The effect of thermal consolidation on bearing characteristics of static drilling rooted energy pile

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Abstract. Static drilling rooted energy pile is a new type of energy pile technology recently developed and promoted in soft soil areas. This technology has the advantages of green, low-carbon and good heat exchange effect. But its related basic theoretical research is still lacking. Therefore, the physical model of the static drilling rooted energy pile is deduced. The thermal consolidation settlement process of the soil around the pile is analyzed firstly based on one-dimensional thermal consolidation theory. And, the load transfer method is combined to derive the pile circumference under the coupling effect of load and temperature variation. The governing equations for frictional resistance and pile deformation are deduced and solved with numerical programs. Then an actual engineering case with static drilling rooted energy pile in soft soil area is calculated and analyzed. The results show that the temperature increase of the surrounding soil increases the relative displacement of pile and soil, which resulting in a decrease in the upper side friction resistance and an increase in the lower side friction resistance; and this effect will be enlarged during the cooling stage. The maximum axial addition stress caused by the temperature variation reaches 75.4% of the stress by load, so the influence of temperature changes on the bearing capacity of the pile foundation cannot be ignored. The research laid a theoretical foundation for the analysis and popularization of the static drilling rooted energy pile in coastal soft area.

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1. Introduction

In the latest Fourteenth Five-Year Plan of China, it is clearly pointed out that the energy revolution should be promoted, and a clean, low-carbon, safe and efficient energy system should be built. Thus, it has been put forward to develop and utilize geothermal energy in accordance with local conditions. The geothermal energy is a new clear energy and a renewable thermal energy from the earth. It has wide resource reserves, large distribution, and broad development prospects [1]. Ground source heat pump is a technology that utilizes the shallow geothermal energy. By pre-burying heat pumps and piping systems around the underground structure, it can realize the conversion of heat energy with the building, which greatly reducing the building energy consumption of heating and air conditioning systems. The energy pile which relies on the pile foundation as the carrier of the heat exchange tube to replace the use of ground source heat pumps, not only meets the needs of economic and energy saving, but also saves construction land [2].

In coastal soft soil areas, the traditional pile foundations have its problems as energy piles, such as mud pollution of bored piles, noise of precast piles or soft ground site disturbances. As a new type of green and environmentally friendly pile type, static drill rooted piles solve the shortcomings of the traditional precast pile construction, such as the difficulty of squeezing the soil and the difficulty of transmitting the interlayer, and it can effectively improve the compression, anti-horizontal bearing capacity [3]. Static drill rooted energy piles combine the advantages of static drill rooted piles and energy piles. Compared with traditional energy piles, they have higher bearing capacity, less mud...
discharge, lower cost, and it can avoid soil squeezing effects to ensure the quality of heat exchange tubes and so on.

There have been a few researches focusing on the static drilling rooted energy pile. Huang and Deng firstly proposed the static drill rooted energy pile construction method based on a project in Ningbo, and introduced the development process of the technology and its application case [4-7]. Li et al. adopt a self-designed model test system, analyzed the thermal response of the model in the viscous-sand double-layer foundation [8]. Wang et al. analyzed the influence of temperature load on static drill rooted energy pile based on indoor experiment and finite element model [9]. Fang carried out a 33-day in-situ test on static drill rooted energy pile in a quality inspection office building project in Ningbo [10]. They studied the changes of the pile body and soil temperature, the radial pressure around the pile, the axial stress and strain of the pile body. In fact, during the operation of static drill rooted energy pile, there is a heat exchange between the cement-soil around the pile and the soil, and the soil around the pile is simultaneously subjected to thermal-hydraulic coupling. In addition, the response of the pile and soil under temperature variation will affect the cement soil-soil interface. Therefore, it is necessary to comprehensively consider the deformation of the soil under the coupled heat-hydraulic-mechanical effect and the coordinated relationship between the cement soil and soil deformation.

Combined with the thermal-mechanical coupling effect and Terzaghi’s one-dimensional consolidation theory, a load transfer model for the static drilling rooted energy pile considering the thermal consolidation of surrounding soil is derived in this paper [11,12]. And the newly proposed model will be used to analyze and calculate an actual engineering case in a soft soil area.

2. Physical model and basic assumptions
Static drill rooted pile is a new type of pile foundation that integrates high-strength pre-stressed pipe pile, drilling, deep mixing, grouting, and pile planting. Static drill rooted energy piles use static drill rooted pile as the carrier of the ground source heat pump, after forming cement soil through spiral drilling and grouting mixing, the precast pile and heat exchange pipe are tied into the hole to form a new combined pile foundation; as shown in Fig 1. The destructive interface of static drill rooted pile is usually between cement soil and soft soil, and composite pile formed by bamboo joint pile and cement soil is regarded as an entirety. Then the load transfer method is applied to analyze the bearing characteristics of the pile foundation.
On the other hand, the heat exchange between the energy pile and the surrounding soil causes thermal consolidation of the soft soil, which affects the negative friction of the pile foundation and further affects the bearing characteristics of the pile foundation.

In order to simplify the problem, some basic assumptions are made in the following.

1. The top of the pile is fully constrained; and the bottom of the pile is partially constrained and a spring is used to simulate the reaction force in the bottom of pile.

2. The temperature change from the heat exchange is evenly distributed in the soil layer, which resulting in thermal consolidation of the surrounding soil.

3. The soil around the pile is permeable at the top and impervious at the bottom. Thus the one-dimensional consolidation analysis method of saturated soil can be used to analyze the thermal consolidation and compression of the soil.

4. The influences of piles on the thermal consolidation settlement of soil are ignored.

3. Thermal consolidation of soil

The soft soil layer around the energy pile is composed of solid soil particles and water, and the heat exchange of the energy pile will cause temperature changes in the saturated soft soil. The difference in thermal expansion between solid particles and pore water will produce excess pore water pressure, and the dissipation of thermal pore pressure over time will inevitably cause the consolidation and settlement of soft soil.

3.1. Thermal consolidation compression

According to the existing research on soil properties, the main effect of heating on soil compressibility is that the pre-consolidation pressure of the soil will decrease with the temperature arising. A widely used relationship between the pre-consolidation pressure and the temperature change, which is proposed by Cekerevac, is applied here\textsuperscript{[11]}.

$$\Delta P_{ci} = -\gamma P_{ci} \ln \left( \frac{T + \Delta T}{T} \right)$$

Fig 1. Physical model of static drill rooted energy pile
where $\Delta T$ is temperature variation; $\gamma$ is temperature-influenced parameters of pre-consolidation pressure, and usually $\gamma = 0.3 - 0.4$; $P_{ct}$ is pre-consolidation pressure of soil at temperature $T$; $\Delta P_{ct}$ is variation of pre-consolidation pressure.

Based on equation (1), combined with the $e-p$ curve method of soil settlement calculation, the calculation formula of soil settlement considering the influence of temperature $S_1$ can be obtained as following [12].

\[
S_1 = \frac{H}{1 + e_0} C_e \log \left( \frac{P_c}{P_c + \Delta P_{ct}} \right)
\]

(2)

where $S_1$ is soil settlement; $H$ is the thickness of the layer; $e_0$ is the initial void ratio of soil; $C_e$ is the soil rebound index; $P_c$ is the initial effective self-weight stress of soil.

On the other hand, the change in soil void ratio caused by temperature change can be seen as the thermal expansion and contraction of the material. The following formula is used to describe the soil deformation caused by temperature changes.

\[
\Delta e_T = -\alpha_T \Delta T
\]

(3)

where $\Delta e_T$ is the thermal strain of soil caused by heating; $\alpha_T$ is thermal expansion coefficient of saturated soil under drainage conditions ($^\circ$C$^{-1}$). It can be seen from this formula that when the temperature rises, thermal expansion of the soil occurs, followed by rebound.

From equation (3), the settlement value of the site soil due to thermal expansion and contraction of the $S_2$ soil can be obtained:

\[
S_2 = \Delta e_T H
\]

(4)

Combining equation (1)–(4), the total soil settlement $S_c$ can be obtained as following.

\[
S_c = \frac{H}{1 + e_0} C_e \log \left( \frac{P_c}{P_c + \Delta P_{ct}} \right) + \Delta e_T H
\]

(5)

3.2. Thermal consolidation process

(1) Constant temperature condition

Heating the saturated soil foundation will produce excess pore water pressure. Based on a series of experimental results, Campanella et al [13], proposed the following formula to determine the excess pore water pressure caused by heating.

\[
\Delta u_T = \frac{\Delta T \left( \left( n_v (\alpha_T - \alpha_s) + \alpha_s \right) \right)}{m_v}
\]

(6)

where $n_v$ is porosity; $\alpha_T$ is water expansion coefficient, and it will change with temperature and pressure; $\alpha_s$ is soil expansion coefficient; $\alpha_s$ is the volume change coefficient of the soil structure due to physical changes under the action of temperature; $m_v$ is volumetric compressibility of soil.

Combining with Terzaghi’s one-dimensional consolidation theory, a formula for the dissipation of excess pore pressure over time under constant temperature conditions can be obtained.

\[
u_T (z, t) = \Delta u_T \sum_{m=1}^{\infty} \frac{4 \frac{\pi m z}{2H} e^{-\frac{z^2 m^2 C_v}{4H^2}}}{\pi m}
\]

(7)

where $C_v$ is consolidation coefficient; $H$ is drainage distance; $z$ is depth.

According to equation (7), the excess pore water pressure $u_T$ at any time $t$ and any depth $z$ can be calculated. Then the degree of consolidation can be computed by the dissipation of the excess pore water pressure in the soil layer.

\[
U_t = \frac{\Delta u_{T_{max}} - u_T}{\Delta u_{T_{max}}}
\]

(8)

where $\Delta u_{T_{max}}$ is maximum excess pore water pressure generated by heating.

(2) Variation temperature condition
Equation (8) is the thermal pore pressure consolidation degree at a constant temperature, and the Terzaghi modified formula is used to describe the thermal consolidation degree under variable temperature conditions:

\[ U(z,t) = \sum_{n} U_{\text{t}_{n}} \frac{\Delta u_{\text{t}_{n}}}{} \]  

(9)

The excess pore water pressure change is divided into multiple levels in a linear manner; \( U(z,t) \) is the modified thermal consolidation degree (%), under the condition of multi-level constant velocity pore pressure changes; \( t_{n} \) and \( t_{n-1} \) are the end point and start point of each level of pore pressure change (s). The time is calculated from zero. When calculating the degree of consolidation at time \( t \) during a certain level of pore pressure change, \( t_{n} \) is changed to \( t_{n} \); \( \Delta u_{\text{t}_{n}} \) is the pore pressure (kPa) of the \( n \)th level due to temperature changes. When calculating the thermal consolidation degree at a certain time \( t \) during the pore pressure change, the corresponding pore pressure change at that time is used.

The soil around the pile is divided into \( n \) layers along the depth direction, and the total settlement of each layer is \( S_{c} / n \). By assuming the consolidation time \( t \), the degree of consolidation along the depth direction at the time \( t \) can be known from equation (10), and the settlement of each layer is equal to the consolidation degree of the layer multiplied by the total settlement of the layer

\[ S_{c}(z,t) = U(z,t) \frac{S_{c}}{n} \]  

(10)

By integrating the consolidation settlement of each soil layer, the thermal consolidation settlement of the soil ground at any time can be obtained.

4. Governing equation and solution

4.1. Governing equation

Once the relative displacement between the pile and the soil occurs, load transfer will occur at the pile-soil interface, and the relationship between the pile side friction and the pile-soil relative displacement can be expressed by the \( \tau - z \) curve. Zhao et al. introduced a concept of negative friction to improve a bilinear model [14]. The model can be used to calculate the positive and negative friction of pile side by judging the relative displacement between pile and soil. The mathematical expression is as follows:

\[ \tau(z) = \begin{cases} \tau_{\text{max}} \frac{S_{p} - S_{b}}{S_{u}} & 0 \leq S_{p} - S_{b} \leq S_{u} \\ \tau_{\text{max}} \frac{S_{p} - S_{d}}{S_{u}} & 0 \leq S_{p} - S_{d} \leq S_{u} \\ \tau_{\text{max}} \frac{S_{p} - S_{s}}{S_{u}} & S_{p} - S_{s} \geq S_{s}, S_{u} - S_{p} \geq S_{u} \end{cases} \]  

(11)

where \( S_{p} \) is pile displacement; \( S_{s} \) is soil displacement around the pile; \( \tau_{\text{max}} \) is maximum friction resistance at pile side; \( S_{c} \) is critical displacement of pile and soil.

Vijayvergiya put forward an expression to calculate the maximum friction resistance as following:

\[ \tau_{\text{max}} = \lambda \left( \sigma_{m} + 2C_{m} \right) \]  

(12)

where \( \lambda \) is coefficient of friction. Ling used a cylindrical interface model device to compare the shear strength of the three contact surfaces of precast pile-soil, precast pile cement-soil, and cement soil-soil [16]. The test showed that the shear strength will increase with the strength of cement-soil increase and increase, and the friction performance of the precast pile-cement soil interface is obviously better than that of the cement soil-soil interface. In this article, combined with the field pile test, the coefficient of friction of the cement soil and soil interface is 0.53. Additionally, \( \sigma_{m} \) is average value of vertical effective stress from surface to pile tip; \( C_{m} \) is average value of undrained shear strength of soil within the scope of the pile, and the expression is
where \( \gamma \) is soil bulk density; \( c(T) \) is cohesion; \( \phi(T) \) is friction angle. According to the existing research [17], these parameters can be calculated as follows:

\[
c(T) = 19.79e^{-0.01199T} \\
\phi(T) = 18.96e^{-0.00117T}
\]

Combined Eqs.(7)–(10), the control equation of coupled temperature transfer model can be obtained.

\[
\frac{\partial^2 S}{\partial^2 z} = \frac{\lambda U \Delta S}{A E_p} \left[ \sigma_m + 2c(T) + 2\gamma \tan \phi(T) \right]
\]

where \( \Delta S \) is the difference between \( S_p \) and \( S_s \); and the sign of this value is judged by the relative displacement of the pile and soil; \( U \) is pile perimeter; \( A \) is area of pile section; \( E_p \) is elastic modulus of pile.

4.2. Derivation

4.2.1. Mechanical response.

Tong established a load transfer model considering the temperature effect through Zuo’s hyperbola [17]. The model calculates the pile displacement when the upper part is free and the lower part is constrained by the soil. In the actual project, the energy pile did not officially start to operate during the gradual accumulation of the superstructure load, and the upper part of the pile foundation can also be considered as a free end. Therefore, at this stage, set the temperature change \( \Delta T \) is 0, and only calculate the pile displacement and friction resistance distribution under the upper load of the pile at room temperature. Combining the former, the calculation model is deduced as following.

Divide the pile into \( N \) sections. When \( N \) is large enough, there are:

\[
\frac{\partial^2 S_i}{\partial^2 z} = \frac{S_{p_{i+1}} - 2S_{p_{i}} + S_{p_{i-1}}}{\Delta z^2}
\]

where \( S_{p_i} \) is top displacement of the \( i \)-th pile unit; \( \Delta z \) is the length of the pile unit.

During the mechanical action stage, the settlement of the soil around the pile under thermal consolidation is not considered, and the displacement of the pile-soil interface is mainly the displacement of the pile, substituting Equation (17) into Equation (16)

\[
S_{p_{i-2}} - 2S_{p_{i-1}} + A_i S_{p_i} = 0
\]

where

\[
A_i = 1 - \frac{\lambda U \Delta z^2}{A E_p S_{p_{i}}} \left[ \sigma_m + 2c_i(T) + 2\gamma \tan \phi(T) \right]
\]

Through Eq.(18), a total of \( n-1 \) equations can be listed, and there are a total of \( n+1 \) unknowns. Therefore, in order to solve all the unknowns, it can be solved by introducing two boundary conditions. A dummy joint is set on the top of the pile. There is no friction on the side of the pile joint, so it is only supported by the external force \( Q \) and the top of the pile. The displacement expression of the pile section is:

\[
S_{p_0} = S_{p_1} + \frac{Q \Delta z}{E_p A}
\]

where \( S_{p_0} \) is top displacement of dummy pile joint; \( S_{p_1} \) is displacement of pile top.

It can be obtained from the force balance of pile end that

\[
Q = \pi D \int_0^L \tau(z) dz + K_i \left( S_{p_0} - S_m \right)
\]
where \( D \) is the diameter of the pile; \( L \) is pile length; \( K_c \) is base reaction coefficient; \( S_{pt} \) is pile displacement at pile tip; \( S_{sn} \) is soil displacement around pile. Combining Eqs.(18), (19) and (20), the pile deformation can be solved, and the distribution of the pile body friction resistance can be obtained by Eq.(11).

### 4.2.2. Temperature variation response.

At this stage, the loading of the superstructure has ended, and the pile foundation is in a stable state under the action of mechanics. Assuming that the temperature effect is the initial loading, the mechanical properties of the pile foundation under the temperature effect are calculated.

Combine Eqs.(11), (16) and (17), there are

When 
\[
S_{u} - S_{pt} > S_u
\]

\[
S_{pt} = 2S_{p(i-1)} - S_{p(i-2)} + B_i \Delta S_i S_u
\]

where \( \Delta S_i = 1 \) and

\[
B_i = \frac{2U \Delta z^2}{AE_p S_u} \left[ \sigma_m + 2c_1(T) + 2\gamma z \tan \phi(T) \right]
\]

when, \( 0 < S_u - S_{pt} < S_u \),

\[
C_i S_{pt} = 2S_{p(i-1)} - S_{p(i-2)} + B_i S_i
\]

where \( C_i = 1 + B_i \)

when \( S_{pt} - S_u > S_u \), the solution is the same as Eq.(22).

when \( 0 < S_{pt} - S_u < S_u \),

\[
D_i S_{pt} = 2S_{p(i-1)} - S_{p(i-2)} - B_i S_i
\]

where \( D_i = 1 + B_i \).

The upper structure has been completed in the temperature action stage, and the pile tip will not be displaced when it is in a fully constrained state, thus \( S_{pt} = 0 \).

It can be seen from the balance equation that

\[
F_{pt} = F_{pb}
\]

where \( F_{pt} \) is pile end reaction force; \( F_{pb} \) is pile bottom reaction.

According to the generalized Hooke's law, the thermal strain can be deduced as

\[
\varepsilon = \frac{\sigma}{E} + \alpha \Delta T = \frac{F_{pt}}{EA} + \alpha \Delta T
\]

where \( E \) is elastic modulus of pile; \( A \) is cross sectional area of pile. For the situation where both ends are fully constrained, \( \varepsilon = 0 \).

Therefore, the temperature internal force \( F_{pb} \) is obtained as

\[
F_{pb} = -E A \alpha \Delta T
\]

The force balance of pile bottom under the action of temperature internal force is

\[
F_{pb} = \pi D \int_0^L \tau(z) dz + K_c \left( S_{pt} - S_{sn} \right)
\]

From Eqs. (7)-(10), it can be seen that the displacement caused by thermal consolidation changes with time is nonlinear along the pile direction. Numerical method is used to calculate below.

The numerical calculation process is listed in the following.

1) Divide the pile into \( N \) elements. The displacement of the pile top is set as 0.
2) The settlement \( S_{si} \) of the soil around the pile is calculated by thermal consolidation of the soil.
3) Using the recurrence relationship, the pile displacement $S_{pi}$ and the pile side friction $\tau_{ni}$ can be obtained.

4) Bring the pile side friction resistance $\tau_{ni}$ and the pile end soil displacement $S_{sn}$ into the boundary equation (17), and $S_{pn}'$ is solved.

5) If $|S_{pn}' - S_{pn}| \leq \varepsilon$ meet the accuracy requirements, otherwise return to the first step to recalculate until the requirements are met.

5. Calculation and analysis

5.1. Engineering cases and calculation parameters

Based on an in-situ test of static drilling rooted energy pile in Ningbo \cite{17}, the bearing characteristic of this new energy pile is studied. According to survey data, the soils around the pile are mainly composed of sludge, clay and silty clay. The basic physical mechanics of each soil layer are listed in Table 1.

| Soil layer   | Soil depth | $C_v$ (cm$^2$/s) | $\gamma$ (kN/m$^3$) | $C_c$ (/) | $C_e$ (/) | $C_e$ (kPa) | $\phi$ (°) | $E_s$ (MPa) | $\epsilon_0$ (/) |
|--------------|------------|------------------|----------------------|----------|----------|------------|--------|-----------|-------------|
| Sludge       | 0-8m       | $6.0 \times 10^{-3}$ | 17.2                | 0.34     | 0.061    | 10.4       | 8.3    | 2.28      | 1.482       |
| clay         | 8-15m      | $8.5 \times 10^{-3}$ | 19.0                | 0.09     | 0.014    | 37.8       | 16.9   | 6.99      | 0.855       |
| silty clay   | 15-20m     | $4.5 \times 10^{-3}$ | 18.8                | 0.23     | 0.045    | 24.4       | 14.2   | 4.46      | 1.183       |
| silty clay   | 20-37m     | $4.2 \times 10^{-3}$ | 18.7                | 0.20     | 0.030    | 19.8       | 13.3   | 4.41      | 1.302       |
| clay         | 37-45m     | $5.1 \times 10^{-3}$ | 18.3                | 0.12     | 0.023    | 24.3       | 13.3   | 5.28      | 0.826       |
| silty clay   | 45-52m     | $7.3 \times 10^{-3}$ | 19.1                | 0.10     | 0.018    | 26.7       | 16.9   | 6.65      | 0.797       |
| silty clay   | 52-57m     | $9.6 \times 10^{-3}$ | 19.2                | 0.08     | 0.008    | 42.0       | 16.8   | 8.28      | 0.777       |

The other calculation parameters are as following. The effective pile length $L$ is 52 m; pile diameter $d$ is 500mm; elastic modulus of pile concrete $E$ is 38 GPa; the structural load at pile top surface $Q$ is 1080 kN; and $S_u$=5mm. The reaction coefficient of the pile bottom $K_c$=3000N/mm; thermal expansion coefficient of soil $\alpha_s=1 \times 10^{-5}$/°C$^{-1}$; thermal expansion coefficient of cement soil $\alpha_{c-s}=7 \times 10^{-6}$/°C$^{-1}$; the measured value of the average temperature of the pile body in the in-situ test is shown in Fig 2.

![Fig. 2. The measured temperature variation with time in energy pile](image-url)

In order to simplify the calculation process, the temperature variation is roughly divided into three stages, that is the heating stage, the constant heating stage and the cooling stage, respectively. These three stages can be described by following expresses.

\[
\Delta T = \begin{cases} 
1.63t + 20 & 0 \leq t \leq 10 \\
36.2 & 10 < t \leq 14 \\
-0.8t + 47.4 & 14 < t \leq 33
\end{cases}
\]  

(30)
5.2. Analysis

(1) the heating stage

Fig. 2 shows the variation of pile side friction and pile settlement caused by the thermal consolidation settlement of soil around the pile with the increase of time and temperature when the heat exchange tube begins to work in practical engineering. The pile top is in a constraint state, and the poor compressibility of the soil layer at the pile tip is higher than that of the upper soil layer, which can be regarded as a partial constraint on the pile tip. Therefore, in the heating stage, the expansion of the pile body due to temperature is moving along the pile top, as shown in Fig. 3 (c). Fig. 3 (a) shows the change of pile side friction under the temperature load. Compared with Fig. 3 (c), it can be seen that the settlement of some piles on the pile is small, and the displacement of soil exceeds the displacement of pile due to the increase of thermal consolidation settlement, which produces negative friction on the pile. The position of the neutral point gradually rises from 12.1 m on the first day to 20.6 m on the seventh day, and the maximum negative friction reaches 3.54 kPa. From the 7th day to the 14th day, the position of the neutral point moved up to 12 m, because the expansion displacement of the pile grew faster than the thermal consolidation settlement.

Fig. 3 (b) shows the change of pile side friction under the combined action of building load and temperature load. Under the action of only building load, the lateral friction of the pile increases gradually along the pile body, and the maximum lateral friction is 19.55 kPa. The compressibility of the clay layer in the second layer of the soil layer is better than that of the surrounding soil layer, so the lateral friction of the pile body at 16 ~ 18 m is a higher value. When the pile is under common load, the maximum average friction is at the pile end and increases with time, which is 41.25 kPa at 14 days. The side friction of the pile in the 36 ~ 52m section almost coincides from 7 to 14 days, because the expansion displacement of the pile on the pile-soil interface is greater than the thermal consolidation settlement of the soil around the pile and exceeds the critical value, resulting in the pile-soil slip. The side friction of this section is the ultimate side friction.

Fig. 3 (d) shows the variation law of axial force of pile body along depth under different heating time (0 day, 1 day, 3 day, 7 day, 14 day). When the structural load is applied only, the axial force at the pile top is the largest, and the axial force decreases gradually along the pile body, and the axial force at the pile end decreases to 0 kPa. Under the combined action of temperature load and structural load, the maximum axial force caused by thermal consolidation settlement and self-expansion of soil around the pile is always located at the top of the pile, and the axial force increases with the increase of temperature. When the pile body is heated to 14 days, the maximum additional axial force at the pile top due to temperature is 788.8 kN, which is close to the 908 kN measured by Fang [10] in the field experiment. The overall performance is the nature of the friction pile. The maximum axial force of pile top under temperature load and structural load is 1868 kN, which is 40.6 % of the ultimate bearing capacity of the pile, and it does not affect the safety of the pile foundation.
(2) The cooling stage

During the cooling stage, the pile shrinks and the expansion displacement along the different sections of the pile decreases gradually. The consolidation settlement caused by the cooling of soil will be larger than the thermal consolidation settlement caused by the heating. It can be seen from Fig. 4(a) that the position of neutral point increases from 18.4 m to 36.2 m, and the maximum negative friction resistance is 10.17 kPa.

In order to simulate the evolution law of pile side friction under long-term temperature load, a set of 120 day data to maintain the temperature of the pile at 21 degrees Celsius was added after the cooling stage. Compared with the initial temperature, the influence of this group of data on the stress and deformation characteristics of piles is similar to that of 33 d, which is because the settlement of soil around piles under thermal consolidation is nearly completed.

Fig. 4(b) shows the total lateral friction. Although the negative friction increases and the neutral point move down due to the decrease of the settlement of the pile, it can be seen from the figure that there is still no negative friction on the pile side. The total friction of the pile in the 46 ~ 52 m section is equal to the limit friction of the section, indicating that the expansion displacement caused by heating is still residual. In Fig. 4(c), the pile body gradually recovered due to the cooling expansion, and the settlement at the pile end reduced to 7.55 mm at 33 d. Figure 4(d) is the change of axial force under the combined action of load when cooling, and the maximum axial force at the top of the pile decreases from 1761.6 kN to 1136.47 kN.
6. Conclusion

Combined with theoretical and numerical analysis for an in-situ test, the influence of soft soil thermal consolidation on the pile-soil interaction of static drilling rooted energy pile is studied. The main conclusions are obtained as following.

(1) The control equations of pore water pressure, soil deformation and pile-soil interface deformation are established for the new energy pile. By comparing with an existing in-situ tests, it is proved that the model simulate well for the stress deformation characteristics of the energy pile under thermal consolidation of soil around the pile.

(2) The pile top of the simulation object is completely constrained, and the pile end is partially constrained. The expansion caused by heating is the maximum displacement of the pile top downward development at the pile end. When the temperature came to the highest value at 14th day, the displacement of pile tip was about 33.08 mm due to heating, and the thermal expansion displacement of pile rapidly decreased to 7.55 mm when the temperature was decreased. In the cooling stage, the displacement of pile end maintained constant.

(3) The temperature rise causes the thermal consolidation settlement of the soil at the pile side and the thermal expansion displacement of the pile body to change the relative displacement of the pile-soil, resulting in the decrease of the lateral friction at the upper part of the pile body and the increase of the lateral friction at the lower part of the pile body. In the cooling stage, the thermal expansion displacement of the pile begins to decrease, and the soil around the pile also increases, which results in the decrease of the upper side friction of the pile and the increase of the lower side friction.

(4) At the highest temperature of 14th day, the maximum additional axial force at the top of the pile due to temperature is 788.8 kN, which reaches 75.4 % of the superstructure load. Thus, the additional axial stress caused by temperature load cannot be ignored.

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