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MODEL TESTS ON CHARACTERISTIC OF SUCTION CAISSONS IN SATURATED FINE SAND UNDER INTERMITTENT LOADING

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ABSTRACT

Suction caissons are often used for the caissons of both offshore oil platforms and offshore wind power projects because of their advantages of simple construction, economical cost, and reusability. In this study, model tests were conducted in sand in order to investigate the effects of the caisson installation method on the penetration depth and the critical suction. Results of the test program showed that the method of changing the frequency of suction during different stages of the process can increase the penetration depth of the caisson. Combining with the deformation of the soil body inside and outside the caisson, the existing method for calculating the critical suction is modified, and the critical suction calculation equation of the discontinuous penetration test is proposed. Based on the test results, the calculation equation of the soil heave height can be more accurate predicted. The analysis results verify that the calculation method and the actual results are in good agreement

Keywords: intermittent suction installation, model test, sand, suction caisson

INTRODUCTION

Suction bucket caisson is a foundation form applied to both offshore oil platforms and offshore wind power projects. It is also called a suction caisson or a suction caisson. Due to its advantages of simple construction, economical cost, and reusability, it has been widely used in recent years and has attracted attention at home and abroad.

Suction control is a major problem in the process of the penetration of suction caissons in sand. The seepage in the caisson caused by suction affects the effective stress of the soil and re-duces the installation resistance of the suction caisson [1-4]. The large suction will cause large inflow of the soil near the end of the suction caisson side wall to the inside, and the internal mud surface will swell, that is, "the phenomenon of the soil heave plug". When the seepage is too large, it will also cause caisson piping [5, 6].

Therefore, important design aspects in connection with the installation of suction caissons are to determine the critical under pressure and the amount of "soil heave plug" inside the cylinder during penetration.

The phenomenon of "soil heave plug" was first discovered in 1980 at the site of the installation of suction caissons at the Gorm site, and its impact cannot be ignored [7]. On the one hand, the internal mud surface of the suction caisson because of the soil heave plug is brought into contact with the caisson roof in advance so that the suction caisson cannot penetrate to the designed depth [8, 9]. It will lead to a decrease in the bearing capacity of caisson. A scholar [10] pointed out that the response of the suction caisson was mainly depended on the degree of development of the internal soil heave. On the other hand, it will cause changes in the seepage field around the caisson and increase the degree of erosion of the surrounding soil (for shallow sea areas, this effect is particularly significant).

Given that the emergence of "soil heave plug" has an ad-verse effect on the safety and stability of the caisson itself and the superstructure, many scholars have done research on the soil heave plug characteristics in the caisson installation process, such as a scholar [11]. The influence of suction and installation depth on the development of soil heave was discussed, and the soil heave height was calculated numerically. Other scholar [12] derived the theoretical calculation equation of the soil heave height based on the assumption that the volume of the soil inside the caisson is *m* times of the volume of the soil displaced by the skirt moved into the caisson. He point-ed out that the coefficient *m* is not affected by the model size, which is mainly related to soil strength. A study of excessive internal soil uplift was carried out through the model tests, for example [13].

In order to improve the quality of caisson installation, a scholar [14] added permeable stones to the upper part of the inner soil and other scholar [15] introduced a filter layer consisting of geotextile and gravel. These measures delayed the time for the sand to contact the inner roof, but so far there are no measures that can effectively suppress the development of soil heave plug. In view of the development process of soil heave plug, many tests and theoretical studies have found that they are related to the suction and the real-time penetration depth of the suction caisson. Based on the internal soil stability, a previous researcher [16, 17], API specification [18] and DNV specification [19] gave equations for calculating suction critical penetration values. Whittle, Rauch, and Chen all agreed that the base sidewall penetration. The displaced soil volume is equivalent to the internal soil heave volume [20].

In order to achieve the desired penetration effect of the suction caisson, some scholars consider using a non-continuous suction penetration method. A scholar [21-22] proposed the concept of intermittent suction penetration and obtained a result that its final penetration depth is greater than the continuous suction penetration; a scholar [23] took an intermittent penetration scheme for indoor model tests, different durations of suction pulses were used in 4 tests and it was found that proper selection of the time of impact suction and the peak value of suction force can effectively reduce the up-lift height of soil heave. A scholar [24] first made the model sink by a certain depth under the action of its own weight and counterweight during the test and applied five levels of stable suction in the second stage. The test results showed that the seepage flow drag reduction effect of this test scheme is obvious, and an ideal final penetration depth could be obtained.

As mentioned above, in the existing studies on the characteristics of discontinuous suction penetration, the same suction was applied with the same frequency along the tests and the influence of the frequency of the suction didn't be considered. However, the frequency of the suction directly affects the suction and the development process of the soil heave. The project described in this paper aimed at addressing the influence of the frequency of the suction changing on the mechanism of suction installation tests.

TEST DEVICE AND SOIL PREPARATION

SAND TANK

The sand tank (1 m long, 1 m wide, and 0.8 m high) is used to carry out model tests in saturated fine sand. Its dimension is large enough to eliminate the effect of the boundary on the result. The sand tank is welded with galvanized steel which reduces the friction between the soil and the inner wall and prevents the corrosion of the tank. A drainage channel is provided at the bottom side wall of the tank to adjust the height of the water level. Through the bottom drainage it can also accelerate the consolidation of the sand in the tank.

INSTRUMENTATION

A schematic diagram of a test device is shown in Fig. 1. The system includes model tank, suction caisson, displacement sensor, negative pressure sensor, and suction piping. The model penetrates into the soil (sand) through suction which is provided by an intelligent vacuum pump and is sent to the top of the model through the suction pipeline. A ball valve is provided in the middle of the suction pipeline to control the rate of suction. The vacuum pump used in the test was a water and air vacuum pump WKA1300-24A. A vertical displacement sensor (LVDT) and a negative pressure sensor are arranged on the top of the model caisson. Two sensors connected through a data acquisition instrument (Data Taker DT-80G) for data acquisition. The ranges of the LVDT and negative pressure sensor are 300mm and ±0.1 MPa. The accuracy is 0.1mm and 0.15 kPa, respectively.

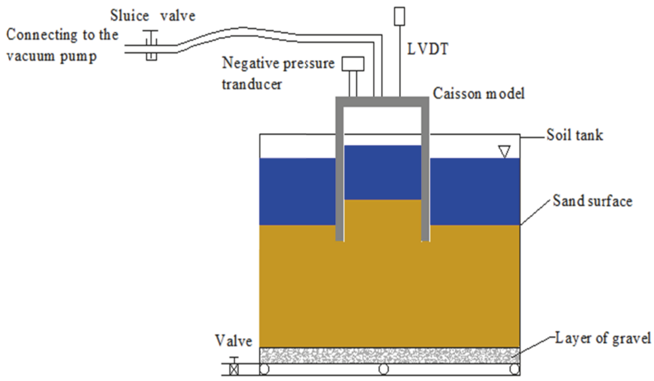


Fig. 1. Test tank and model

MODEL DETAILS

The structure of the model is barrel-shaped and machined from steel with a diameter of 120mm, a skirt wall thickness of 2mm, a roof thickness of 10mm, and a length of 240 mm. The caisson model had no internal stiffeners. A suction hole is provided for connecting the suction pipe, and a negative pressure sensor connection hole is provided on the other side.

To observe the phenomenon of soil heave inside the caisson during the penetration process, the model of organic glass was processed follow the same dimension as the steel caisson.

FORMATION OF SAND SAMPLE

Since the suction caisson is applied in the field of ocean engineering, soils with marine characteristics should be se-lected for the test soil. The test sand was taken from Qingdao Golden Beach.

The basic physical parameters were as follows: average particle size D50=0.097 mm, non-uniform coefficient Cu=1.78, curvature coefficient Cc=0.997, natural porosity ratio e=0.62, and maximum pore ratio e_{max}=0.903, mini-mum void ratio e_{min}=0.61, specific gravity Gs=2.69; mechanical parameters: internal friction angle φ=34°, dilatancy angle ψ=10°, internal cohesive force c=0.8 kN/m^2, compression modulus Es=686 kN/m^2.

SOIL PREPARATION

First, gravel with a thickness of 10 cm is laid at the bottom of the model tank as a drainage layer, and two layers of geotextiles are laid on the gravel cushion as an inversion filter to prevent the loss of fine sand and clay particles as the model tank drains. The "sand rain method" is used to layer the sand, and at the same time, the water is slowly poured into the model tank and the water level is higher than the sand surface. Then, the sand was consolidated by circulating drainage, and each test cycle was repeated twice. After consolidation of the sand, ensure that the water level in the model tank exceeds about 10 mm from the surface of the sand, ensuring that the sand in the model tank is always saturated. To ensure the reliability and repeatability of the test results, each test condition was strictly controlled.

Sand density and permeability coefficient are two influencing factors of its mechanical properties. At the same time, it has a great influence on the sink penetration characteristics of the suction caisson. Sand consolidation is achieved by circulating drainage, and the relative compactness of the sand after consolidation is determined. The degree is 0.997 and the permeability coefficient is 0.002 cm/s.

TESTING PROCEDURE

TESTING PROGRAMS

The test includes Continuous Suction Penetration(CSP)and Intermittent Suction Penetration According to Frequencies of Suction(IPT), as shown in Table I.

Tab 1. Test arrangements

Model diameter/mm	Model height/mm		Code	Test method
120	240		CSP	Apply suction continuously.
			IPT-1	Apply suction at an interval of 2s.
			IPT-2	Phase I (before 2 min): apply suction continuously. Phase II (after 2 min): apply suction at intervals of 2s.
			IPT-3	Phase I (before 2 min): apply suction continuously. Phase II (after 2 min): apply suction at intervals of 4s.
			IPT-4	Phase I (before 2 min): apply suction at intervals of 1s. Phase II (after 2 min): apply suction at intervals of 4s.

The traditional penetration method is to first make the caisson penetrate into a certain depth under the action of its own weight. Then apply continuous suction to make it penetrate to a predetermined depth. In the intermittent suction installation test, the frequency of suction is mutative, and the intermittent penetration time is longer.

Existing research results indicate that as the penetration depth increases, the height of soil heave increases gradually, and the rate of increase gradually slows during the later period of the experiment. Simultaneously, the seepage flow gradually weakens. According to this rule, different suction frequencies are adopted in different stages of the IPT-II, IPT-III, and IPT-IV tests. Considering the dimensions of the caisson and the test time, after many trials, 2 min time node is considered as a time separator.

TESTING PROCEDURE

To make the test results reproducible, it is vital to keep each testing condition unchanged, especially the relative densities of sand.

First, placing the caisson vertically on the sand surface so that it will plunge under its own weight. When the caisson is in a stable state, the drainage outlet of the roof to form a sealed environment inside the caisson. After end of self weight penetration, the caissons are penetrated by means of under pressure inside the caissons corresponding to a prototype rate of about 1–2 *m/h*. A relatively high under pressure is needed to initiate further penetration. This 'set-up' is probably due to some pore pressure redistribution and consolidation of the remolded zone near the caisson wall during the rest period. The penetration resistance decreases after some penetration, indicating that the sand become remolded again. The caisson continues to penetrate into the sand until the inner roof contacts the sand surface.

The above is the conventional penetration testing process. In this process, the suction is applied continuously. However, intermittent penetration tests (IPT) are used in this paper which the suction is applied discontinuously (Table. I)

Now, take the intermittent suction (IPT-I) as an example. The caisson first penetrated under the action of its own weight. Then, by controlling the intelligent vacuum pumping, the suction pump was turned off after 2s. Pause for 2s and then exhaust for 2s to form a "cyclical" loading way until the inner roof contact with the sand surface. In the same way, test conditions IPT-II, IPT-III, and IPT-IV were performed, and each set of test conditions was repeated at least three times.

The purpose of this paper is to study the penetrating characteristics and suction dissipation mechanism of the suction caisson during the intermittent installation. The characteristics of the caisson itself and the deformation of the surrounding soil are not considered.

TESTS RESULTS

COMPARISON OF TESTS CONSULTS

The test time and suction penetration depth of various working conditions are shown in Table 2. From the consideration of the penetration depth and the time-consuming of the test, a test plan that is more suitable for suction penetration is tried to be determined.

Tab. 2. Test time and final penetration depth

	CSP	IPT-I	IPT- II	IPT- III	IPT- IV
Test time / s	98	372	251	196	471
Final penetration depth / mm	104.9	158.1	156.9	149.2	163
Peak of suction /kPa	-1.29	-3.53	-4.42	-2.66	-4.99

Considering the penetration depth of suction, the depths of the caissons obtained by conventional suction penetration tests CSP and four intermittent penetration tests IPT-I, IPT-II, IPT-III, IPT-IV are 104.9mm and 158.1mm, respectively. The penetration depths of mm, 156.9mm, 149.2mm, and 163mm obtained by different penetration frequency methods are less than 9.3%. However, compared to the conventional penetration tests, the final penetration depth obtained by the intermittent penetration method is significant. The maximum difference reached 55.4%.

From the time-consuming aspect of the test, for the intermittent penetration tests that can obtain a large penetration depth, the longest time-consuming (IPT-IV) is 471s, and the shortest time-consuming (IPT-III) is 148s. The test time-consuming difference exceeds 218 %. It can be seen that due to the use of different suction frequency, the intermittent penetration test takes a lot of time.

In summary, in the two time-consuming tests (IPT-II and IPT-III), the base final penetration depth (with a self-invasive depth of 70 mm) reached 226.9 mm and 219.2 mm, respectively, which is the base length. 94.5% and 91.3% of the total penetration depth exceeds the conventional penetration test depth of 52mm and 44.3mm. It can be seen that the IPT-II and IPT-III intermittent pendulum penetration method is a better solution. The continuous application of suction during the early penetration process, and the intermittent application of suction during the later period can achieve a greater penetration depth in a shorter period of time. It is more suitable for suction caisson penetration.

COMPARISON OF MAXIMUM SUCTION AND CRITICAL SUCTION

Under intermittent suction, pore water pressure is generated inside the soil heave. The pore water pressure continuously diffuses and dissipates due to the presence of dynamic seepage. This effect may even cause sand liquefaction. Based on the analysis of soil stability, the internal soil of the caisson will undergo infiltration and destruction if suction exceeding the critical is applied. So, it is necessary to predict the critical suction.

Several authors have used numerical methods to study the critical suction's calculation method on the conventional suction penetration methods. For example, Feld used the numerical model to analyze the conditions that caused the osmotic failure, and the outlet hydraulic gradient was used as the controlled condition to obtain the critical suction calculation equation of the undivided plate cylinder type:

$$\frac{P_{crit}}{\gamma' D} = 1.32\left(\frac{L}{D}\right)^{0.75} \tag{1}$$

Senders used a finite element numerical model to suggest a critical suction calculation method:

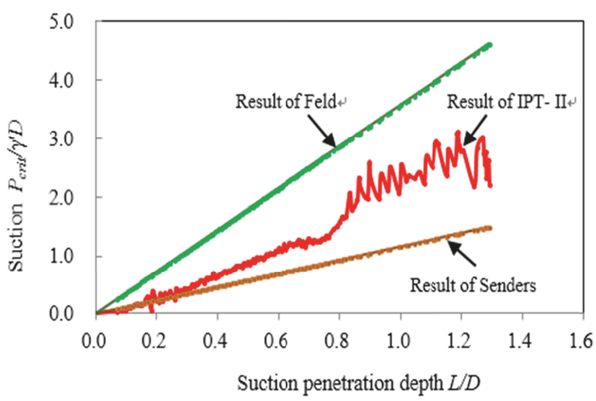
$$\frac{P_{crit}}{\gamma' D} = \left\{\pi - \arctan\left[5\left(L/D\right)^{0.85}\right](2-\frac{2}{\pi})\right\}\frac{L}{D} \tag{2}$$

However, there is no critical suction calculation method for the intermittent penetration method.

Fig. 2 (a) and Fig. 2 (b) are the result of the comparison between the maximum suction applied in IPT-II and IPT-III and the critical suction calculated using the Feld equation and the Senders equation. (The actual suction in the process of intermittently applying suction has a large fluctuation range, so take the maximum suction value for comparison).

It can be seen that the suction applied during the IPT-II and IPT-III tests are greater than the critical suction calculated by the Senders equation. In the test, the inside of the cylinder was found through the entire top of the plexiglass cover. It is stable and there is no infiltration damage such as piping. It shows that the critical suction calculated according to the Senders equation is conservative, which is caused by the assumption that the permeability coefficient of the soil is constant in the process of sinking. It is consistent with the conclusion of Chen Fei. The IPT-III test was anastomosis better than the IPT-II test. The reason was that the IPT-III test had a longer pause between suction in the later period of the experiment, which made the dissipation of suction relatively thorough, made the soil density relatively larger, and made the soil permeability coefficient relatively higher.

(a) Experiment IPT- II



(b) Experiment IPT- III

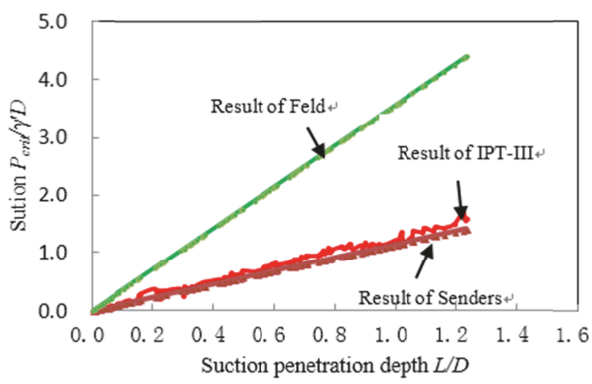


Fig. 2. Comparison between applied suction and existing critical suction

This is consistent with the view that the permeability coefficient of sand permeability test data and theoretical results in Dr. Senders's thesis are in good agreement, but the consistency of the permeability coefficient is not good.

The actual applied suction is less than the critical suction calculated with the Feld equation. This is because Feld's equation assumes that sand is an undrained material, and Feld does not clearly determine the variation law of shear strength of sand. The Feld equation is rarely used.

In summary, the above two calculation methods are not suitable for the critical suction calculation of the intermittent suction penetration test. A new calculation method needs to be introduced and the influence of the sand permeability coefficient with the depth and the seepage flow should be considered.

A scholar considered the change of the permeability coefficient k_{fac} of the soil inside and outside the suction caisson during the test, and used the finite element method to calculate the critical suction force equation:

$$\frac{P_{crit}}{\gamma' D} = \left(\frac{L}{D}\right)\left(1 + \frac{\alpha_1 K_{fac}}{1-\alpha_1}\right) \qquad \textbf{(3)}$$

where:

$$\alpha_1 = 0.45 - 0.36(1 - e^{-2.08L/D})$$

$$k_{fac} = {k_i}/{k_o}$$

For the calculation of k_{fac} in (3), there are many methods in the existing research results. For example, Houlsby and Byrne assumed that k_{fac} is a constant value. Senders thought that in the process of continuously applying suction, the soil heaves inside the suction base was on the soft state. The coefficient k_{fac} is a factor of not less than 1.

In fact, during the penetration process, the suction induces seepage in the soil around the caisson. The permeability coefficient ratio k_{fac} of the soil inside and outside is constantly changing. In the penetration test of intermittently applied suction, the permeability coefficient of the soil heave in the caisson is relatively small and has good permeability due to the presence of seepage at the beginning of the test. After intermittent suction is applied, it is equivalent to the intermittent period when suction is stopped. When the earth plug is unloaded, and the suction is dissipated, the permeability coefficient of the soil heave decreases at this stage. When suction is applied again, the seepage occurs again within the soil heave, and the permeability coefficient increases. This cycle occurs. Therefore, the value of the permeability coefficient k_{fac} is related to the size of the suction.

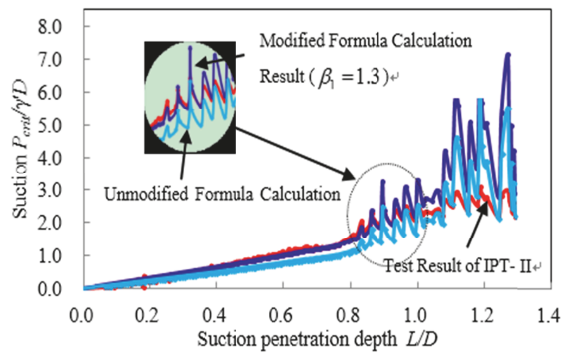
Some scholars have obtained that ABAQUS finite element software considering the stress-seepage coupling was employed to figure out the influence of variation of water level on the stability of cement shear wall reinforced approach channel side slope, in combination with the strength reduction technique [25]. And Some scholars have obtained a certain relationship between permeability coefficient and stress through laboratory tests and engineering practice, such as negative exponential equations, negative power exponential equations, hyperbolic equations and exponential equations, among which the most widely used one is the negative exponential equation which is studied in the pumping test [26].

With Louis' empirical relationship, the permeability coefficient ratio $k_{fac}=4k_0\ exp(-\alpha 1P)$ is defined here, and the permeability coefficient k_0=0.006cm/s of sand under the effect of initial effective stress, and P is the actually applied suction force [27]. Substituting k_{fac} into (3) yields the following equation:

$$P_{crit} = \beta\gamma' L \left(1 + \frac{4\alpha_1 k_0 \exp(-\alpha P_1)}{1-\alpha_1}\right) \tag{4}$$

Fig. 3(a) and (b) are the comparison of the maximum suction applied in the two intermittent penetration methods of IPT-II and IPT-III and the critical suction calculated by (4). It can be seen that the IPT-III test result is very close to the equation calculation result. And the initial segment of the IPT-II test result is close to the equation calculation result. However, the later deviation is relatively large because of the intermittent action of the suction at the later stage of the IPT-II test is relatively short and the suction is not fully dissipated, the actually applied suction value is relatively large [28]. Here, the correction coefficient β is added to 1.3. Fig. 3 (a) proves that the experimental data is in good agreement with the theoretical calculation results. Therefore, equation (4) can be used as the critical suction calculation equation for the intermittent penetration of the suction caisson in saturated fine sand. The value of β in the equation is: in IPT-II with continuous suction for early stage and intermittent suction for 2s at late interval, β_1=1.3; for the IPT-III duration of the continuous application of suction at the early stage and the intermittent application of the suction at the late interval 4s, β_2=1.0.

(a) Experiment IPT-II



(b) Experiment IPT- III

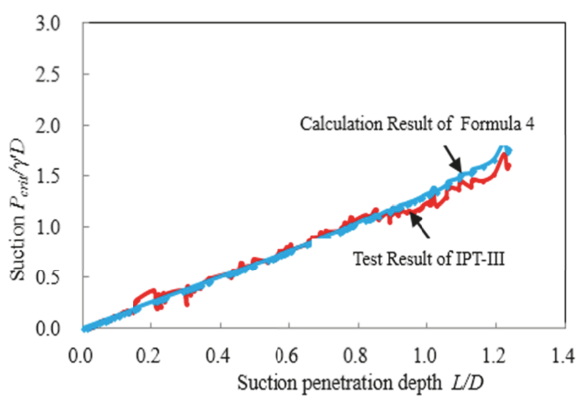


Fig. 3. Comparison between applied suction and existing critical suction

THE DEVELOPMENT LAW OF SOIL HEAVE IN THE PROCESS OF INTERMITTENT PENETRATION

In order to better study the development law of the internal soil heave under intermittent suction, the IPT-III test was conducted with the organic glass caisson [29]. The development of soil heave during the test is shown in Fig. 4.

As shown in Fig. 4, the suction is continuously applied during the initial stage of IPT-III test, the soil heave develops a certain lag and the caisson sinks slowly. The penetration depth increases significantly as the suction increases [30]. The rise of the plug is faster, and at the same time the speed of the caisson sinks faster, and the rate of growth of the soil heave slows down [31]. At the later stage, the suction is applied intermittently, and the height of the soil heave changes significantly. Otherwise under the effect of the fluctuation suction, the height of the plug decreases.

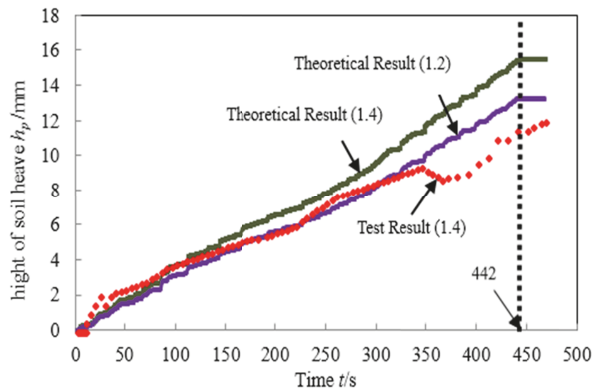


Fig. 4. Curves of soil heave during intermittent suction insertion

The internal soil heave touches the inner cap at 442s. Because the water inside the soil heave is still being pumped out, the caisson sinks a bit, while the soil heave has a slight increase.

If the suction increases to a larger value at the later stage of the test, the height of soil heave will develop rapidly. The caisson used in this experiment is a small-scale model, and the suction rate is small. Hence, the soil heave is almost linearly developed. The height of soil heave measured in the experiment is compared with the theoretical calculation result [32], as shown in Fig. 4.

In order to obtain the calculation equation of soil heave height, it is assumed that the volume of the internal soil of the caisson is m times the penetration volume, that is, the equation for the soil heave height is:

$$P_{crit} = \beta\gamma' L \left(1 + \frac{4\alpha_1 k_0 \exp(-\alpha P_1)}{1-\alpha_1}\right) \tag{5}$$

In the above equation:

- m is the coefficient of soil heave height calculation;
- h is the penetration depth;
- D_0 is the diameter;
- D_i is the diameter.

If the cyclic loading time is short and the permeability coefficient of soil heave is small, it is assumed that the pore water confined in the soil heave does not seep during the cyclic loading, and the diffusion and dissipation of the pore water pressure can be neglected. But If the cyclic loading time is long and a large soil permeability coefficient, this assumption will inevitably cause some errors. Therefore, it is necessary to study the influence of the intermittent suction on the soil heave inside the caisson. The pore water pressure generation, diffusion and dissipation were taken into account together. Try to take m was 1.2 and 1.4 and after trial calculations, it is better that the initial suction e is applied when m is 1.2. Therefore, the calculation equation of the soil heave height during the intermittent penetration process of the suction caisson in saturated fine sand can be expressed as:

$$h_p = 1.2h\left(D_0^2 / D_i^2 - 1 \right) \quad (6)$$

From Fig. 4, the actual soil heave obtained in the later stage of the experiment is less than the theoretical value calculated by (6). It indicates that the dissipation of suction during the intermittent penetration test makes the soil heave effectively controlled. This measure delays the time that it touches the inner lid and leads to a larger final suction penetration depth. As the suction is dissipated and the rule of the soil heave changes, the soil heave rise rate and the caisson sink rate no longer have a linear relationship.

CONCLUSION

This paper proposes a new type of intermittent suction penetration test program, which changes the frequency of suction on the basis of the traditional suction penetration test scheme and introduces the permeability coefficient ratio k_{fac} of the soil inside and outside the suction caisson. Considering the influence of variation of permeability coefficient of sand with depth and seepage, the calculation method of critical suction in intermittent suction test is analyzed and discussed.

By observing the development process of the internal soil heave during the test and embedding the relationship between the volume of the soil inside the caisson and the volume of soil that penetrates the caisson, the calculation equation of the soil heave height in the intermittent suction test is studied.

The results showed that during the penetration testing process, the suction is continuously applied to the suction caisson, and the method of intermittently applying the suction at the later stage is more suitable for the suction caisson.

The actual soil heave value obtained in the later stage of the intermittent penetration test is less than the theoretical value, which indicates that the dissipation of suction in the experiment makes the soil heave development effectively controlled and delays the time for the plug to contact the inner top cover. At the same time, because the suction has dissipated and the soil heave development law changes, the rise rate of the soil heave and the caisson's sink rate has no longer been linearly related. Based on the results of the soil heave measurement, the calculation equation of the soil heave height is deduced, and the results confirm this calculation method is suitable for intermittent frequency suction penetration test.

Through the comprehensive comparison between the theoretical settlement results and the experimental data, the calculation method of the existing critical suction is not suitable for the intermittent suction penetration test. The calculation of the critical suction by Senders tends to be safer, and the Feld calculation is more conservative. Considering the change of the permeability coefficient of the soil inside and outside the caisson, the prediction method of the suction is improved, and it is proved that this method is more suitable for the intermittent penetration test.

Even in the same experimental phase, the suction's rate directly affects the penetration depth. Therefore, the determination of the suction's rate in each test phase is the focus of the next step.

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