Research Article

In Situ Test Research on Friction Resistance of Self-Anchored Test Pile

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The traditional static load test method has been considered as the most direct and reliable method to determine the bearing capacity of single pile, but it has some disadvantages, such as inconvenient operation, laborious test, high cost, and being time-consuming. In this paper, a new type of pile testing method, self-anchored pile testing method, was proposed, and the in situ test was carried out for the first time. This method allows the upper and lower piles to provide force to each other and does not occupy other construction spaces. It had the advantages of simple operation and being economical and practical. Based on the Q - w curve, axial force distribution curve, and hyperbolic function model of load transfer, this paper studied the evolution law of friction of self-anchored test pile and the load transfer process of self-anchored test pile. The results show that the load transfer process of self-anchored pile-soil interface can be divided into three stages: elastic, elastic-plastic, and limit state. The friction of the upper and lower piles starts from the bottom of each pile and then gradually increases. The soil around the upper and lower piles gradually undergoes nonlinear deformation and shear failure, and the pile soil reaches the yield state. By analyzing the hyperbolic function model of load transfer, it shows that the hyperbolic function model can be better applied to the self-anchored test pile, which has reference value for the selection of the function model of self-anchored test pile in the future.

1. Introduction

With the development of economy, there are more and more high-rise buildings, and pile foundation has become particularly significant [1]. For a long time, the static load test method has been considered as the most direct and reliable method to determine the bearing capacity of single pile. However, static load test also has many well-known shortcomings, such as troublesome to operate, high cost, and being time-consuming [2–4]. If the bearing capacity of pile is used to provide force to each other, the above problems can be solved simply and effectively. This method is called self-anchored test pile method. Self-anchored test pile is a new kind of test pile; place a jack on the top of the upper pile, that is, on the ground, set a vertical through hole in the upper pile, and place a load transfer rod in the vertical through hole when the pile is tested. The steel rebar of the upper pile is anchored on the antiforcing beam, and the jack applies load to the upper and lower piles simultaneously through

the load transfer rod. The upper end of the load transfer rod is against the bottom of the jack, and the lower end of the load transfer rod is connected with the cover plate of the lower pile. When testing pile, place load transfer rod in the vertical through hole. Let the load transfer rod be against the top of the lower pile, and transfer the jack load to the lower pile. Vertical load is provided by jack placed above the ground. The top of the upper pile is pulled because of the anchorage reaction, while the lower pile is pressed by the jack through the load transfer rod. The top of the upper pile is tensioned, and the top of the lower pile is under pressure. The tensile bearing capacity of the upper pile and the compressive bearing capacity of the lower pile can be measured. The schematic diagram of self-anchored test pile is shown in Figure 1.

Compared with other test piles, not only is self-anchored test pile suitable for heavy loads foundation pile test, but also it does not need to set up anchor piles and additional operating space. It is easy to operate and does not need to be

FIGURE 1: Schematic diagram of self-anchored test pile.

loaded. In addition, the jack is located on the ground to ensure the verticality of the test pile during loading. The jack can also be exchanged according to the requirements of load size, which is convenient and simple. When the jack, oil pump and oil pressure pipe leak oil, the maintenance is more convenient. The jack and load transfer rod can also be recycled, which can save the test cost.

The load transfer method was first proposed by Seed [5], which is the most widely used method in bearing capacity of single pile. The load transfer method can divide the load transfer function into pile side load transfer function and pile end load transfer function from the classification of pile body position. Xiao [6] proposed an approximate formula for calculating the parameters of hyperbolic model to describe the nonlinear relationship between shear stress and shear displacement of soil around piles. Nie and Chen [7] used hyperbolic function model and then modified parameters a and b in the formula, and the relationship curve between pile top load and displacement was determined. Dang et al. [8] compared the hyperbolic load transfer function with the actual project, and the simulation results were in good agreement with the actual situation. Tang et al. [9] proposed a more accurate hyperbolic parameter distribution function based on the theory of Copula function. Cui et al. [10] used hyperbolic model to simulate the excavation foundation in weathered rock, and the simulated values were in good agreement with the experimental values.

Up to now, hyperbolic function model has been applied in uplift pile and top pressure pile, and the coincidence degree is high under different geological conditions. However, whether the self-anchored test pile is suitable for hyperbolic function model is still unclear. In this paper, through the in situ test, the pile friction resistance-pile-soil displacement curve of the self-anchored test pile is obtained, and then the linear fitting is compared with the in situ test results to study the regular characteristics of load transfer of the self-anchored test pile.

2. General Situation of In Situ Test

2.1. Engineering Geology of Test Site and Parameters of Test Pile. The in situ test site is located in the south of Jiangsu Province, China, and the geological conditions are shown in Table 1.

The length of self-anchored test pile in this test is 4.3 m; in order to paste strain gauges on the inner side of the pile, the pile is divided into several pile sections with different lengths. In order to obtain the axial force of the test pile under load, strain gauges are pasted at different positions of the test pile depth. The strain gauge model is BFH-120-3AA metal strain gauge. The sensitive grid size of strain gauge is $3\text{ mm} \times 2\text{ mm}$, with nominal resistance of $120\ \Omega$. The sensitivity coefficient is $2.08 \pm 1\%$, and accuracy class is A. In order to prevent the strain gauge from being affected by water in the ground, the epoxy adhesive is sealed first. After a few days, the epoxy adhesive is completely dried, and then silica gel A and silica gel B were applied on the surface of the strain gauge in the ratio of 4 : 6. The position of strain gauges in pile is shown in Figure 2. After the strain gauge is pasted on each pile, each pile is welded into a pile, which is the upper pile. The splicing process of each steel pipe pile section interface is made of four welded steel rebars, and the length of welded steel rebar is 160 mm. Then XQ injection adhesive anchors glue is applied on the joint surface of steel pipe. The pile consists of a section of 0.5 m, three sections of 0.8 m, and a section of 1.4 m steel pipe pile. The fabrication of self-anchored test pile is shown in Figure 3. The depth of self-anchored test pile in soil is 3.4 m. The self-anchored test pile consists of two sections, the length of the upper pile is 3.8 m, and the length of the lower pile is 0.5 m. The position of the equilibrium point is calculated by static penetration in geological exploration. The dividing point of upper and lower piles is called the equilibrium point when the friction of the upper pile = the friction of the lower pile + the end resistance.

The elastic modulus of the self-anchored test pile is 206 GPa, the diameter of the pile is 108 mm, and the thickness of the pile wall is 4 mm. The diameter of the load transfer rod is 76 mm.

The data of the strain gauge of the test pile is collected by the DH3820 high-speed static strain data collection instrument. The sampling frequency of the high-speed static strain data collection instrument is 100 Hz, which can accurately collect and record the varying signal in the test and ensure the accuracy of the data collected in the test.

2.2. Introduction of Test Steps. After the test pile is made, the pile is pressed into the planned position. Load after 36 days [11] and place the pressure sensor above the jack for accurate recording. The test adopts the method of equal and gradual loading. The test is divided into 15 levels for loading. According to the data obtained by static cone penetration test during geological exploration, the ultimate bearing capacity is estimated to be 30 kN. According to the technical code for testing of building foundation piles [11], the load of

TABLE 1: Distribution of soil layer at test pile location.

Soil layers	Geotechnical name	Soil depth (m)	Characteristic value of side friction resistance (q_{si}/kPa)
①	Silty clay	0.9	25
②	Muddy silty clay	2.0	18
③	Clay	2.4	65

FIGURE 2: Schematic diagram of strain gauges position of self-anchored test pile.

FIGURE 3: Pile segment splicing photos. At the bottom right of the picture is the pasting diagram of strain gauge on the inner wall of the test pile.

each level is 1/10 of the estimated ultimate bearing capacity of the pile, and the load of each level is increased by 2.00 kN. It should be noted that the load of the first stage is twice the

average load. Loading should be carried out according to the slow maintaining load method. After the load is applied, the pile top displacement should be recorded at 5, 10, 15, 15, and

15 min intervals in the first hour and at 30 min intervals after one hour. In two consecutive hours, the settlement of pile is less than or equal to 0.1 mm per hour. It is considered that the settlement of test pile is relatively stable, and the next level of loading can be carried out. The test photos are shown in Figure 4. The vertical displacement of the upper pile is recorded by the dial indicator at the top of the upper pile, and the vertical displacement of the lower pile is recorded by the dial indicator at the upper part of the load transfer rod. After loading, the load transfer rod connected by the jack generates pressure by contacting the top cover of the lower pile, and the jack generates tension by connecting the steel rebars with the top of the upper pile. The upper and lower piles generate upward and downward displacement and deformation, respectively. Therefore, the friction of the soil beside the pile is mobilized.

3. Test Results

3.1. *Q-w Curve.* The *Q-w* curve can reflect the load transfer law and the interaction between pile and soil. It can be seen from Figure 5 that the displacement of the test pile in the early stage of the test process changes a little. Under the first six levels of load, the displacements of the upper and lower piles are small, and the *Q-w* curve is roughly linear. This shows that the upper and lower piles are in the elastic stage, and the cumulative displacement is only 0.07 mm. With the increase of load, the rate of displacement growth increases gradually, and the *Q-w* curve becomes nonlinear. After the 7th level loading, the displacement of each level begins to increase and there is a significant increase. From the 7th level to the 13th level, the displacement increases obviously, and the nonlinear deformation of the soil around the pile occurs gradually, which indicates that the self-anchored test pile is in the elastic-plastic stage. From the 13th level to the 16th level, the displacement of the upper and lower piles increases sharply, and the test pile is in the limit state. In this test, most of the time, the test pile is in elastic state, and the friction of the upper pile has a great potential in the later loading stage. When the 16th level is loaded, the displacements of the upper and lower piles both appear to increase sharply, and the displacement exceeds more than 2 times of the load of the previous level; the settlement was not stable within 24 hours [11]. Finally, it can be considered that the test pile reaches the maximum loading value. At this time, the upper and lower piles are destroyed at the same time, showing a certain mutation.

3.2. *Analysis of Axial Force Distribution of Pile.* From the axial force distribution diagram of self-anchored test pile, the axial force of upper and lower piles shows a linear trend of downward decrease. The axial force of the upper pile at the middle equilibrium point is zero, and the axial force distribution of the pile is shown in Figure 6. It can be seen from the figure that the axial force of self-anchored test pile increases with the increase of loading level. For the upper pile, the reduction in the axial force of the pile near the equilibrium point continues to increase. In the test of self-

anchored test pile, although the loading mode is different from that in the traditional test pile, the bearing characteristics of friction pile are more obvious in the self-anchored test pile.

4. Analysis of Friction Resistance

In this paper, through the axial force between two adjacent strain gauges on the self-anchored test pile, the pile friction is obtained. The calculation formula of pile friction is as follows:

$$Q_i = \frac{P_{i-1} - P_i}{U \cdot l_i}, \quad (1)$$

where P_{i-1} and P_i represent the axial forces of the upper and lower test points of the i -th pile segment, U is the circumference of the pile, and l_i is the length of the i -th pile segment.

According to formula (1), the side friction resistance of the pile can be calculated, and the distribution curve of the measured friction resistance of the self-anchored test pile can be obtained, as shown in Figure 7.

It can be found from Figure 7 that, with the increase of load, the friction of the upper and lower piles of the self-anchored test pile increases with the depth. At different loading time, the increasing rate of the pile's friction is different. The larger the load, the greater the rate of increase of friction. The friction of self-anchored test pile is different, and the degree of the friction is different in each layer of soil.

In order to analyze the variation law of the friction resistance of self-anchored test pile, the concept of pile friction ratio is introduced in this paper. The pile friction ratio is the section friction/the maximum friction value of the whole pile. Figure 8 shows the distribution curve of the pile friction ratio with depth. The corresponding stages in the figure are the elastic stage, the elastic-plastic stage, and the limit state of the self-anchored test pile friction ratio distribution.

It can be seen from Figure 8(a) that when the load is less than 14 kN, the friction of the upper pile increases with the increase of depth, and the maximum rate of increase of side friction with depth is from 2.0 m to 2.9 m. The friction resistance of the lower part of the upper pile is greater than that of the upper part, and it accounts for a larger part of the upper pile. However, with the increase of the load, the proportion of the friction of the lower part of the upper pile is gradually decreasing. For the lower pile, the pile friction presents an increasing trend, but the growth trend is slow.

It can be seen from Figure 8(b) that the side frictions of the upper and lower piles are increasing, but the friction resistance of the lower pile increases faster. In the elastic-plastic stage, the friction resistance of each section of self-anchored test pile is not changed much and tends to be consistent with the increase of load. It shows that, with the increase of the load, the stress of each section of the self-anchored test pile is also continuously improved. The soil around the lower part of the upper and lower piles gradually undergoes nonlinear deformation.

FIGURE 4: In situ test.

FIGURE 5: Q-w curve of self-anchored test pile.

It can be seen from Figure 8(c) that when the last two levels of load are applied, the friction of the upper and lower piles increases rapidly. With the increase of the loading level, the frictions of the upper and lower piles are close to the limit value, and the increase range is small. The friction of the test pile at 30 kN is the ultimate friction of the pile. The shear failure occurs, the pile soil reaches the yield state, and the self-anchored test pile is in the limit state.

5. Hyperbolic Function Analysis of Load of Self-Anchored Test Pile

The load transfer method can better reflect the nonlinearity between pile soil and the stratification of foundation, and the calculation is simple, which can be well applied to

FIGURE 6: Distribution of axial force of self-anchored test pile.

engineering [12]. Load transfer method can well simulate the load transfer relationship between pile and soil, and the key point of load transfer method is to choose the load transfer function.

The pile is divided into many tiny elastic elements along the vertical direction, and each element is connected with the soil by nonlinear spring, as shown in Figure 9.

The pile top load is P_0 , and the axial force of each section from top to bottom is $P_1, P_2, P_3, \dots, P_n$. The microelastic element at depth z is denoted as dz . It can be seen from the figure that the pile displacement $s(z)$ at depth z is

FIGURE 7: Distribution curve of pile friction along depth.

FIGURE 8: Continued.

FIGURE 8: Distribution curve of pile friction ratio along depth. (a) Elastic stage. (b) Elastic-plastic stage. (c) Limit state.

FIGURE 9: The calculation model of single pile by load transfer method.

$$s(z) = s_t - \int_0^z \frac{P(z)}{EA} dz. \quad (2)$$

The axial force $P(z)$ at depth z is

$$P(z) = P_t - \pi d \int_0^z \tau_s(z) dz. \quad (3)$$

In the formula, s_t is the displacement of pile top; P_t is the pile top load; E is the elastic modulus of pile body; A is the cross-sectional area of pile shaft; d is the diameter of the pile.

According to the static equilibrium condition of any element on the pile, the pile friction at depth z is

$$\tau_s(z) = -\frac{1}{\pi d} \frac{dP(z)}{dz}. \quad (4)$$

According to formulae (2)–(4), the load transfer differential equation between the pile side friction at depth z and the pile displacement can be obtained.

$$\frac{d^2 s(z)}{dz^2} - \frac{\pi d \tau_s(z)}{EA} = 0. \quad (5)$$

The relationship between friction and pile-soil relative displacement can truly present the mechanism of pile-soil interaction, and the load transfer method is practical and easy to calculate [13]. The hyperbolic function model is used for the fitting analysis of the upper and lower piles because the hyperbolic function model has high fitting degree and is practical [14].

Many scholars at home and abroad have done a lot of research and experiments on the choice of function model, which proves that hyperbolic function model can better simulate the nonlinear relationship between pile and soil [15–19]. In 1957, Seed proposed a method to divide the pile into many elements, each element has nonlinear relation with soil, and the load transfer relationship between pile and soil is simulated by numerical analysis [20]. Kraft et al. established the theoretical load transfer curve according to the principle of shear-displacement method [21]. Goel and Patra discretized the pile into several elements and obtained the load displacement behavior of sand according to the load transfer method [22]. According to the load transfer function of compressed pile, Jeong et al. proposed the theoretical analysis method of Mindlin's solution [23]. The mathematical expression of the hyperbolic function model is as follows:

$$\tau = \frac{s}{a + bs}. \quad (6)$$

FIGURE 10: Linear fitting curve of upper and lower piles. (a) Curve fitting of upper pile. (b) Curve fitting of lower pile.

TABLE 2: The load transfer parameters a and b and the determinable coefficient R^2 .

	a	b	R^2
Upper pile	0.002	0.021	0.989
Lower pile	0.003	0.016	0.993

In the above formula, τ is the pile side friction; s is the relative displacement of pile and soil; a and b are the load transfer parameters of the soil around the pile. By transforming the formula, we can get the following result:

$$\frac{s}{\tau} = a + bs. \quad (7)$$

According to formula (3), it can be seen that s/τ and s have a linear relationship. The load transfer parameters a and b can be obtained by hyperbolic linear fitting. The fitting curve is shown in Figure 9.

Through linear fitting, the load transfer parameters a and b of the upper pile are 0.002 and 0.021, respectively, and the load transfer parameters a and b of the lower pile are 0.003 and 0.016, respectively, as shown in Figure 10.

In order to judge the fitting degree accuracy of linear fitting curve, the determinable coefficient R^2 is introduced. R^2 is the overall degree of fit, and the closer the value of R^2 is to 1, the higher the fitting accuracy is.

Through the data analysis software Origin, the resolvable coefficient R^2 of upper and lower piles can be obtained as 0.989 and 0.993, respectively. The load transfer parameters a and b and the determinable coefficient R^2 are summarized in Table 2. It shows that the relationship between the friction resistance of the upper and lower piles and the pile-soil displacement follows the law of hyperbolic function very well. In addition, it is proved that some nonlinear characteristics of hyperbolic function model can well show the characteristics between pile and soil.

6. Conclusion

- (1) According to $Q-w$ curve, self-anchored pile test can be divided into three stages. During the first six loads, the $Q-w$ curve is approximately linear, indicating that the self-anchored test pile is in the elastic stage. During the period from the 7th load to the 13th load, the soil around the self-anchored pile gradually deforms nonlinearly, and the self-anchored test pile is in the elastic-plastic stage. During the period from the 13th load to the 16th load, the displacement of the upper and lower piles increases sharply, and the self-anchored test pile is in the stage of limit state.
- (2) The evolution laws of the friction of the upper and lower piles are as follows. In the elastic stage, the side friction of the upper and lower piles increases with the increase of the depth, and the growth rate is larger. In the elastic-plastic stage, the frictions of the upper and lower piles are increasing, and the soil around the lower part of the upper and lower piles gradually undergoes nonlinear deformation. In the limit state, the soil around the pile reaches the yield state, and the self-anchored test pile is in the limit state.
- (3) Based on the in situ test results of self-anchored test pile and the understanding of the development law of pile friction of self-anchored test pile, the introduction of pile friction ratio is helpful to deepen the understanding of load transfer mechanism between self-anchored test pile and soil and provides a reference for future research.
- (4) In this paper, the in situ test results of self-anchored test pile are in good agreement with the curve obtained by fitting the hyperbolic function model after deformation and are in good agreement with the

actual working conditions, which proves the feasibility and correctness of this method.

Data Availability

Some or all data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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References

- [1] A. M. Kaynia, *Analysis of Pile Foundations Subject to Static and Dynamic Loading*, CRC Press, Boca Raton, FL, USA, 2021.
- [2] W. H. Zhong, M. L. Shi, and S. Y. Liu, "Load transfer performance of overlength piles," *Rock and Soil Mechanics*, vol. 26, no. 2, pp. 307–310+318, 2005.
- [3] X. R. Zhu, P. F. Fang, and H. M. Huang, "Research on super-long pile in soft clay," *Chinese Journal of Geotechnical Engineering*, vol. 25, no. 1, pp. 76–79, 2003.
- [4] W. B. He, *Study on Time Effect of the Settlement of Single Pile under Static Load*, Zhejiang University, Hangzhou, China, 2003.
- [5] H. B. Seed and L. C. Reese, "The action of soft clay along friction piles," *Transactions of the American Society of Civil Engineers*, vol. 122, no. 1, pp. 731–754, 1957.
- [6] Z. R. Xiao, "Determination of hyperbolic model and its corresponding parameters in single-pile analysis," *Soil Engineering and Foundation*, vol. 156, no. 3, pp. 60–63+75, 2002.
- [7] G. X. Nie and F. Chen, "Using load-transmission function method in single pile bearing," *Soil Engineering and Foundation*, vol. 18, no. 2, pp. 29–32+45, 2004.
- [8] F. N. Dang, N. Liu, and W. A. He, "Hyperbolic model of load transfer method for single pile in Xi'an area and its engineering application," *Chinese Journal of Geotechnical Engineering*, vol. 29, no. 9, pp. 1428–1432, 2007.
- [9] X. S. Tang, D. Q. Li, C. B. Zhou et al., "Probabilistic analysis of load-displacement hyperbolic curves of single pile using Copula," *Rock and Soil Mechanics*, vol. 33, no. 1, pp. 171–178, 2012.
- [10] Q. Cui, Y. F. Cheng, X. L. Lu et al., "In-situ pull-out test and parametric study of load-displacement model for hole digging foundation in the strong weathered rock mass," *Rock and Soil Mechanics*, vol. 39, no. 12, pp. 4597–4604, 2018.
- [11] China Academy of Building Research, *Technical Code for Testing of Building Foundation Piles*, China Architecture & Building Press, Beijing, China, 2014.
- [12] W. Z. Liu, S. Qu, H. Zhang et al., "An integrated method for analyzing load transfer in geosynthetic-reinforced and pile-supported embankment," *KSCE Journal of Civil Engineering*, vol. 21, 2017.
- [13] X. Z. Xi and L. Z. Chen, "Analytical fitting method of loading-displacement curve for upper pile segment under O-Cell pile testing," *Journal of Shanghai Jiaotong University*, vol. 45, no. 10, pp. 1498–1503, 2011.
- [14] B. W. Shen, "Nonlinear analysis of single piles under negative skin friction," *Journal of Hefei University of Technology*, vol. 31, no. 12, pp. 2024–2027, 2008.
- [15] Y. Ma, *Research on the Bearing Behavior of Super-long Drilled Pile*, WuHan University, Wuhan, China, 2008.
- [16] G. Y. Cui, K. L. Chen, and B. Yang, "Comparative analysis of load-displacement curve fitting models for rock-socketed uplift piles," *Railway Engineering*, vol. 60, no. 12, pp. 83–87, 2020.
- [17] J. Z. Ye and J. Zhou, "In-situ test research on the functional mode of skin friction of cast-in-place piles," *Building Science*, no. 6, pp. 56–58+68, 2005.
- [18] Y. P. Jiang, M. Yang, and J. H. Xiong, "Analytical equation of loading-settlement curve under O-Cell pile testing method," *Chinese Journal of Rock Mechanics and Engineering*, vol. 25, no. S1, pp. 3258–3264, 2006.
- [19] H. B. Xiao, H. H. Zhong, and Y. J. Zhang, "Numerical iteration method for determining load-settlement relationship of a single pile," *Rock and Soil Mechanics*, vol. 22, no. 5, pp. 592–596, 2002.
- [20] C. W. Wang, G. X. Mei, and L. H. Song, "Generalized load transfer function and its application," *Rock and Soil Mechanics*, vol. 21, no. 6, pp. 1217–1220+1224, 2007.
- [21] L. M. Kraft, R. P. Ray, and T. Kagawa, "Theoretical t - z curves," *Journal of the Geotechnical Engineering Division*, vol. 107, no. 11, pp. 1543–1561, 1981.
- [22] S. Goel and N. R. Patra, "Prediction of load displacement response of single piles under uplift load," *Geotechnical & Geological Engineering*, vol. 25, no. 1, pp. 57–64, 2007.
- [23] S. Jeong, S. Ahn, and H. Seol, "Shear load transfer characteristics of drilled shafts socketed in rocks," *Rock Mechanics and Rock Engineering*, vol. 43, no. 1, pp. 41–54, 2010.