Field Tests on Thermal Performance of Bridge Pile Groups under Embankment Load and Cap Constraint Condition

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1. Introduction

Shallow geothermal energy is a kind of clean and sustainable energy with large reserves, no pollution, and no carbon emission [1]. The energy pile combines the technology of a traditional ground source heat pump and pile foundation, which cannot only meet the requirements of the bearing performance of the pile foundation but also realize the heat exchange between the building and the shallow geothermal energy through the circulation of heat transfer fluid in the buried pipe of the pile [2]. The pile foundation of a bridge has the characteristics of a large diameter and long pile body, and when energy pile technology is grafted onto bridge pile foundations, shallow geothermal energy can be better utilized. The deicing and snow-melting protection of bridge deck slabs in winter can be accomplished by using bridge piles to extract geothermal energy.

For the study of unconfined energy piles, Bourne-Webb et al. measured the stresses and deformations of monopiles in summer and winter, respectively, and analyzed the stress mechanism and bearing capacity calculation method of cast-in-place concrete piles under temperature [3, 4]. Cecinato and Loveridge used numerical simulation to analyze the effect of heat transfer tube length, heat transfer capacity, fluid flow rate, pile length, and pile diameter on the heat transfer efficiency of energy piles [5]. Ren et al. investigated the thermodynamic response of prestressed high-strength concrete pipe pile- (PHC-) based energy piles under cooling conditions [6]. The stress-strain characteristics of concrete cast-in-place pile body and the variation law of pile side resistance under working conditions in summer and winter were analyzed, and the influence law of structural load and pile side and pile tip constraints was discussed [7]. The structural response of the CFG energy pile under temperature load, structural load, and thermodynamic coupling was analyzed, and the influence of different heating powers on the comprehensive thermal conductivity of soil was discussed [8, 9]. The influence of pile tip constraint on the heat-induced stress and pile top displacement of concrete pile under summer conditions was studied [10, 11]. Park
et al. conducted a series of in situ thermal performance tests on large-diameter energy piles in regenerated soft soils and found that the large-diameter energy piles could provide adequate heat transfer [12].

For the study of energy piles with upper load constraints, Wang et al. conducted in situ tests on energy piles at different temperatures and various mechanical loading levels and found that the mechanical response due to temperature changes in the pile was affected by the mechanical loading level [13]. Yavari et al. investigated the mechanical behavior of an energy pile in saturated clay under thermomechanical loading using model piles, and tests under various axial loads showed greater thermally irreversible settlement at higher axial loads [14]. Yang et al. performed hot and cold cycles on energy piles with different pile top loads by developing a three-dimensional mathematical model of energy piles and found that the higher the pile top load, the greater the additional pile top settlement [15].

For the study of energy piles with bearing constraints, Laloui et al. measured and analyzed the pile stresses and deformations, the calculation method of stress induced by attached heating was proposed, and the thermodynamic coupling characteristics caused by the operation of a single energy pile in a pile-raft foundation were discussed [16, 17]. The thermal stress and cumulative deformation of the energy pile in the 2×2 or 1×5 pile group under the heating condition of a single energy pile, as well as the influence law of the operation of a single energy pile on the stratum temperature field, cap, and adjacent pile body, were analyzed, and it is proved that the group effect among energy piles can trigger the effect in the pile raft foundation [18–20]. The pile body temperature and stress variation rule of the microsteel pipe pile group with cap under different intermittent durations in winter working conditions were analyzed, and the pile foundation heat transfer performance coefficient under different operation modes was discussed [21]. The influences of pile spacing on the thermal performance of an energy pile system through numerical simulation were mainly evaluated [22].

Previous studies mainly focused on the thermal responses of energy piles without pile top constraint or with rigid cap. However, there are relatively few field tests on the thermal responses of the bridge pile group under embankment load and cap constraint. Hence, field tests on the thermal responses of the 2×2 pile group under the combined action of embankment load and temperature are carried out. A type of heat exchange tube is used to match the steel cage mounted in sections to measure the temperature and strain variation of the pile body. The thermal stress of the pile body, pile top cumulative deformation, and heat transfer efficiency per unit area are analyzed, which can provide technical support for the design and application of energy pile in bridge engineering.

2. Field Test Description

2.1. Properties of Soil Layers. The field test relies on the bridge pile foundation project of National Highway No. 310 located in Sanmenxia City, China. The pile foundation of the abutment is a reinforced concrete bored pile, the pile foundation is worked as a 2×2 group, the length of the pile is 25 m, and its diameter is 1.2 m, and the concrete strength grade is C25. From top to bottom, the soil layers are loess-like silt (0~1.8 m), gravel sand (1.8~10.1 m), silt soil (10.1~14.8 m), fine sand (medium dense 14.8~20.0 m), and fine sand (pebble bed 20.0~29.3 m). The groundwater table is 6.0 m below the ground surface. The field soil profiles are shown in Figure 1.

Seasonal changes in soil temperature during the test are represented in Figure 2. The soil temperature in the field gradually tends to be stable along pile depth, and the soil temperature in -7.5~0 m is relatively affected by seasons.
The temperature of the soil layer below -7.5 m is basically stable, and the temperature is between 16 and 18°C.

2.2. New Piping Methods and Construction Procedures. The reinforcement cage is divided into three sections; the lengths of the bottom, middle, and top sections are 9 m, 9 m, and 7 m, respectively. Therefore, the heat exchanger tubes are also arranged in three segments inside the reinforcement cage, and the three segments of heat exchanger tubes are also docked simultaneously when the cage is docked. The schematic diagram and physical drawings of heat exchange tubes on the reinforcement cage are shown in Figure 3. Compared with the traditional U-type or parallel multi-U-type buried pipe form, the buried pipe method not only overcomes the interference problem of heat exchange tube and pile foundation construction but also greatly decreases the conversion joint, reduces the risk of water leakage, and improves the survival rate of buried pipe.

During the construction of energy piles, the following issues need to be considered: (1) Due to the fact that the reinforcing cage is welded in sections on site, it is not possible to complete the piping arrangement directly inside the reinforcing cage. (2) The energy pipe cannot be arranged in the welding area, and welding is easy to damage the pipeline. (3) Multiple pipes crossing the welding area need multiple conversion heads or multiple places for hot melt butt; when any one of the pipe joints cannot withstand the pressure caused by water fracture, the energy pile will not be operational. (4) Protection of the heat exchanger tubes is required when the concrete at the pile head is broken.
A U type of buried pipe energy pile is designed. The construction steps are as follows: (1) In the steel bar factory, the welded steel cage is bound with heat transfer tubes, and the appropriate pipe spacing is selected according to the pile diameter. The pipe layout methods of the top cage, middle cage, and bottom cage are slightly different (Figure 3(a)). The top cage and the middle cage are two unconnected pipes, and the bottom cage is a conventional pipe. The pipe is on one side of the main rib and does not overlap with the main rib. When the concrete pipe is lowered, the acoustic steel pipe and the main rib can protect the energy pipe from damage by contact. (2) After hoisting and welding the steel cage, pipe butt joints are carried out. There are two conversion heads in each welding area, and four conversion heads are needed, which greatly solves the problem of multiple joints. The pipe is fixed in the welding area after cooling. (3) Protect the heat exchange tube of the pile head. The cast-in-place bridge pile of large diameter often cuts the pile head to remove the floating slurry. It is necessary to protect the 2 m depth of the heat transfer tube below the pile top; the steel tube sleeve is fixed to the steel cage and facilitates the stripping of the heat exchange tube.

2.3. Test Equipment and Sensor Arrangement. The test procedure for the thermal response test is that the circulating fluid in the holding tank is controlled at a specific temperature by a refrigeration compressor, and then a water pump delivers the cooled circulating fluid to the heat exchange tubes. After the heat exchange, the circulating fluid finally flows back to the insulation tank. The water inlet of the pump is equipped with a flowmeter in order to control the speed of the water flow in the circulating system. The whole test equipment includes a refrigeration compressor, heat-preserving water tank, circulating water pump, flow meter, intelligent temperature controller, and digital thermometer.

During the test, the cap and rib plate of the bridge foundation have been poured, and the upper subgrade filling has been compacted. Six groups of vibrating string temperature-strain sensors were arranged symmetrically and at intervals along pile depth to measure the temperature and strain inside the pile body. The sensors were located at 3.5 m, 7.5 m, 11.5 m, 15.5 m, 19.5 m, and 23.5 m below the bottom of the cap, respectively. The dimensions of the energy pile, cap, and rib plate and the layout of sensors are shown in Figure 4. The height of the filling subgrade is 4.5 m, and the density is 18.0 kN/m³. The cap and floor are made of C30 concrete, and its weight is about 280 t.

2.4. Testing Condition Design. The design of the test conditions is shown in Table 1, and four groups of field tests were carried out. The test flow rates were 0.4 m/s (268 L/h), 0.6 m/s (402 L/h), and 0.8 m/s (536 L/h), respectively.

The following reasons were taken into consideration: (1) In order to give full play to the economic benefits of energy piles, it is common for multiple piles to share one circulating pump in practical applications; for example, four piles in one cap work with one circulating pump at the same time. However, when the conventional 30~50 m head self-priming pump is running with multiple piles, the flow velocity is lower than that of a single pile. If the test flow velocity is too large, it has no practical reference value. The design flow velocity is considered to be below 1 m/s. (2) The selection of the flow velocity is related to the flow velocity of the subsequent energy pile-based bridge deck deicing system. The deicing system is difficult to operate at a high flow velocity considering practical factors such as the series of multiple piles and the span of nearly 40 m of upper and lower pipes, so the designed flow velocity is 0.6 m/s. Therefore, the energy pile test is also selected near the velocity, which is 0.4~0.8 m/s.

3. Results and Analysis

3.1. Stress and Temperature of Pile Body. Figure 5 shows the variation of pile stress with pile depth for winter and summer conditions.

The stresses in the pile body under winter conditions do not differ significantly from each other, and there is no sudden change in stress in both the concentrated and nonconcentrated areas of the heat exchanger tubes, whose maximum difference in tensile stresses is about 0.2 MPa. In summer condition, stress changes obviously along the pile depth, and the maximum compressive stress difference between the concentrated area and the nonconcentrated area is about 0.5 MPa. The pile stress in summer condition appears to be compressive stress, and the maximum stress difference between the concentrated area and nonconcentrated area of piles is about 2% of the compressive strength of concrete (the compressive strength of C25 concrete is 25 MPa), so the pipe arrangement has almost no effect on the pile structure. Due to the construction schedule, the test without an upper load was carried out in the summer, and the temperature difference between the inlet (40°C) and the pile (16~27°C) was large, so the thermal stress was relatively large. At the same time, the test with the upper load was carried out in winter; the temperature difference between the inlet (5°C) and the pile (below 16°C) is relatively small, resulting in relatively small thermal stress. Under summer conditions without an upper load, the thermal stress of the pile is 3.1 MPa, or about 12.4% of the compressive strength of concrete. When the energy pile is in normal use, the pile foundation is subjected to the action of the thermal load and the upper load at the same time, and both the thermal load and the upper load compress the pile foundation, resulting in the superposition effect, so the thermal stress should be considered in the structural design. In winter working conditions, the maximum thermal tensile stress of a pile is 0.4 MPa, which is about 22.4% of the tensile strength of concrete. Since this part is tensile stress, it does not have a superposition effect with the upper load and can slow down the service strength of the pile foundation, so the operation of the energy pile has little influence on the structure in winter working conditions. However, for a pile foundation without an upper load, the thermal stress of this part should be considered.

The temperature distribution of the pile body under different flow rates is shown in Figure 6. From the figure, it can be seen that with the increase in time, the temperature of the pile body decreases and stabilizes around 72h, which
indicates that the cooling effect of the circulating fluid and the heat transfer from the surrounding soil to the pile body have reached an approximate equilibrium. The temperature change of the pile body at -7.5 m and -11.5 m is most obvious, while the temperature change at the bottom of the pile is small, which is presumed to be due to the circulating fluid mainly exchanging heat in the middle and upper parts of the pile body, and there is not much difference between the temperature of the circulating fluid and the temperature of the pile body when it reaches the bottom of the pile body. With the increase in flow velocity, the temperature change of the pile bottom is not obvious, while the temperature change of the pile top is the largest, and the temperature of the pile body is the lowest when the flow velocity is 0.8 m/s. When the inlet water temperature is controlled at about 5°C, the water temperature is the highest when the flow rate is 0.4 m/s, and the water temperature is the lowest when the flow rate is 0.8 m/s. This is because the faster the flow rate

### Table 1: Test conditions.

<table>
<thead>
<tr>
<th>Pile top load</th>
<th>Cyclic mode</th>
<th>Inlet temperature (°C)</th>
<th>Flow rate (m/s)</th>
<th>Running time (d)</th>
<th>Test name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment load</td>
<td>Winter mode</td>
<td>5</td>
<td>0.4</td>
<td>7</td>
<td>Test 1</td>
</tr>
<tr>
<td>Embankment load</td>
<td>Winter mode</td>
<td>5</td>
<td>0.6</td>
<td>7</td>
<td>Test 2</td>
</tr>
<tr>
<td>Embankment load</td>
<td>Winter mode</td>
<td>5</td>
<td>0.8</td>
<td>7</td>
<td>Test 3</td>
</tr>
<tr>
<td>No load</td>
<td>Summer mode</td>
<td>40</td>
<td>0.6</td>
<td>7</td>
<td>Test 4</td>
</tr>
</tbody>
</table>

Figure 4: Diagrams of subgrade load and cap size: (a) stereoscopic, (b) cross-section, and (c) longitudinal section.
is, the less heat exchange in the pile body is sufficient. When
the circulation is stabilized, the effluent temperatures at
0.4 m/s, 0.6 m/s, and 0.8 m/s are 13.2°C, 12.5°C, and 11.4°C,
respectively, and the temperature difference between the
inlet and outlet is 7.3°C, 6.9°C, and 6°C, respectively.

3.2. Thermal-Induced Axial Force of Pile. The axial free
strain $\varepsilon_{T\text{-free}}$ caused by the change of pile temperature can
be calculated by the following:

$$\varepsilon_{T\text{-free}} = \alpha \Delta T,$$

where $\alpha$ is the linear expansion coefficient of reinforced
concrete, and its value is $10 \mu e/\circ C$ [22], and $\Delta T$ is the
temperature change value of the pile body.

In actual engineering, due to the constraints of the soil
around the pile and the upper load, the additional axial temper-
ature stress $\sigma_T$ is caused, and $\sigma_T$ can be calculated as follows:

$$\sigma_T = E(\varepsilon_T - \alpha \Delta T),$$

where $E$ is Young’s modulus of the pile, Young’s modulus of the
C25 pile is 28 GPa, and $\varepsilon_T$ is the actual strain of the pile.
measured during the test. It is agreed that the compressive stress is positive and the tensile strain is negative.

After the subgrade soil is compacted and stabilized, the axial force distributions of the pile shaft under winter conditions are shown in Figure 7 (test 1). The figure shows that the temperature of the outlet at the beginning of the test is 15.5°C, and the axial force distribution of the pile at this time is the distribution of the axial force of the pile under the subgrade load. When the test was conducted for 1, 4, and 7 days, the outlet temperature decreased by 1.5°C, 2.5°C, and 3°C, respectively. It can be seen that the outlet temperature changed gradually and slowly as the test was conducted. When the outlet temperature began to decrease, the pile body produced temperature additional tensile stress. The change of axial force at the top and bottom of the pile body was not obvious with time, while the change of axial force in the middle of the pile body was relatively obvious. When the outlet temperature decreased by 1.5°C, 2.5°C, and 3°C, the axial force at -11.5 m of the pile body decreased by 113 kN, 109 kN, and 71 kN, respectively.

Figure 8 presents the thermal axial force distributions of the pile body under different testing conditions. The thermal axial force increases the most in the early stage, and its increasing trend gradually slows down with the testing time increasing; the thermally induced axial force change at the top and middle of the pile body was relatively obvious. When the outlet temperature decreased by 1.5°C, 2.5°C, and 3°C, the axial force at -11.5 m of the pile body decreased by 113 kN, 109 kN, and 71 kN, respectively.

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3.3. Thermal-Induced Displacement of Pile Top. During the operation of the energy pile, the thermal expansion and cold contraction of the pile body are caused by the temperature change of the pile body, and the displacement of the pile top and pile end is changed. According to the axial force
Figure 8: Continued.
analysis of winter working conditions, it can be seen that the pile bottom displacement changes little while the pile top displacement changes greatly, so the pile top displacement is mainly calculated. The pile top displacement is estimated according to the strain measured by the pile body. Meanwhile, since the construction period is long, the change in ground temperature will interfere with the displacement calculation. However, the short-term ground temperature...
change during the test time is very small, and its influence is not considered, as shown in the following:

$$s = -\sum (\varepsilon_i \Delta l_i),$$  \hspace{1cm} (3)

where $s$ is the displacement of the pile top, the settlement of the pile top is positive, and the uplift is negative; the pile body is divided into several equal strain pile sections along the length, $\Delta l_i$ is the length of each pile section, and $\varepsilon_i$ is the measured strain of each pile section.

Figure 10 shows the variations of pile top displacement with the testing time. The negative direction of the $x$-axis is set to the number of days the load is applied (unit: d); the positive direction is set to the energy pile test time (unit: h). The early stage of the test is the embankment construction stage. As the embankment load increases, the pile top displacement increases accordingly. After the embankment load stabilizes, the pile top displacement reaches 1.0 mm. Under cooling conditions, the temperature of the pile body decreases slowly, and the displacement of the pile top gradually increases. Then, the temperature drop of the pile body gradually stabilizes, and the displacement of the pile top also gradually stabilizes, and finally the pile top displacement reaches 0.33 mm (0.27% of pile diameter).

Figure 11 presents the pile top displacement variations of each field test, and the pile top displacement changes of the bored pile under different conditions are mainly compared. In both literature [18] and this paper, a $2 \times 2$ cap pile foundation is used. The test in literature [7] was a single-pile foundation. Load types are divided into structural dead load and short-term load. The low cap in literature [18] and the bridge cap in this paper are both structural dead load. The single-pile foundation in literature [7] is short-term loading. The pile body stress was relatively larger because the short-term loading and field test were carried out simultaneously in literature [7], and it resulted in more obvious changes in pile top displacement. When the test is carried out under the structure’s dead load, the load tends to be stable with time, and the pile stress caused by the load is not calculated separately, so the pile top displacement changes relatively slowly. The comparison between the dead load test of the cap pile group in literature [18] and the no-load test of this test shows that the pile top displacement is larger when the pile top is not loaded. At the same time, by comparing the pile top displacement of this test in winter and summer, it can be found that the pile body displacement reaches -0.98 mm in the summer working condition and 0.33 mm in the winter working condition under the constant load constraint of the roadbed.

For the constrained pile group with cap, the rigid body is formed by the cap connecting pile groups, and the displacement of the pile top caused by the movement of some energy piles will be accompanied by the displacement of adjacent piles and diagonal piles. When one energy pile is operated separately, the pile top displacement has the greatest effect on the diagonal pile, and the adjacent pile has the second effect.

Figure 12 shows the variations of the pile top displacement with the average temperature increasing or decreasing.
It is assumed that the increase or decrease of the average temperature of the pile conforms to the principle of concrete thermal expansion, and the same thermal strain values are generated everywhere in the pile.

Comparing the distributions of the pile top displacement in winter and summer, the pile top displacement increment in the early period of the summer condition is larger, while that of the winter condition is smaller. This indicates that the restraining effect of the cap is smaller under the summer condition and greater under the winter condition. The increase in the pile top displacement in the later period of the summer condition is reduced, which indicates that the restraining effect of the cap is increasing. When the pile body is thermally expanded and the pile top produces upward displacement, the cap cannot fully restrain it at first, but it needs to wait until the reaction force of the adjacent pile and the diagonal pile reaches a certain point before a better restraint effect can occur. When the pile body undergoes cold contraction, the pile top produces downward displacement. Due to the large contact surface between the cap and the foundation soil, its restraint effect is immediately effective, and then the adjacent piles and the corner piles are jointly restrained. Therefore, the restraining effect of the cap in the early stage of the winter condition is greater than that of the summer condition, but may be slightly smaller in the later stage.

3.4. Thermal-Induced Tensile Stress. Comparisons of thermal tensile stresses versus temperature change in winter conditions are shown in Figure 13. For the CFG pile tested by You et al. [8, 9] in Beijing, the relationship between the maximum additional temperature stress of the pile body and the temperature change is $\sigma_T = 111\Delta T$, and it is located at about 2/3 of the pile length (at 13 m depth of the pile body). Ren et al. [21] tested the microsteel piles in Jiaozuo, Henan Province, and found that the relationship between the maximum additional temperature stress and temperature change of the pile body is $\sigma_T = 203\Delta T$, and the maximum stress is located at 2/5 of the pile body (at the pile body of 5.7 m); Amatya et al. [23] tested the pipe pile in London, England. The relationship between the maximum additional temperature stress of the pile body and the temperature change is $\sigma_T = 177\Delta T$, and it is located at about 2/3 of the pile length (at the pile body 15 m). For the large-diameter cast-in-place pile tested in this paper, the maximum additional temperature stress temperature change relationship is $\sigma_T = 162\Delta T$; it is located at 3/5 of the pile body (at 15.5 m of the pile body).

Through the analysis of different pile types, it can be seen that the thermal tensile stress of the CFG pile body tested by You et al. [8, 9] in Beijing has a relatively small response coefficient to temperature. The reason may be that the pile is not subject to load constraints. Compared with the miniature steel pipe piles tested by Ren et al. [21] and the additional temperature stress of the PHC piles tested by Amatya et al. [23], the response coefficient to temperature is smaller. The digging and filling piles are close to each other. This is due to different pile types. The response coefficient of thermal stress to temperature generally increases under the influence of load. At the same time, the temperature stress response coefficient of pile caps in this paper due to temperature changes will also be constrained by the rigidity of the bridge pile cap.

From the heat-induced stress of the energy pile of different pile types, it can be seen that the response degree of different pile types to the temperature change of the pile body is slightly different. The pile [7] is relatively close to the cast-in-place pile in this test. Therefore, in actual engineering, the influence of pile body thermal stress on the pile structure should be fully considered for different pile types.

3.5. Side Resistance. The pile body shrinks when subjected to cold, which causes a relative displacement of the pile body with the surrounding soil, which in turn generates lateral frictional resistance. From the stresses obtained from two
adjacent layers of strain gauges, the average pile side friction resistance is:

$$f = \left( \sigma_{T,j} - \sigma_{T,j-1} \right) D / 4 \Delta L,$$  \hspace{1cm} (4)

where $D$ is the pile diameter, $\Delta L$ is the distance between two adjacent layers of strain gauges, and $j = 1, 2, 3, \ldots$ denotes the position of strain gauges from the top of the pile to the bottom of the pile. Provide that the friction is positive upward and negative downward.

Figure 14 shows that the initial outlet temperature equals 15.5°C and gradually decreases during the process of temperature drop ($\Delta T_c$ is the temperature drop of the water outlet). The $\Delta T_c$ is -3°C after 7 days. Under winter conditions, the pile body shrinks from both ends to the middle, and the upper part of the pile body moves downward relative to the surrounding soil, so the lateral resistance of the upper part of the pile body increases, while the middle and lower
parts of the pile body move upward relative to the surrounding soil, so the lateral resistance of the middle and lower parts of the pile body decreases; the lateral friction resistance at the bottom of the pile body is the smallest, probably due to the restraining effect on the bottom of the pile body; however, the lateral resistance of the whole pile body does not reach the negative value.

4. Analysis of Heat Transfer Performance

4.1. Heat Exchange and Heat Transfer Efficiency. Through the inlet and outlet water temperatures $t_{in}$ and $t_{out}$, the specific heat capacity $c_w$ of the thermal fluid, and the mass flow rate $m_w$ converted from the flow rate under various working conditions, the heat transfer power $Q$ is obtained. According to the pile diameter, the heat exchange amount is divided into $q_L$ and $q_A$. The heat exchange amount per meter unit length of the energy pile with a pile diameter of 0.3–0.5 m is expressed in $q_L$, and for the energy pile with a pile diameter greater than 0.6 m per square meter of pile-soil contact area, the amount of heat exchange is represented by $q_A$.

The equations of the heat exchange $q_L$ and $q_A$ of the energy pile are as follows:

$$Q = c_w m_w (t_{out} - t_{in}),$$ \hspace{5cm} (5)

$$q_L = \frac{Q}{L},$$ \hspace{5cm} (6)

$$q_A = \frac{Q}{A},$$ \hspace{5cm} (7)

where $Q$ is the heat transfer power (W), $q_L$ is the heat transfer per unit length (W/m), $q_A$ is the heat transfer per square meter of pile-soil contact area (W/m²), $c_w$ is the heat of the heat transfer fluid capacity (J/(kg·K)), $m_w$ is the mass flow rate of the heat exchange fluid (kg/s), $t_{out}$ is the temperature of the circulating fluid at the outlet, $t_{in}$ is the temperature of the circulating fluid at the inlet, $L$ is the length of the heat exchange body (m), and $A$ is the effective area of pile-soil contact (m²).

Combined with the actual layout of the heat exchange tubes on the steel cage, the actual length of the pipe layout in this paper is about 18 m, and the effective ground-soil contact area is 67.8 m². The inner diameter of the test heat pipe is 15.4 mm, and the mass flow rate $m_w$ converted from the three flow rates of 0.4 m/s (268 L/h), 0.6 m/s (402 L/h), and 0.8 m/s (536 L/h) are 0.0745 kg/s, 0.1118 kg/s, and 0.1490 kg/s, respectively. Multiply the hourly inlet and outlet temperature difference ($t_{out} - t_{in}$) by the thermal capacity $c_w$ of the thermal fluid and the mass flow rate $m_w$ under this working condition to obtain the hourly heat transfer power $Q_j$ under this working condition, and calculate one day. The internal average heat transfer power, and then calculate the heat transfer $q_A$ per square meter of pile-soil contact area in a day, as follows:

$$Q_j = c_w m_w (t_{out-j} - t_{in-j}),$$ \hspace{5cm} (8)

$$Q = \frac{1}{j} \sum_{j=1}^{J} Q_j,$$ \hspace{5cm} (9)

$$q_A = \frac{Q}{A},$$ \hspace{5cm} (10)

where $Q_j$ is the heat transfer power per hour (W), $q_A$ is the heat transfer per square meter of pile-soil contact area (W/m²), $c_w$ is the heat capacity of the heat transfer fluid.
The cast-in-place pile soil is 67.8 m². After calculation, the total temperature difference is 18 m, and the effective contact surface of the two piles is close. The length of the CFG pile [8, 9] shows that the heat transfer efficiency of a different pile type is lower than that in summer, which is mainly related to the difference in temperature between the heat transfer fluid and the pile body in winter and summer. The temperature difference $T$ between the outlet and inlet of the large-diameter bored pile in summer is in the range of $-14°$ to $16°$C, while the temperature difference in winter is in the range of 7° to 8°C. Therefore, when other variables are the same, the greater the temperature difference between the thermal fluid and the pile body, the greater the heat transfer.

In the summer condition, the average heat exchange of CFG piles [8, 9] and large-diameter cast-in-place piles is $q_L = 120$ W/m and $q_A = 107$ W/m² ($q_L = 286$ W/m). It shows that the heat transfer efficiency per unit length/area of the two piles is close. The length of the CFG pile [8, 9] is 18 m, and the effective contact surface of the large-diameter cast-in-place pile soil is 67.8 m². After calculation, the total heat transfer power of the large-diameter cast-in-place pile (7254 W) is about 3.3 times that of the CFG pile (2160 W) [8, 9]. The heat exchange $q_L$ per meter of pile length for large-diameter cast-in-place piles is about 2.38 times that of CFG pile.

In winter conditions, the curves of the miniature steel pipe pile and the large diameter cast-in-place pile are close, $q_L = 60$ W/m and $q_A = 58$ W/m² ($q_L = 155$ W/m), and the total heat exchange power of the large-diameter cast-in-place pile is 4068 W, which is approximately. The total heat transfer power (780 W) of the microsteel pipe pile is 5.2 times; compared with $q_L$ per meter of pile length, the large diameter cast-in-place pile is about 2.58 times that of the microsteel pipe pile. This is because the CFG piles [8, 9] and microsteel piles [21, 23] have shorter pipe lengths, and the shallow ground temperature energy utilization rate is relatively low.

The heat transfer efficiency of different pile types is compared with the total heating power ($W$), the heat transfers per meter of pile length $q_L$, and the heat transfer per square meter of contact area $q_A$, and it is found that $q_L$ alone cannot clearly reflect the total heating power ($W$). Under the test comparison of different pile diameters, the total thermal power ($W$) contrast ratio is very different from the $q_L$ contrast ratio. Although the total thermal power can clearly reflect the heat transfer efficiency of an energy pile, it is not enough to simply consider the $q_L$ factor to estimate the heat transfer characteristics (design reference value) of an energy pile without actual measurement data. The comparison shows that in the case of large differences in pile diameters, the comparison of $q_L$ is much smaller than the comparison of total thermal power ($q_L$ is 2.38 times, $Q$ is 3.3 times, $q_L$ is 2.58 times, and $Q$ is 5.2 times), and other reference factors are needed to supplement. This paper suggests that when specifying the reference value of energy pile design, the effective pile length $L$, effective pile-soil contact area $A$, pipe length, circulating flow rate, pile-inlet temperature difference, and other factors should be considered. When the pile diameter is small, the pipe layout is relatively

![Figure 15: Distributions of heat exchange per meter pile length versus time.](image-url)
small, and only the effective pile length can be considered; when the pile diameter is large, the pipe layout in the pile body is longer, and a regional pipeline has been formed. At this time, only the length in the direction of the pile body is considered. It is definitely not feasible, and the calculation and design should be carried out according to the actual dense pipe area. Figure 15 also shows that the time for heat exchange to reach stability is mostly between 48 h and 72 h. The CFG pile [8, 9] has a smaller change after 48 h, but there is a slight rise and fall; the heat exchange of microsteel piles [21, 23] in winter and summer is obviously stable at 48 h, and then the change in heat exchange decreases slightly. The heat exchange in winter tends to be stable at 48 h, and the heat exchange in summer tends to be stable at 72 h. According to the analysis of various scholars, the heat exchange rate is close to stable in the first and mid-term of the test, and
then the change is small. Most of the studies begin to show steady and slow changes after 48 h~72 h. For the test with a small pile temperature difference, the heat exchange stability has been advanced. The inlet-pile temperature difference in winter is lower than that in summer, while the heat exchange in winter tends to be stable at 48 h and the heat exchange in summer tends to be at 72 h, and the stable time of heat exchange in winter is shorter than that in summer.

The comparisons of the heat exchange performance of cast-in-place energy piles are presented in Figure 16. For parallel double-U cast-in-place piles, the temperature difference between inlet and outlet water is 2°C, the heat exchange is 3400 W, and the mass flow rate is 0.5 kg/s. For parallel double-helix cast-in-place piles, the temperature difference between the inlet and outlet water is 3°C, heat exchange is 2600 W, the mass flow rate is 0.4 kg/s. For this test, the temperature difference between inlet and outlet water is 14°C, heat transfer is 7254 W, and the mass flow rate is 0.111 kg/s. The temperature difference in literature [24] is small, which cannot give full play to the heat exchange performance of cast-in-place piles. However, the heat exchange performance of the new U-type buried pipe cast-in-place piles is better than that of the conventional parallel double-U cast-in-place piles and parallel double-spiral cast-in-place piles.

4.2. COP. The coefficient of performance (COP) of energy pile is given by

$$\text{COP} = \frac{Q}{P},$$

where $Q$ is the heat transfer power (W) and $P$ is the energy consumption of the energy pile system, which is measured by the cumulative electric power meter.

The comparisons of COP between the microsteel pipe pile and the large-diameter cast-in-place pile are shown in Figure 17. The time of both tests is all in the winter, and the flow rate is about 0.6 m/s. The COP value is calculated according to the obtained heat exchange power and power consumption. Since the heat transfer power of large-diameter cast-in-place piles is greater than that of microsteel pipe piles and the power consumption of the two is not much different, the COP value of large-diameter cast-in-place piles is greater than that of microsteel pipe piles; that is, the heat exchange performance of large-diameter cast-in-place piles is stronger than that of micro-steel pipe piles. Among them, the average COP of microsteel pipe piles is about 1.25, and the average COP of large-diameter cast-in-place piles is about 3.0, which is 2.4 times that of steel pipe piles.

5. Conclusions

Based on the project of national highway No. 310 in Sanmenxia City, China, field tests on the thermal responses of the $2 \times 2$ pile group in summer and winter conditions were carried out. The heat transfer performance under the new heat exchanger tube arrangement is analyzed, and the heat transfer calculation method for large-diameter energy piles is summarized. In these field test conditions, the following conclusions can be obtained.

1. For the type of piping, there is no sudden change of stress in the concentrate and nonconcentrate areas of pile in winter, and the stress in the concentrate and nonconcentrate areas of pile in summer changes slightly, which is about 2% of the compressive strength of C25 concrete, and does not affect the stability of pile structure. The pile thermal stress has little influence on the structure under winter condition, while the pile foundation shows a compressive state because of the superposition effect of both the thermal load and the upper load under summer condition; therefore, this part of thermal stress should be considered in structural design.

2. Under the combined action of temperature load and embankment dead load, the maximum stress induced by temperature is about 0.4 MPa, which is located at 3/5 of the pile depth (about 15.5 m) after 7 days of winter working condition. The relationship between stress and temperature is as follows: $\sigma_T = 162\Delta T$. The energy pile top displacement increases by 0.33 mm, which is only about 0.27% of pile diameter, and does not affect the safety of the pile group.

3. The heat exchange per unit length and unit area of the energy pile increases with the temperature difference increasing. The heat exchange in winter is lower than that in summer, gradually decreases with the cycle time increasing, and finally stabilizes. For energy piles with large diameter bored piles, the average heat exchange per square meter of pile-soil contact area in summer ($\Delta T = -14 \sim -16^\circ C$) and winter ($\Delta T = 7 \sim 8^\circ C$) is 107 W/m² and 58 W/m², respectively.

As a whole, the heat transfer performance and thermal response of large-diameter bridge energy piles under the action of embankment loads and temperature loads are investigated, and the feasibility of a heat transfer pipe laying method applicable to segmented steel cages is analyzed. However, the application of bridge energy piles also needs to consider the influence of bridge structure loads and vehicle loads in actual engineering, and the application of bridge energy piles is also worth exploring and studying.

Data Availability

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

Conflicts of Interest

All authors declare that there is no conflict of interests regarding the publication of this article.
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