz et al. [29, 30] was adopted to simulate the stress-strain behavior of soil under static loading. It has been demonstrated that the HSS model, as an extension of the Hardening Soil model (HS model), can simulate the unrecoverable plastic shear deformation and volume deformation of soil with increasing stress, especially the nonlinear behavior of soil under small strain [29, 30]. A detailed introduction of the HSS model can be found in reference [29, 30]. Limited to space, a brief introduction to its basic theory and parameters is provided here.

The shear yield function of the HSS model in principal stress space can be expressed as follows:

$$f_{12} = \frac{2q_a}{E_{50}} \left( \sigma_1 - \sigma_2 \right) \left( \sigma_1 - \sigma_2 \right) - \frac{2(\sigma_1 - \sigma_3)}{E_{irr}} - \gamma^p,$$

$$f_{13} = \frac{2q_a}{E_{50}} \left( \sigma_1 - \sigma_3 \right) \left( \sigma_1 - \sigma_3 \right) - \frac{2(\sigma_1 - \sigma_2)}{E_{irr}} - \gamma^p,$$

where $q_a$ is the average effective stress, $E_{50}$ and $E_{irr}$ are the initial and peak shear modulus, respectively, and $\gamma^p$ is the plastic strain.
where $\gamma^p$ is the cumulative plastic shear strain; $q_a$ is the asymptotic strength; $E_{50}$ is the secant strength corresponding to 50% strength; and $E_{ur}$ is the loading and unloading (rebound) modulus of the soil.

The HSS model adopts the nonassociated flow rule, and its plastic potential function is as follows:

$$
g_{i3} = \frac{\sigma_1 - \sigma_2}{2} - \frac{\sigma_1 + \sigma_3}{2} \sin \Psi, \tag{2}
g_{i2} = \frac{\sigma_1 - \sigma_2}{2} - \frac{\sigma_1 + \sigma_2}{2} \sin \Psi,
$$

where $\Psi$ is the dilatancy angle, which is related to the increment of plastic volumetric strain $\Delta \varepsilon^p_v$ and plastic shear strain increment $\Delta \gamma^p$. The relationship can be expressed as follows:

$$
\Delta \varepsilon^p_v = \sin \Psi \Delta \gamma^p. \tag{3}
$$

The HSS model considers the stress-related characteristics of the $E_{50}$ modulus and $E_{ur}$ modulus, which can be expressed as follows:

$$
E_{50} = E_{50}^{ref} \left( \frac{c' \cot \varphi' - \sigma_3}{c' \cot \varphi' - \sigma_{ref}} \right)^m, 
E_{ur} = E_{ur}^{ref} \left( \frac{c' \cot \varphi' - \sigma_3}{c' \cot \varphi' - \sigma_{ref}} \right)^m, \tag{4}
$$

where $E_{50}^{ref}$ and $E_{ur}^{ref}$ are the $E_{50}$ modulus and $E_{ur}$ modulus corresponding to reference stress $\sigma_{ref}$. $m$ is a parameter related to soil properties.

The cap yield surface of the HSS model is expressed as follows:

$$
\tilde{f}^c = \frac{\tilde{q}^2}{\alpha^2} + p^2 - p_p^2, \tag{5}
$$

where the average principal stress $p = \sigma_n/3$; $\alpha$ is the ratio of the intercept of the cap yield surface on the $p$ and $q$ axes in the $p$-$q$ plane; $p_p$ is the initial consolidation pressure; and $\tilde{q}$ is the deviatoric stress, which can be calculated by $\tilde{q} = \sigma_3 + (\delta^{-1} - 1)\sigma_2 - \delta^{-1}\sigma_3$, in which $\delta = (3 - \sin \varphi')/(3 + \sin \varphi')$. 

![Figure 13: Three-dimensional finite element model. (a) Working condition 1. (b) Working condition 2. (c) Working condition 3. (d) Working condition 4.](image-url)
Table 3: HSS model input parameters.

| Parameters | Description                        | Soil layer number | Reinforced soil |
|------------|------------------------------------|-------------------|-----------------|
| \( c' \)   | Effective cohesion                 | 1                 | 46              |
| \( \varphi' \) | Effective friction angle          | 11.8              | 41              |
| \( \Psi \)  | Dilatancy angle                    | 0                 | 9               |
| \( \sigma_{\text{ref}} \) (kPa) | Reference stress for stiffness    | 100               | 100             |
| \( m \)    | Power for the stress-level dependency of stiffness | 0.8               | 0.8             |
| \( v_{\text{ur}} \) | Poisson ratio                    | 0.2               | 0.2             |
| \( G_{\text{ref}} \) (MPa)     | Reference shear modulus at very small strain | 79.5              | 540             |
| \( E_{\text{ref}} \) (MPa)     | Reference tangent stiffness for primary oedometer loading | 9.3               | 63.5            |
| \( E_{\text{ref}} \) (MPa)     | Reference secant stiffness in standard drained triaxial test | 9.3               | 63.5            |
| \( K_0 \)   | \( K_0 \) value for normal consolidation | 0.79              | 0.34            |
| \( R_f \)   | Failure ratio                      | 0.9               | 0.9             |
| \( \gamma_{0,7} \) (10^{-4}) | Shear strain at which \( G_i = 0.7G_0 \) | 2.4               | 3.2             |

The HSS model adopts the famous Hardin-Drnevich model [31] to describe the hyperbolic relationship between shear stiffness and strain in the small strain region, which can be described as follows:

\[
G = G_0 \left( \frac{c'}{c'} \cot \varphi' - \sigma_{\text{ref}} \right)^m
\]

where \( G_0 \) is the initial shear modulus of the soil, and \( \gamma_{0,7} \) is the shear strain at which \( G/G_0 = 0.7 \). \( G_0 \) and reference initial shear modulus \( G_{\text{ref}} \) meet the relationship represented by the following:

\[
G_0 = G_{\text{ref}} \left( \frac{c'}{c'} \cot \varphi' - \sigma_{\text{ref}} \right)^m.
\]

The HSS model includes 11 HS model parameters (\( c' \), \( \varphi' \), \( K_0 \), \( \Psi \), \( m \), \( \sigma_{\text{ref}} \), \( v_{\text{ur}} \), \( E_{50} \), \( E_{\text{ref}} \), \( E_{\text{oed}} \), and \( R_f \)) and 2 small strain parameters (\( G_{\text{ref}} \) and \( \gamma_{0,7} \)). The meanings of these parameters and their values in this numerical simulation are shown in Table 3. Parameters \( K_0 \), \( \Psi \), \( \sigma_{\text{ref}} \), \( v_{\text{ur}} \), and \( m \) can be determined by referring to existing research results [32–34]: \( K_0 = 1 - \sin \varphi' \); for sand and silt, \( \Psi = \varphi' - 30 \) if \( \varphi' > 30 \), else \( \Psi = 0 \), and for cohesive soil \( \varphi' = 0 \); \( \sigma_{\text{ref}} \) is usually 100 kPa; \( v_{\text{ur}} \) is usually 0.2; for sand and silt, \( m \) is usually 0.5, and for cohesive soil, \( m \) is usually 0.5–1; and \( m = 0.8 \) in this study. Other model parameters can be determined by performing laboratory soil tests: \( c' \), \( \varphi' \), \( R_f \), and \( E_{50} \) can be determined by performing conventional triaxial consolidated drained shear tests; \( E_{\text{ur}} \) can be determined by performing triaxial consolidated drained loading and unloading tests; and \( G_{\text{ref}} \) and \( \gamma_{0,7} \) can be determined by performing resonance column tests. To ensure the accuracy of the material parameters, the average value of the parameter for three representative soil samples is usually required as the final parameter value. It should be noted that the strength of the soil after foundation treatment was significantly improved, which can be confirmed by the parameters of the improved soil. However, the method of determining model parameters for improved soil is the same as that for unimproved soil.

4.4. Analysis of Numerical Simulation Results. The comparison between the numerical and measuring results of settlement with time for the 110-kV power distribution buildings is shown in Figures 14 and 15, respectively. As shown in the diagrams, for the two buildings, the predicted value is in good agreement with the measured value in the overall trend, and the settlement of each monitoring point continuously increases with time. For the 110-kV power distribution building in Figure 14, at the initial stage of settlement observation, the predicted results are relatively higher than the measured results, which may be related to the construction environment around the substation, i.e., the road loads at both ends of the substation and the nonuniformity of soil improvement. However, these factors cannot be well considered in numerical simulations. In the overall trend, the numerical simulation results are in agreement with the
Figure 15: Comparison between numerical and measuring results of settlement with time for 220-kV power distribution buildings.

Figure 16: Settlement of a typical monitoring point in the middle of the substation building. (a) Monitoring point N3. (b) Monitoring point N6. (c) Monitoring point S3. (d) Monitoring point S6.
FIGURE 17: Empirical prediction curve for the settlement of power distribution buildings. (a) 110-kV power distribution building. (b) 220-kV power distribution building.

FIGURE 18: Continued.
measured results, which can provide a reference for the foundation settlement prediction of follow-up works.

According to the development law of monitoring data and numerical analysis data, the settlement rate of the substation foundation decreases with time, but the settlement is still unstable 300 days later. Therefore, the observation time is extended to 900 days, and monitoring and prediction data of forecast points in the middle of the substation with large settlements are shown in Figure 16. The settlement of the substation foundation finally tends to be stable after approximately 700 days. The final maximum settlements of the 110-kV and 220-kV power distribution buildings are approximately 22.4 mm and 40.7 mm, respectively. Although the total settlement and differential settlement of the project are within the allowable range of the code, it is recommended not to connect the sealing kit during equipment installation due to the high settlement requirements of electrical equipment in power distribution buildings. In addition, Figure 17 shows the settlement prediction results and best-fitting curves for all monitoring points of 110-kV and 220-kV power distribution buildings. It was found that the settlement of power distribution buildings can be predicted by the empirical formula:

$$ s = a(1 - b^t), $$

where $s$ is the settlement (mm), $t$ is time (day), and $a$ and $b$ are two empirical constants. In this project, the values of $a$ are -20.728 and -38.385 for the 110-kV and 220-kV power distribution buildings, respectively, and $b$ is 0.991.

The deformation nephograms of the foundation and structure are shown in Figure 18. Figures 18(a) and 18(b) show the deformation nephograms one week after completing the construction of the 220-kV and 110-kV power distribution buildings, respectively. Figure 18(c) shows the deformation nephogram when the settlement of the two buildings tends to be stable. The settlement of the foundation and building significantly increases with time and eventually tends to stabilize. In addition, the settlement in the middle parts of the two buildings is greater than those on both sides, and this trend agrees with the measured settlement results in the substation, which demonstrates the rationality of the numerical simulation method.

5. Conclusions

In this study, settlement observations of substations on soft clay foundations are carried out, and numerical simulations are performed to predict the settlement of substation foundations using the three-dimensional geological engineering software ZSOIL.PC. The law of foundation settlement is analyzed, and the following main conclusions are obtained as follows:

(1) For the two power distribution buildings, the cement-soil deep mixing method is adopted to improve the soft soil ground in strict accordance with relevant specifications. However, there are still certain degrees of accumulated settlement and differential settlement in power distribution buildings of substations, but the settlement is within the requirements of codes, and the project remains safe. The key to controlling differential settlement is to ensure the uniformity of foundation treatment.

(2) Based on the three-dimensional geological engineering software ZSOIL.PC, the three-dimensional finite element model of soft soil foundations and substation buildings is established. The numerical analysis method for predicting the settlement of soft soil foundations in substations is proposed using appropriate element types, material constitutive models, and calculation parameters. Then, the rationality of the method is validated by comparison with the measured data. An empirical formula is proposed to predict the settlement of power distribution buildings.
(3) The settlement of soft soil ground in the substation increases with time, but the growth rate gradually decreases and ultimately tends to stabilize after a certain period of time. The main reason for settlement is that although the bearing capacity of soft soil ground has significantly increased after treatment, the soil has not been fully consolidated after the construction is completed and will be further consolidated under the action of additional stress caused by the superstructure. The research results can provide a reference for the subsequent installation and construction of electrical equipment and the design of substation engineering in soft soil ground.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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