Research Article

Numerical Simulation of the Construction Process of Long Spiral CFG Piles

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1. Introduction

Cement fly-ash gravel piles (CFG piles) composite foundation is widely used in engineering for the advantages of fast construction speed, low cost, and the utilization of industrial waste-fly ash [1–3]. The construction process of long spiral CFG piles is divided into four sequences, as shown in Figure 1. First, the auger is moved to the designed pile position. Afterwards, the drilling tool is drilled into the soil until the targeted pile depth is reached (Step 2). During this process, the surrounding soil is disturbed. In sequence three, the concrete is pumped into the drill pipe through the concrete pump. When the hollow drill pipe is filled with concrete, the valve on the drill pipe head is opened, and then the drilling tool is lifted at a constant speed (Step 4). The hardening of the concrete takes place in the subsequent process.

The underground of Zhengzhou area often contains loose and saturated sandy and silty soil [4]. When the construction of CFG piles is carried out in this area, the vibration load generated by the drilling tool will lead to liquefaction of saturated soil. The liquefaction of saturated sand directly leads to construction difficulties, pile quality defects [5, 6], and road cracks, which cause not only waste of resources, but also more serious environmental pollution problems [7, 8]. Engineering practice shows that the quality faults of piles and the cracking of surrounding soil during construction are closely related to the liquefaction of saturated soil [9, 10]. Sun et al. [11] and Zhao et al. [12] found that there is a large excess pore water pressure in saturated sandy and silty soil after the construction of CFG piles. Ji et al. [13] and Yu et al. [14] measured the excess pore water pressure on-site generated during the construction of CFG piles in clay. However, the parameters directly observed through field tests are limited, and it is difficult to observe the changes of soil state during the construction of CFG piles. Numerical simulation can make up for the deficiency of experiments, so it was carried out to research the above problems.

Several finite element models have been established by using Lagrangian element method to investigate the changes
of soil stress and displacement during the construction of hammer driven piles and static piles [15–18]. Yang [19] considered the construction process of bored cast-in-place piles by equating the drilling process with the load of hole-wall. However, it did not simulate the dynamic process in a real sense. So far, numerical models of the construction process of long spiral CFG piles have not been established because of the complexity of the construction process.

The construction process of CFG piles is a large deformation problem in geotechnical engineering, while grid distortion problems that cause simulation interruption are often produced as using finite element simulation. There are three numerical methods that can handle large deformation problems: the implicit remeshing and interpolation technique by small strain (RITSS) [20–23], an efficient Arbitrary Lagrangian–Eulerian (EALE) implicit method [24–27], and the coupled Eulerian–Lagrangian (CEL) approach [28–32]. Neither of the first two methods can effectively simulate the fragmentation and flow of soil involved in the construction of long spiral CFG piles, while the CEL method in ABAQUS can make up for the above deficiencies. Therefore, this method is tried to be used for modeling.

The CEL method is applied in this paper to numerically analyze the construction process of long spiral CFG piles and to research the influence of the construction on the saturated sand foundation. The numerical analysis method is briefly introduced and two numerical models are given to simulate the construction process of CFG piles. The field test and test results are shortly introduced to illustrate the influence of field construction. The results of simulation are discussed and compared with theoretical analysis results.

2. Coupled Eulerian–Lagrangian Method

2.1. CEL Method. Noh [33] proposed the coupled Eulerian–Lagrangian method to deal with the large deformation problems. The principle of the CEL method is to use Lagrangian elements for objects with greater stiffness, while Euler elements are used for materials that are prone to greater deformation in the analysis. This method that combines the advantages of the Eulerian formulation with the advantages of Lagrangian formulation overcomes mesh distortion problems.

The Lagrangian structure can invade Eulerian material while Eulerian material is forced to move around when Lagrangian structure interacts with Eulerian materials. The Eulerian material is tracked as it flows through the mesh by computing its Eulerian volume fraction (EVF). The EVF of material in each Eulerian element is between 0 and 1. 1 means that the element is completely filled with the material. On the contrary, if there is no material in the Eulerian element, its EVF is 0.

An explicit time integration scheme is used in the CEL method and the central difference rule is employed for the solution of the nonlinear system of differential equations. The analyses are dynamic and the time duration modeled is meaningful which directly affects the simulation time. The advantage of explicit calculations is that no iteration is needed, but the calculations are not stringently stable. Numerical stability is guaranteed by introduction of the critical time step size, which is roughly proportional to the smallest element length and inversely proportional to the elastic stiffness and the wave velocity of the material. Discussion of mesh density, velocity, and critical time step in particular problems can also be found in Qiu and Henke [34] and Tho et al. [35].

2.2. Constitutive Model. Basic constitutive models are applied in this paper for the stability of the simulation. The Mohr–Coulomb plastic model in cooperation with the linear elastic model is used in this paper [36]. Nevertheless, the expansion angle should be set reasonably because the material is prone to large expansion under shear load in the Mohr Coulomb plastic model. Moreover, the value of elastic modulus should be in line with the reality for the dynamic elastic modulus of saturated sand will decrease under continuous dynamic load [37] and the dynamic triaxial test data of silty sand can be seen from Figure 2 [38].
Figure 2: Changes of dynamic elastic modulus with dynamic strain [38].

2.3. Contact Formulation. General contact that can automatically calculate the contact surface between Lagrangian structure and Eulerian materials is applied in the CEL method. The contact is realized by setting a friction coefficient in the penalty contact method. The coefficient is 0.3.

2.4. Calculation of Excess Pore Water Pressure. There is a state of stress balance in soil before the construction of CFG piles, and this state will change because of the dynamic disturbance caused by the construction. The excess pore water pressure $\Delta u$ caused by the construction is calculated via [39]

$$\Delta u = -\Delta \sigma_1 + A(\Delta \sigma_1 - \Delta \sigma_3),$$

where $A$ is Skempton pore pressure parameter and $\Delta \sigma_1$ and $\Delta \sigma_3$ are the major and minor total principal stress increments caused by construction, respectively, which take tension as positive.

3. Numerical Model

Numerical model based on CEL method assumes the following: (1) the soil is fully saturated and the groundwater level is on the top surface of the soil; (2) the soil particles and water are incompressible; (3) the construction process of the CFG pile is short and it is an undrained process; (4) soil is an ideal isotropic continuous medium subject to Mohr–Coulomb plasticity.

3.1. The Model for Drilling Process. The foundation to be analyzed is saturated sand foundation, and a three-dimensional model with a scale of 1/10 is created to simulate the drilling process. The CFG drilling tool has an outer diameter of 4 cm, an inner diameter of 2 cm, and a pitch of 4 cm and the bit is simplified as a conical tip. The distance between the CFG pile and the model boundary is about 15 d, where d is the diameter of the model piles. A void region with a height of 25 cm is discretized on top of the soil section, in order to enable soil movement into this region during the drilling process, as shown in Figure 3.

The CFG drilling tool is modeled as a volumetric rigid body with 23701 C3D4 elements and the soil and pore area consist of 98,000 elements that are EC3D8R (eight-node Eulerian element). The material parameters used in the numerical simulations are depicted in Table 1. The values of parameters of saturated silty sand are based on the parameter tests of field sand. The initial ground stress is calculated with the parameter of static Earth pressure $K_0 = 0.4$, which takes into account the influence of the soil skeleton pressure and the hydrostatic pressure.

The rotation velocity $v_r$ and the penetration velocity $v_z$ of the drilling tool are controlled by the rotation and translation degrees of freedom of the reference point. The ratio between the rotation velocity and the penetration velocity is varied from 5 to 15 U/m (rotations per meter) to determine the influence of this parameter. The Eulerian boundaries and displacement boundaries are set around and at the bottom of the soil to simulate the real boundaries.

3.2. The Model for Concrete Pouring Process. Pucker and Grabe [28] completed the simulation of the drilling process of the full displacement piles, but the process of pouring concrete was not considered. The process of pouring concrete involves the flow of materials, which is reflected in the interaction between the concrete and the soil.

The model contains 101,640 elements and the element type is EC3D8R (eight-node Eulerian element). The parameters of sand are the same as above, and the US-UP model is adopted by the concrete as shown in Table 2.

In Figure 4, a pile hole is reserved in the soil with a diameter of 4 cm and a depth of 50 cm, and the initial positions of the soil and concrete are set. The pile hole is divided into 25 sections from top to bottom and each section is 2 cm, in which the speed of concrete and the Eulerian boundary are set. Therefore, the simulation of the process of pouring concrete is realized by letting the concrete material be injected into the pile hole at a constant speed.

4. Results

The model is analyzed through total stress method. The calculation results given next are mainly related to the drilling model since the drilling process of CFG piles has a great disturbance to the soil. The changes of the equivalent stress $q$ (Mises stress) with the drilling depth $z$ are presented in Figure 5. $L$ is the length of model piles, and $z$ is the vertical depth of the position of drill bit (the same below). It can be seen that as the drilling depth increases, the equivalent stress around the drilling tool gradually increases. The range of stress field affected by drilling is about 9 d.

4.1. Dynamic Responses. The changes of rotation velocity of the drilling tool to adapt to the differences in soil properties lead to the complexity of the construction process during the construction of CFG piles. Four models under different velocity ratios are established and analyzed in this paper in order to illustrate the disturbance of drilling tool to the soil during drilling process. The acceleration responses of the soil are calculated, and the vibration frequencies are obtained through Fourier transform performed on the acceleration time history curve. The frequency domain curves of soil acceleration are drawn next. It can be as a reference for performing dynamic triaxial tests and shaking table tests through the analysis of the vibration frequency of soil during the construction of CFG piles.
Table 1: Parameters of silty sand.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$</td>
<td>1920</td>
<td>Density (kg·m$^{-3}$)</td>
</tr>
<tr>
<td>$E$</td>
<td>10</td>
<td>Young’s modulus (MPa)</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.25</td>
<td>Poisson’s ratio</td>
</tr>
<tr>
<td>$\varphi$</td>
<td>15</td>
<td>Friction angle (°)</td>
</tr>
<tr>
<td>$\psi$</td>
<td>0.1</td>
<td>Expansion angle (°)</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>1</td>
<td>Cohesive force (kPa)</td>
</tr>
<tr>
<td>$\epsilon$</td>
<td>0.8</td>
<td>Void ratio</td>
</tr>
<tr>
<td>$A$</td>
<td>3</td>
<td>Pore pressure parameter</td>
</tr>
</tbody>
</table>

Table 2: Parameters of concrete.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$</td>
<td>2400</td>
<td>Density (kg·m$^{-3}$)</td>
</tr>
<tr>
<td>$c_0$</td>
<td>10</td>
<td>Velocity (m·s$^{-1}$)</td>
</tr>
<tr>
<td>—</td>
<td>0.25</td>
<td>Dynamic viscosity (kg·m$^{-1}$·s$^{-1}$)</td>
</tr>
</tbody>
</table>

Figure 3: Numerical model of drilling process with mesh of the soil in detail.

Figure 4: Numerical model for concrete pumping process with mesh in detail.
The relationship between acceleration frequency and amplitude for four velocity ratios at a distance of 3d from the center of drilling tool is shown in Figure 6. The symbol h is the vertical depth of a plane, and r is the horizontal distance from pile center. It is clear that the acceleration amplitude and frequency increase with the increment of velocity ratio. When the velocity ratio is 5, the rotation velocity is 25 r/min, and the vibration frequency is about 1 Hz.

Figure 7 shows the relationship between acceleration responses of soil and radial distance. According to Figures 6 and 7, the acceleration responses of soil decrease linearly with the increase of the radial distance, and the acceleration amplitude is consistent with the vibration frequency.

As shown in Figures 6 and 7, the acceleration in the soil changes irregularly between positive and negative during the drilling process. According to the principle of mechanics, it is concluded that the soil is subjected to irregular reciprocating shear forces during the drilling process. This kind of shear vibration load will have a greater impact on the characteristics of saturated soil and then affect the structure and bearing capacity of soil. Therefore, reasonable measures should be taken to reduce the vibration effect generated during the construction process.

4.2. Changes of Pore Water Pressure. The liquefaction state of saturated soil is closely related to the change of pore water pressure. The excess pore pressure generated during the construction of CFG piles is calculated based on the stress field, the causes of which are analyzed in this section, and adds it to...
the initial static pore pressure to obtain the pore pressure during the construction of CFG piles. The liquefaction behavior of saturated sand foundation is studied by the analysis of changes for pore pressure during the construction of CFG piles.

The excess pore water pressure at different distances from the center of drilling tool is given in Figure 8 as the drilling depth is 3d, 6d, 9d, and 12 d. L is the length model piles, and z is the vertical depth of the position of the drill bit (the same below). The excess pore pressure above and below the drill bit is increasing at the beginning of drilling process due to the squeezing and vibration of the drilling tool. With the continuous increase of drilling depth, the excess pore pressure in the upper soil will decrease because of the formation of borehole while the excess pore pressure in the lower soil will gradually increase. The maximum excess pore pressure in soil around drilling tool lies within the plane of the drill bit.

The calculated pore pressure for four velocity ratios is plotted versus the drilling depth z in Figure 9. The pore pressure at the middle of pile increases first and then decreases during the whole drilling process. The specific changes of pore pressure are as follows: at the beginning of drilling process, the pore pressure remains unchanged; when the drill bit is 3 to 5 d above the middle of the pile, the pore pressure begins to increase; when the drill bit reaches the depth of L/2, the pore pressure reaches maximum value, and then it gradually decreases with the increase of drilling depth. The changes of pore pressure at the bottom of pile are similar. It should be noted that the degree of changes for pore pressure at the same depth decreases as the radial distance increases, and the pore pressure fluctuations become more obvious with the gradual increase of the velocity ratio.

Figure 10 illustrates the changes of pore pressure with the concrete pouring height at the middle and bottom of the pile. The pore pressure at the bottom increases rapidly at the beginning of the pouring, and it begins to decrease when the concrete height is 4d higher than the bottom. The pore pressure in the middle of the pile remains basically unchanged at the beginning of the pouring, and it starts to raise slowly when the concrete height is lower than the middle for 3 to 4d. When the concrete height is 3 to 4d higher than the middle, the pore pressure will rise to the maximum value and then decrease slightly.

**Figure 6: Acceleration amplitude and vibration frequency of the soil during drilling process.**
Figure 7: Acceleration amplitude and vibration frequency of the soil during drilling process.

Figure 8: The excess pore water pressure generated during the drilling process of CFG piles.
As mentioned above, there is a large excess pore water pressure generated during the construction of CFG piles in saturated sand foundation. And the closer it is to the pile core, the greater the excess pore water pressure is. According to the effective stress principle, the increase of the pore water pressure will result in the decrease of the effective stress of soil when the total stress remains the same, which will lead to the decrease of shear strength of the soil. When the excess pore water pressure is equal to the initial effective stress, the saturated sand foundation is completely liquefied, which leads to quality defects of the pile and engineering accidents. Therefore, reasonable measures such as reducing the velocity ratio and improving the form of drilling tools should be taken to reduce the value of pore water pressure generated during the construction process.

4.3. Changes of the Void Ratio. The concept of critical void ratio was first proposed by Casagrande [40] to explain the phenomenon of the liquefaction for saturated sandy soil. The changes of the density and void ratio in saturated soil are calculated in this paper during the installation of CFG piles. The soil is always saturated during the construction process based on the assumption. Therefore, the change of void ratio for the soil can be calculated by the density [41].

\[ e = \frac{n}{1 - n} \]
\[ n = \frac{\rho_s - \rho_{sat}}{\rho_s - \rho_w} \]

In these equations, \( e \) is the void ratio of the soil, \( n \) is the porosity, \( \rho_s \) is the density of soil grains, \( \rho_w \) is the density of water, and \( \rho_{sat} \) is the saturated density of the soil.

The vertical distribution of void ratio in a distance of 1 d from the drilling tool is presented in Figure 11 after the drilling process. It can be seen from the figure that the void ratio of the soil is uniformly distributed before the drilling while that is irregular parabolic distribution after the drilling. After the drilling, the void ratio of the soil changes less in the top and the bottom of the pile hole but increases significantly in the middle of the pile. The reason...
for the increase in the soil void ratio around the pile is that, during the drilling process, the rotation of drilling tool causes more soil to be discharged.

It can be concluded that, during the drilling process, the vibration effect generated by drilling tool will result in a loose state of soil which will lead to the decrease of ability to restrain the liquid concrete. Therefore, during the subsequent concrete pouring process, the pressure generated by the flowing concrete will lead to a large radial deformation of borehole, which explains the obvious phenomenon of diameter expansion in the middle of CFG piles from the side.

5. Field Test

The test was carried out in site in order to investigate the influences of the construction of long spiral CFG piles on saturated soil. The test area is located in a project of Zhengzhou North Third Ring Road. As shown in Figure 12, the diameter of CFG piles is 400 mm, the length is 12 m, and the pile spacing is 1.5 m. The excavation depth of the foundation pit was 3.5 m, and precipitation was carried out before excavation. The soil layer below the bottom of the foundation pit is shown in Figure 12. The area is filled up with a clay down to a depth of about 2.6 m followed by a layer of silty sand with a thickness of 2 m which covers an underlying clay layer. The second layer of clay is 2.9 m thick and CFG piles were drilled into the fine sand layer.

Vibrating wire pore pressure sensors were used to measure pore pressure. The principle of the sensor is to convert the collected frequency signal into pore pressure. A total of four pore pressure gauges numbered $1^\circ$, $2^\circ$, $3^\circ$, $4^\circ$ were placed in the center of the foundation pit in order to collect more comparative data. No. 1 and 2 gauges were located in the middle of the pile body while No. 3 and 4 were located on the bottom of the pile, as shown in Figure 12.

Figure 13 demonstrates the changes of pore water pressure during the construction of No. 17 pile. It can be seen from the figure that the change trend of pore pressure at the same depth is consistent during CFG pile construction. At the beginning of drilling, the pore pressure around the gauges remained basically unchanged and then began to rise. When the drill reaches the position of the gauge, the pore pressure rose to the maximum value and then began to drop. The pore pressure decreased to varying degrees at the moment when drilling started to fill the concrete. In the subsequent process, the pore pressure gradually increased until it stabilized. Apparently, the pore pressure after concrete pouring is greater than that before drilling.

6. Comparisons

6.1. Comparison of Numerical Results with Field Date.

During the drilling process, the pore pressure at a point in the soil increased first and then decreased (Figure 9), while the pore pressure at the stage of pouring concrete increased first and then remained unchanged (Figure 10). These trends are consistent with the field results (Figure 13). It is judged that the impact range of the construction is about 9 d based on the stress field. Moreover, Figures 9 and 10 also show that the excess pore water pressure at 9 d away from the pile hole is very small, which is similar to the measurements of Zhao et al. [12].

The comparison between the peak and the initial value of pore water pressure is shown in Figure 14. The initial values of pore water pressure in the middle and bottom of the pile are 2.5 kPa and 5 kPa. During the construction of the CFG pile, the peaks of pore water pressure at a distance of 3 d from the pile core are 3.1 kPa and 5.9 kPa, which increase by 0.6 kPa and 0.9 kPa, respectively, from the initial value. The growth rates are 20% and 18%. And according to the field test results, the initial values of the pore water pressure at the middle and bottom of the pile are 58 kPa and 114 kPa, while the peak pore water pressure is 65 kPa and 121 kPa. The growth rates are 12% and 6%, respectively. The error
between simulation results and field test results is between 8% and 12%. It can be considered that the simulation results are roughly consistent with the field test results.

6.2. Comparison of Numerical Results with Theoretical Calculation Results. The excess pore pressure caused by the expansion of cylindrical cavity derived by Vesic [42] is

\[
\frac{\Delta u}{c_u} = 0.817a \frac{a^2 E \sec \varphi}{2(1+v)(c_u + q \tan \varphi)r^2}. \quad (3)
\]

The excess pore water pressure at the middle and bottom of the pile after drilling process is plotted versus the normalized radial distance \( r/d \) in Figure 15. The friction coefficient \( \mu \) is 0.2 and 0.3. It is obvious that the changes of the excess pore pressure within the range of 3-9 \( d \) from the pile hole is approximately linear in the semilogarithmic coordinate system, and it is consistent with the calculated results of formula (3). When \( r < 3d \), the excess pore pressure calculated from simulation is significantly smaller than the calculated value of formula (3), and the difference has little to do with the friction coefficient. Roy et al. [43] believe that
Figure 13: Pore water pressure varies with the drilling depth during the construction of No. 17 pile. (a) 1$^\text{st}$ and 2$^{\text{nd}}$; (b) 3$^{\text{rd}}$ and 4$^{\text{th}}$.

Figure 14: Comparison between the peak and the initial value of pore water pressure.

Figure 15: Excess pore water pressure ratio $\Delta u/c_u$ versus radial distance.
this difference is mainly caused by the structural damage of the soil. The numerical calculation results in this paper show that the large shear vibration load generated during the installation is the main reason for the above difference.

7. Conclusions

(1) Based on the coupled Eulerian–Lagrangian method, a numerical model for the construction process of long spiral CFG piles including drilling and pouring concrete was established, which solved the problem that the entire construction process of CFG piles could not be simulated before.

(2) Numerical results prove that the vibration load generated during the construction of CFG piles acts on saturated soil in the form of irregular reciprocating shear forces, which is the main reason for the excessive pore water pressure and the deterioration of the soil properties.

(3) The excess pore water pressure field generated during the construction of CFG piles shows a parabolic distribution in the vertical direction, and the void ratio of soil also shows a parabolic distribution after the construction in the vertical direction.

(4) The numerical results qualitatively reproduce the results of field test very well, and they are basically consistent with the results of the excess pore pressure calculated by the cylindrical cavity expansion theory when $r > 3\, \text{d}$.

(5) In this paper, a model of CFG piles under single pile conditions is established. In the future, the impact of the construction of multiple piles on saturated soil foundation should be studied. Moreover, the construction process is divided into two stages because of its complexity. How to completely simulate the CFG pile construction process is another challenge that should be properly addressed in the future.

Data Availability

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

Conflicts of Interest

The authors declare that there are no conflicts of interest regard this paper.

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References


