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RESEARCH ARTICLE-CIVIL ENGINEERING

Study on Uplift Bearing Characteristics of Gravitational Reinforced Composite Suction Caisson Foundation

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Abstract

Suction caisson foundations are often subjected to vertical pullout loads, where the uplift bearing capacity of caisson is only composed of the internal and external friction and caisson weight under the drainage condition, hence the uplift bearing capacity is usually small. However, there is no corresponding bearing capacity improvement technology research. So a new suction caisson foundation, gravitational reinforced composite suction caisson foundation, is proposed to solve this problem. The uplift bearing capacity of the gravitational reinforced composite suction caisson foundation and the shearing characteristics of caisson-cement soil interface were studied through the model test and pushout test. The test results show that the uplift bearing capacity of the gravitational reinforced composite suction caisson foundation is much higher than the uplift bearing capacity of the traditional suction caisson foundation. And the uplift bearing capacity increases gradually with the increase of additional load and reinforcement range. The interface shear strength of caisson-cement soil will increases with the increase of normal stress and cement penetration ratio. The ring rib width is wider, the area of the shear zone above the ring rib is larger. The bearing capacity composition and the calculation method of the interfacial shear strength are proposed for analyzing the uplift bearing characteristics of the gravitational reinforced composite suction caisson foundation. It can provide a reference for the engineering design of the gravitational reinforced composite suction caisson foundation under vertical load.

Keywords Suction caisson foundation · Uplift bearing capacity · Interface shear strength · Improvement technology

1 Introduction

Suction caisson foundation is a closed-top steel tube that is lowered to the seafloor, allowed to penetrate the bottom sediments under its own weight, and then pushed to full depth with the differential pressure produced by pumping water out of the interior. Due to their easy installation, reusability, and low construction costs, suction caissons have been increasingly used in offshore engineering. As the anchoring foundation of the tension leg platform (TLP), suction caisson foundation is mainly subjected to the vertical pullout loads. The uplift bearing capacity of caisson is only composed of the internal and external friction and caisson weight under the drainage condition, so the uplift bearing capacity is very small. At present, there is no report on how to improve the uplift bearing capacity of suction caisson foundation. So, it has important theoretical significance and engineering application value to explore the bearing capacity improvement technology of caisson.

In order to accurately explore the uplift bearing characteristics of suction caisson foundation, there are many scholars have done a lot of research on the uplift bearing capacity of suction caisson foundation, such as Byrne and Houlsby [1], Luke [2], Chen and Randolph [3], Rao et al.[4], Singh et al.[5], Deng and Carter [6]. The uplift bearing capacity of the suction caisson foundation with respect to aspect ratio, uplift rate, and types of soil has been investigated in these studies. Byrne and Finn [7] investigated the effect of uplift velocities on the breakout force of caissons. The breakout force and suction were found to increase with the pullout velocity. Lehane et al.[8] examined the influence of loading rate for embedded tower footings in clay in a series of

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Table1 Soil parameters

w_c(%)	γ(kN/m^3)	w_L(%)	w_p (%)	I_P	S_u(kPa)
40.2	16.8	46.6	28.8	17.8	6.2

Where the *wc* is the water content rate of soft clay

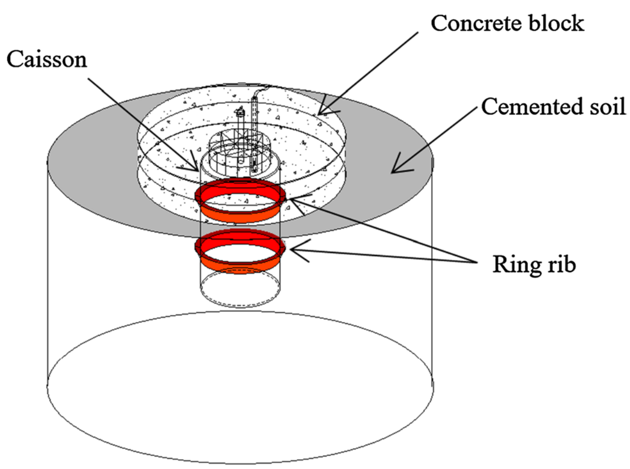


Fig.1 Gravitational reinforced composite suction caisson foundation (New caisson)

centrifuge footing pullout tests. Chen et al.[9] presented an investigation into the uplift resistance of caisson, including the effects of loading rate and skirt length, in a series of centrifuge tests. Jiao et al.[10] carried out model tests for suction caisson foundation with different ratios and pullout rates. Shi et al.[11], Feng et al.[12], Du et al.[13], and Guo et al.[14] concluded that the uplift bearing capacity of suction caisson foundation under high pullout rate is apparently higher than that under low pullout rate. Rao et al.[4] studied the uplift bearing capacity of suction caisson foundation under different pullout rates and analyzed the composition of the uplift bearing capacity. Iskander et al.[15], Acosta-Martinez.[16] from the test results concluded that the passive suction developed at the bottom of the caisson could be taken as part of the uplift bearing capacity. According to the centrifuge test, Acosta-Martinea et al.[17], Mana et al.[18], Lehane et al.[8], Zhu et al.[19], and Chen et al. [9] also proposed that the negative pressure contribution to the uplift bearing capacity of suction caisson foundation under the pullout loading. Clukey and Morrison[20] concluded that about 50–60% of the uplift bearing capacity is provided by the resistance at the bottom of the caisson. Zhu et al.[21] and Wang[22] performed a theoretical studies of the vertical uplift capacity of suction caisson under undrained pullout load. Deng et al.[6] proposed the formulas to calculate the bearing capacity of suction caisson foundation under these three failure modes through finite element and compared with the test results of Singh et al.[5] and Rao et al.[4]. These experimental results demonstrated that uplift induced passive suction plays a significant role in resisting

Table2 New caisson model parameters

No	*D/mm*	*L/D*	R /mm	Weight block /N
1#MC	75	3.0	0	0/60/120
2#MC			125	0/60/120
3#MC			150	0/60/120
4#MC			200	0/60/120

Where the *R* is the reinforcement radius of cement soil. the *D* is the caisson diameter. the *L* is the caisson length

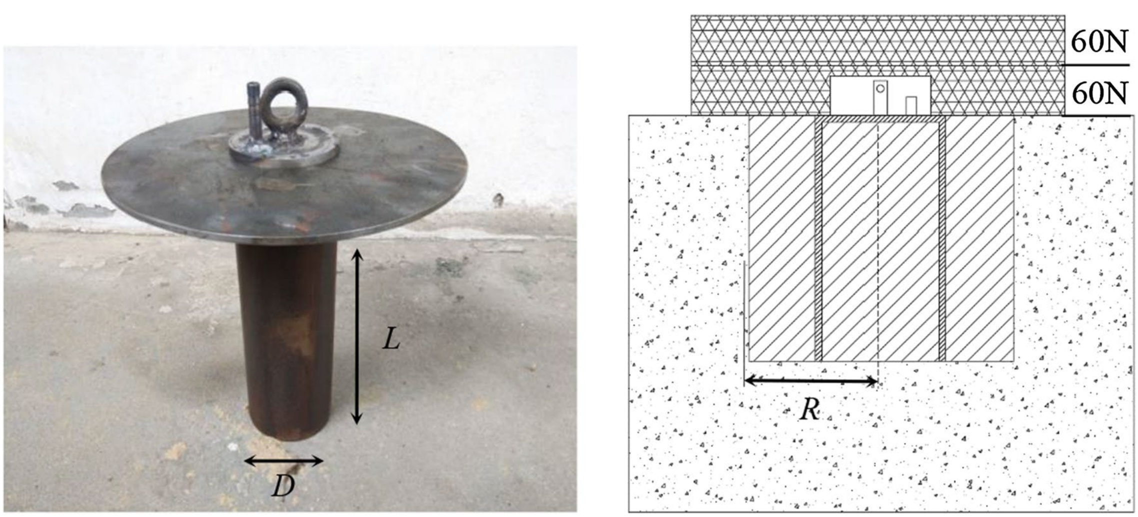


Fig. 2 New caisson model

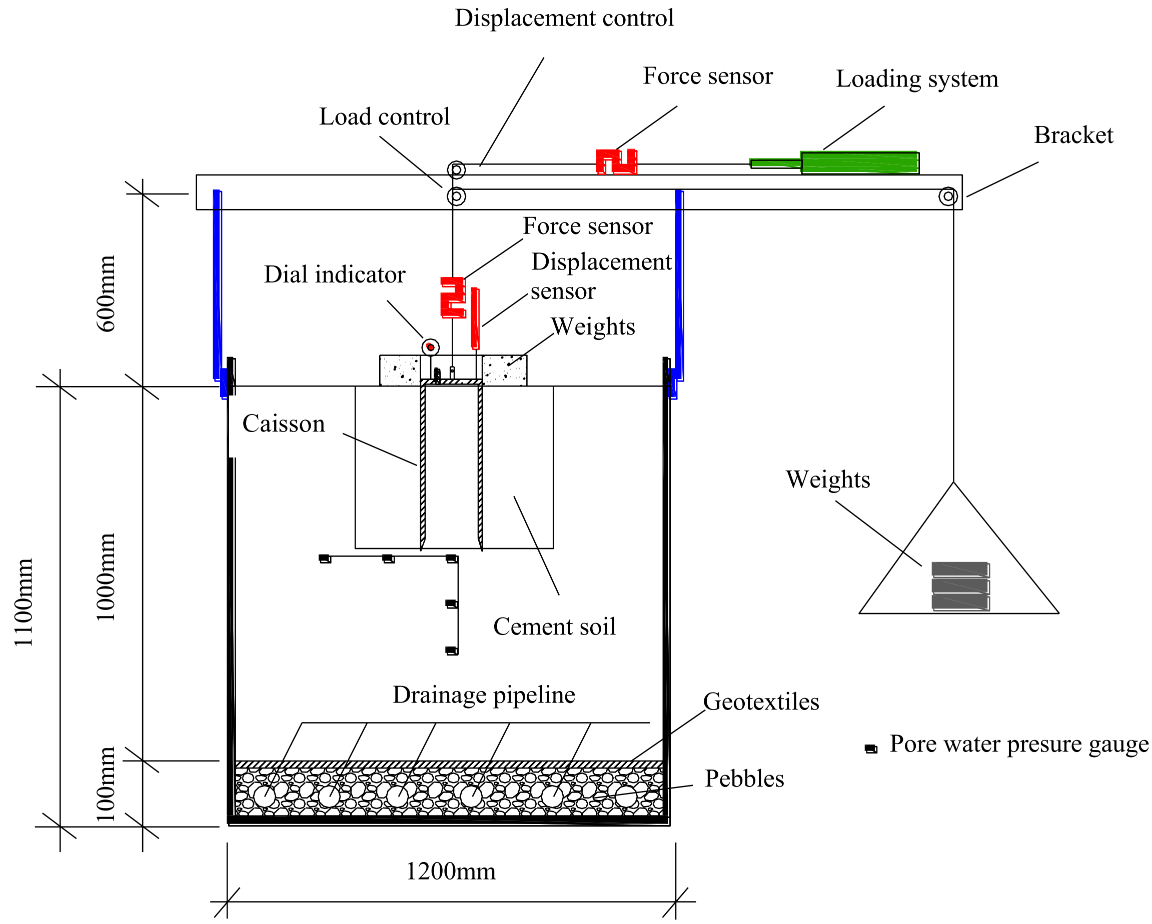


Fig. 3 Sketch of new caisson model test

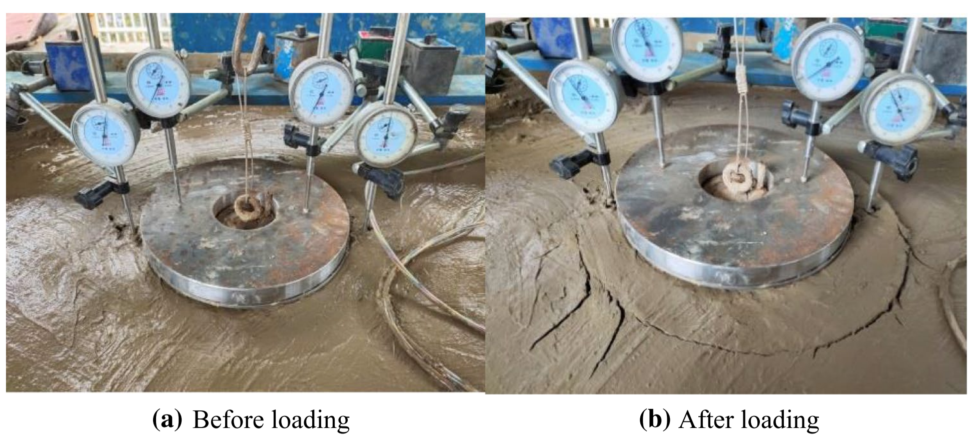
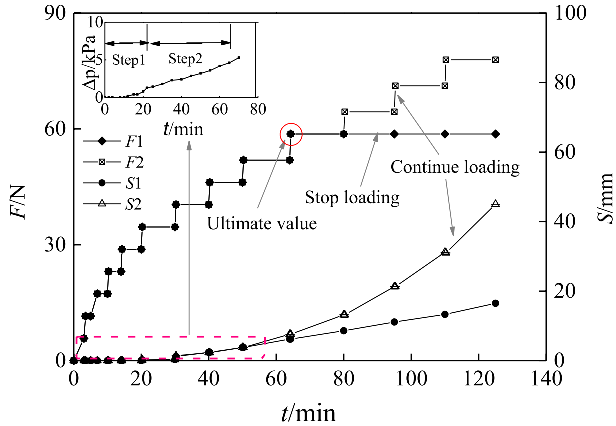


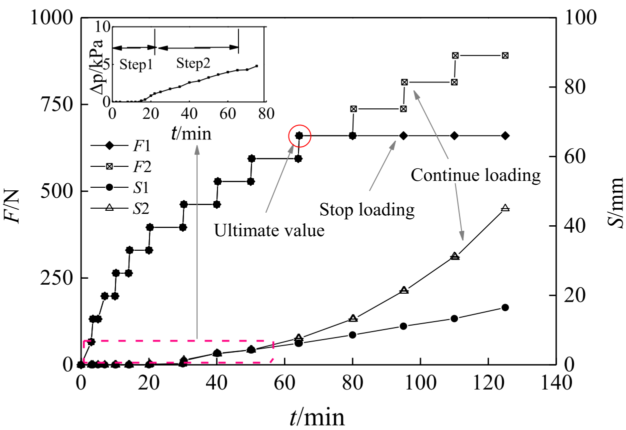
Fig. 4 New caisson model test **a** Before loading, **b** After loading

uplift loading. If the passive suction can be maintained for a longer period, the contribution from the suction will be calculated as part of the uplift resistance.

The uplift bearing capacity is the key problem to the design of caisson. However, the above researches on the uplift bearing capacity and the interface shearing characteristics of caisson have not fully revealed the vertical uplift bearing mechanism of suction caisson foundation. At the same time, there is no corresponding research bearing capacity improvement technology. So, the gravitational reinforced composite suction caisson foundation is proposed to solve this problem. Based on a specially designed experimental system, a series of model tests have been performed to investigate the pullout mechanism of the gravitational reinforced

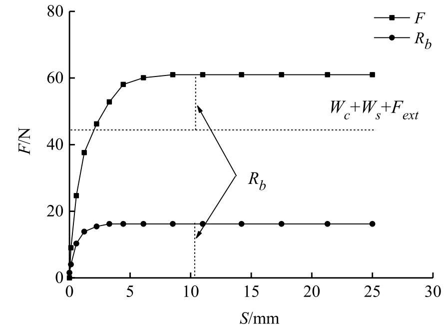


(a) 1#MC caisson model

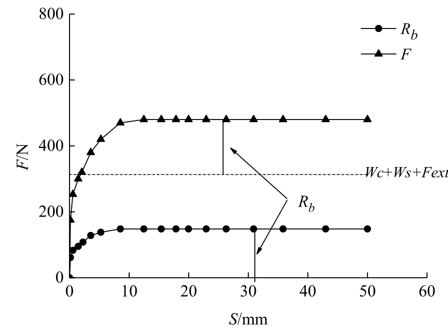


(b) 3#MC caisson model

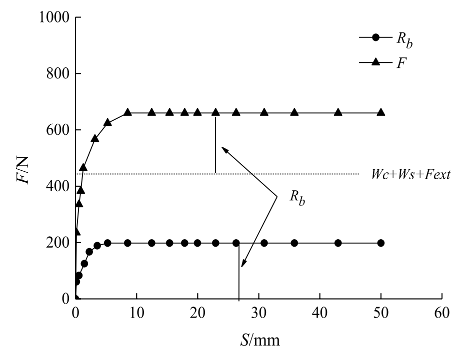
Fig. 5 Load and displacement–time curve **a** 1#MC caisson model, **b** 3#MC caisson model



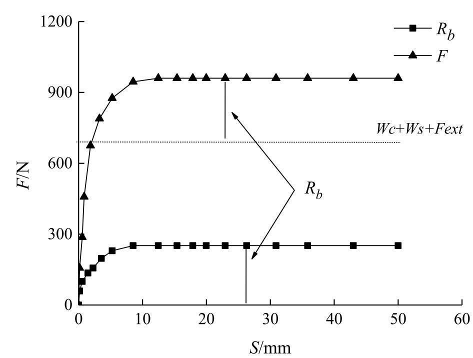
(a) 1#MC (*R*=0mm)



(b) 2#MC(*R*=125mm)



(c) 3#MC (*R*=150mm)



(d) 4#MC (*R*=200mm)

Fig. 6 Load–displacement curve **a** 1#MC ($R=0$ mm), **b** 2#MC($R=125$ mm), **c** 3#MC ($R=150$ mm), (d)4#MC ($R=200$ mm)

composite suction caisson foundation in soft clay. The composition of the uplift bearing capacity and interface shearing characteristics of the gravitational reinforced composite suction caisson foundation are analyzed in detail. It can provide a reference for the engineering design of the gravitational reinforced composite suction caisson foundation under vertical load.

2 New Caisson Model Test

2.1 Tank and Soil

A test tank, which is 1.2-m-long, 1.2-m-wide, and 1.2-m-high, is used to deposit the test soil. The soft clay was prepared by slurry sedimentation method in a small

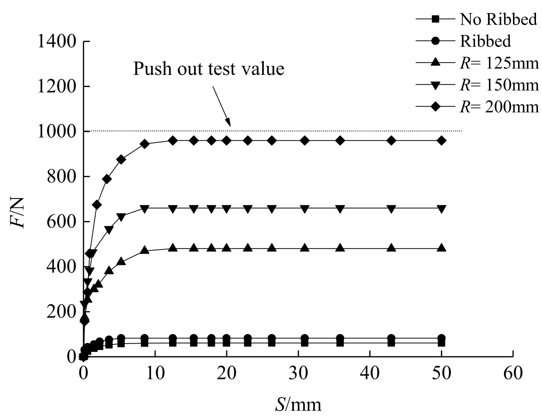


Fig. 7 Load–displacement curve

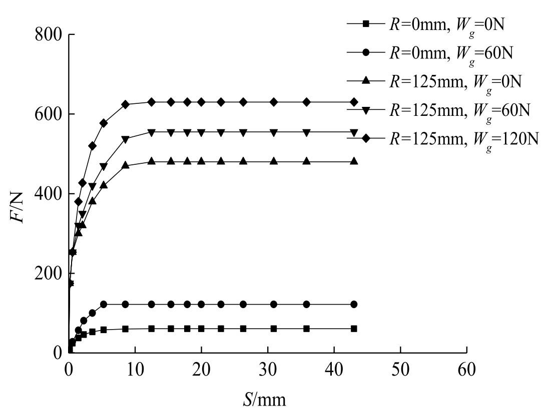


Fig. 8 Load–displacement curve

bucket, and then pour it into the test tank. The initial water content of prepared slurry was about 50–60%. To accelerate the consolidation of the slurry, the drainage pipeline and pebble were deployed at the bottom of the tank. Meanwhile, the geotextile were placed on the pebbles surface. When the undrained shear strength of the soft clay reaches 6 ~ 10 kPa along the depth, starting the test. The soil parameters measured by the geotechnical test are shown in Table 1.

2.2 New Caisson Model

In order to improve the uplift bearing capacity of the traditional suction caisson foundation, the gravitational reinforced composite suction caisson foundation is proposed in this paper. It can be seen from Fig. 1 that the foundation is made up of three parts. The first part is the suction caisson foundation; the second part is the cemented soil consolidation; the third part is the concrete block. The new caisson model is made up of the steel. The reinforcement range and caisson model parameters are shown in Fig. 2 and Table2. Linear variable differential transformer (LVDT) was placed vertically on the caisson lid to measure the uplift displacement. A load cell (range from − 500 to 500 N) was used to measure the uplift load. In order to get the development of negative pressure at the bottom of caisson, the BWMK pore water pressure gauge is arranged at the bottom of caisson, respectively. The arrangement of the test device and pore water pressure gauges are shown in Fig. 3. All test data were automatically obtained using a data acquisition system.

2.3 Loading Pattern

The model test adopts the loading mode, each stage load is one-tenth of the estimated value of the ultimate bearing capacity. When the displacement of caisson is not stable, stop the tests. Plate pullout test was carried out to measure the external friction of caisson. At the same time, pullout test

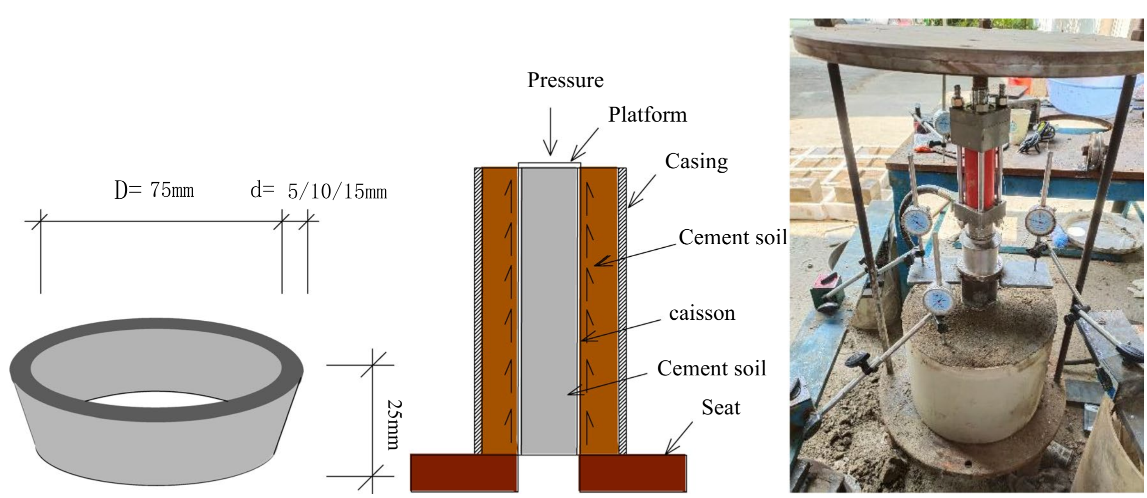


Fig. 9 Pushout test

Table3 Pushout test parameters

a_w/%	d/mm	w/%	R/mm	D/mm	L/mm
8	0/5/10/15	60	150	75	230
12	0/5/10/15	60	150	75	230
16	0/5/10/15	60	150	75	230
20	0/5/10/15	60	150	75	230

Where the a_w is the cement penetration ratio. the w is the water cement ratio

Table 4 Unconfined compressive strength

a_w	8%	12%	16%	20%
q_u(kPa)	362	864	1488	1970

of the unsealed suction caisson was carried out to measure the internal friction.

2.4 Test Results of the New Caisson

Figure 4 shows the test results of the new caisson model. It can be seen that the caisson and cement soil are pulled out as a whole from soft clay. Because the friction between the caisson and cement soil is greater than that the friction between cement soil and soft clay. The load and displacement–time curves of the 1#MC caisson model are shown as Fig. 5a. When the load is applied to the fifth stage, the uplift bearing capacity of the suction caisson foundation is made up of its self-gravity and frictional force. At this time, the caisson did not move but there was negative pressure at the bottom of the caisson. When the load is applied to the eighth stage, the negative pressure at the bottom of the caisson is 2.8 kPa. The negative pressure at the top of caisson increases gradually with the increase of load. When the load is applied to the 10th stage, the negative pressure at bottom of the caisson is 4.6 kPa, and the displacement of the caisson is no longer stable. It was found that the negative pressure at the bottom of caisson continues developing with loading increasing by another two steps after the breakout loading. The displacement–time plot of model caissons did not show any distinct failure. It can be seen that the negative pressure at the bottom of the caisson affects the uplift bearing capacity of the suction caisson foundation.

The uplift load and displacement–time curves of the 3#MC caisson model are shown as Fig. 5b. It can be found that the negative pressure at the bottom of the caisson increased with the pullout load. With the increase of load, the reversed bearing capacity at the bottom of the caisson increases gradually. The test phenomenon is the same with the 1#MC caisson model.

The composition of the uplift bearing capacity of the new caisson models is shown as Fig. 8. It can be seen that the uplift bearing capacity of new suction caisson foundation is made up of its self-gravity(W_c), cement soil(W_s), the frictional force(F_{ext}) of outside wall and the reverse bearing capacity(R_b) at the bottom of new caisson (Fig. 6).

The bearing capacity comparison results of new caisson model are shown in Fig. 7 and Fig. 8. The uplift bearing capacity of new caisson mode is much higher than uplift bearing capacity of traditional suction caisson foundation. And the uplift bearing capacity of new suction caisson foundation increases with the increase of reinforcement scope. It can be seen that the uplift bearing capacity of caisson increases with the increase of additional weight. Therefore, the uplift bearing capacity of suction caisson foundation can be improved by increasing the additional load.

3 Pushout Test of the New Caisson Model

The interface shear strength between caisson foundation and cement soil directly determines the uplift bearing capacity and reinforcement range of suction caisson foundation. In order to study the interface shear strength, the pushout test of caisson model is carried out in this paper. The pushout test is shown in Fig. 9. And the test scheme is shown in Table3. Through the unconfined compressive strength test, the unconfined compressive strength of cement soil under different cement reference ratios is shown in Table 4.

Figure 10 shows the results of pushout test of caisson model with different ring rib widths. It can be seen from the figure that there are two kinds of failure surfaces, one

Fig. 10 Pushout test results

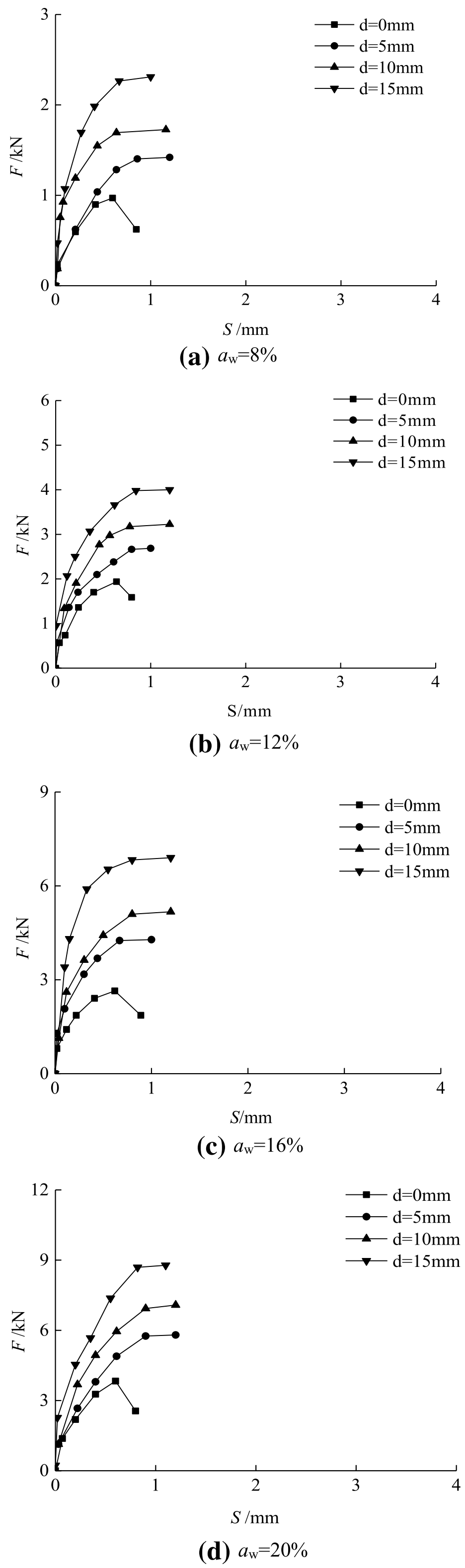


Fig. 11 Pushout test results **a** aw = 8%, **b** aw = 12%, **c** aw = 16%, **d** aw = 20%

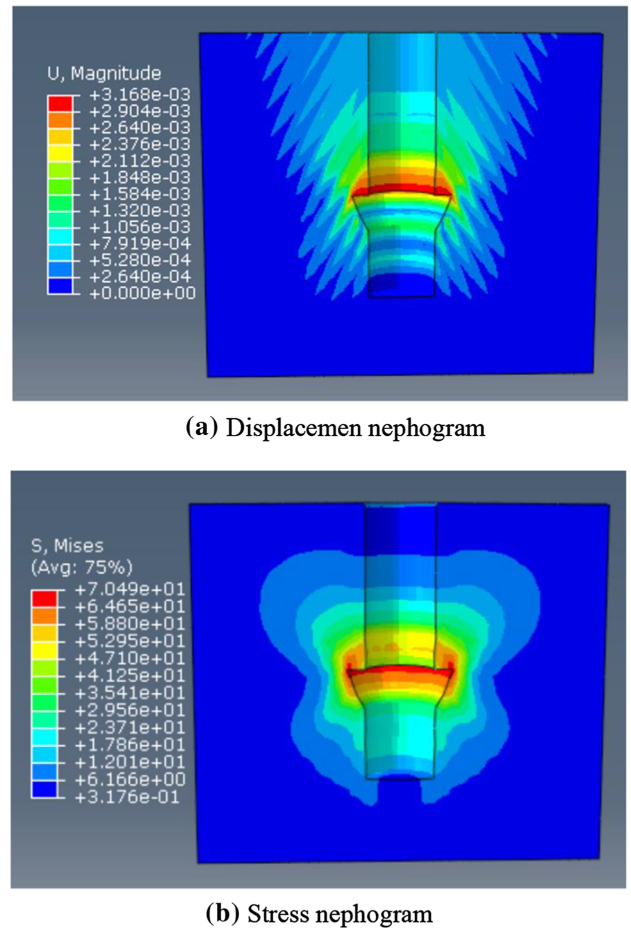


Fig. 12 Finite element analysis results **a** Displacemen nephogram, **b** Stress nephogram

is the failure surface of caisson-cement soil interface, the other is the inverted triangular annular shear failure surface at the top of ring rib. And the shear zone area increases with the increase of rib width. Figure 11 shows the relationship between pushout force and displacement in pushout test. Under the same cement ratio, the ring rib section is larger, the bearing capacity of new caisson model is higher. The bearing capacity of new caisson model increases with the increase of cement ratio.

In ABAQUS, the caisson model had a length of 230 mm, a diameter of 75 mm, and a wall thickness of 5 mm. The Mohr–Coulomb constitutive model was employed for the steel material. To determine the dimensions of the calculation domain, the radius of the model was taken as 10 times the caisson radius in the radial direction, and the height

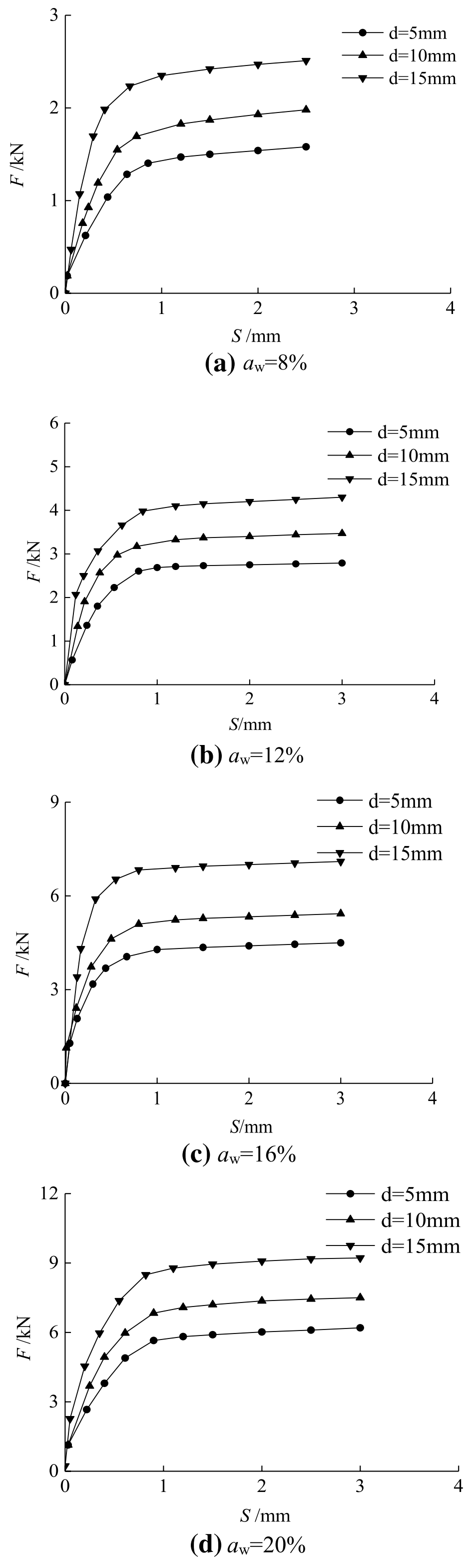


Fig. 13 Analysis of finite element results **a** $a_w = 8\%$, **b** $a_w = 12\%$, **c** $a_w = 16\%$, **d** $a_w = 20\%$

of the model was taken as twice the caisson height in the depth direction. Horizontal constraints were imposed on the vertical boundary, both vertical and horizontal constraints were applied at the bottom of the model, and symmetric constraints were used for the axisymmetric boundary. According to the test, the interface friction coefficient is 0.04. Other parameters in the finite element are refer to the test values. The FE model was established using the C3D8 solid element.

It can be seen from Fig. 12 that there is an inverted triangle shear band above the ring rib, which is more consistent with the experimental phenomenon. Figure 13 shows the finite element results. Under the same cement soil ratio, the ring rib section is larger, the bearing capacity of new caisson model is higher. The bearing capacity of new caisson model increases with the increase of cement ratio. This finite element results is the same as the test results.

4 Interfacial Shear Characteristics of the New Caisson

Through pushout tests, the failure mode of the ribbed caisson model is as shown in Fig. 14. It's interface failure can be divided into two main stages. In the first stage, before the first crack failure surface is formed, the displacement of Z1 is relatively small, and the bearing capacity increases linearly with the displacement. In the second stage, when the first crack failure surface (A2) is formed, the inflection point will appear on the load–displacement curve. At this time, the displacement is Z2, and the cement soil on top of the ring rib reaches the shear strength.

Figure 15 shows the results of shear test of cement soil. It can be seen that the normal stress is larger, the shear strength is higher. Therefore, the relationship of interfacial shear strength is established as the following:

$$\tau_1 = 0.2\left(\frac{\sigma}{q_u}\right)^{0.34} q_u \tag{1}$$

Shear strength of cement soil is established as the following:

$$\tau_2 = \sigma \tan \varphi + c \tag{2}$$

Therefore, combined with formula (1) and formula (2), the formula (3) of overall shear strength of caisson model with ring ribs is proposed as the following:

$$\tau = (1 - \alpha)\tau_1 + \alpha\tau_2 \tag{3}$$

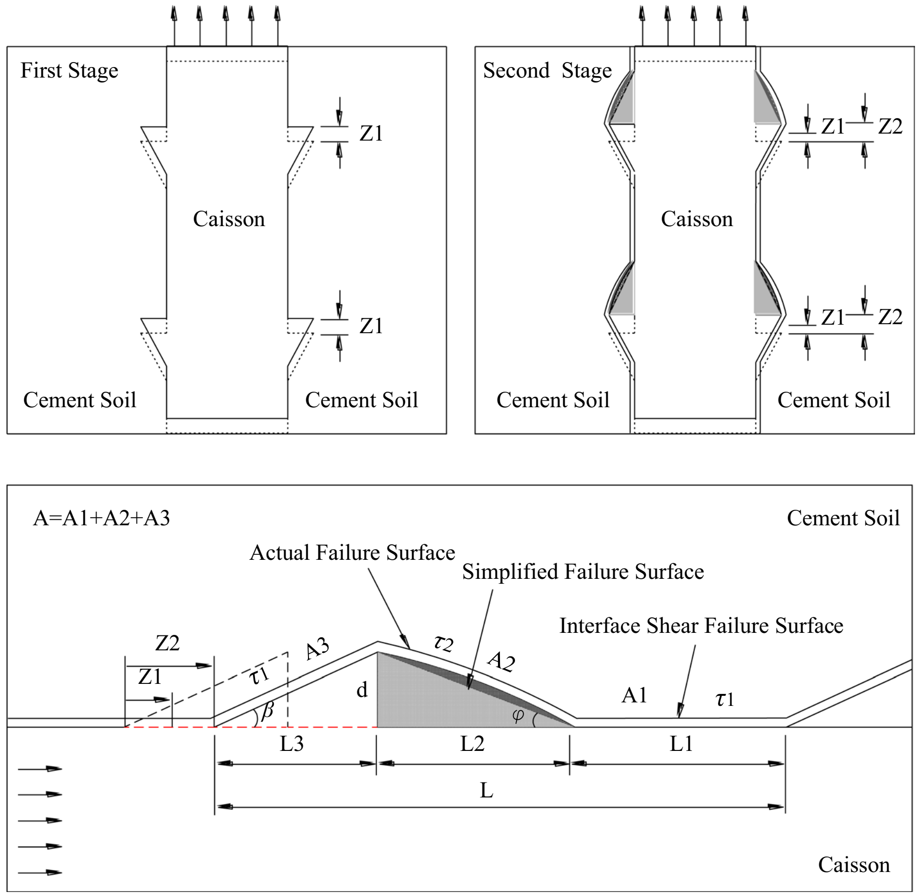


Fig. 14 Failure mode of caisson foundation interface

When the friction angle of cement soil is equal to the inclination angle of ring rib, the α can be obtained by:

$$\alpha = 1/\left[\begin{array}{l} L/d\cos(\pi/4-\varphi/2)\tan(\pi/4-\varphi/2) \\ +2-2\cos(\pi/4-\varphi/2) \end{array}\right] \quad (4)$$

Figure 16a shows the relationship between the overall shear strength and the unconfined compressive strength of cement soil. It can be seen from Fig. 16a that the strength of cement soil is higher, the overall shear strength is greater. And the Ring diameter is bigger, the overall shear strength is greater. Figure 16b shows the relationship between the overall shear strength and the friction coefficient. It can be seen from Fig. 16b that the friction coefficient is bigger, the overall shear strength is greater. Table5 shows the comparative results of the test results and the calculated results. It can be found from the table that the maximum error between the test results and the theoretical value is 16%, and the minimum error is 1%. It can be seen that the formula of overall shear strength is more reasonable.

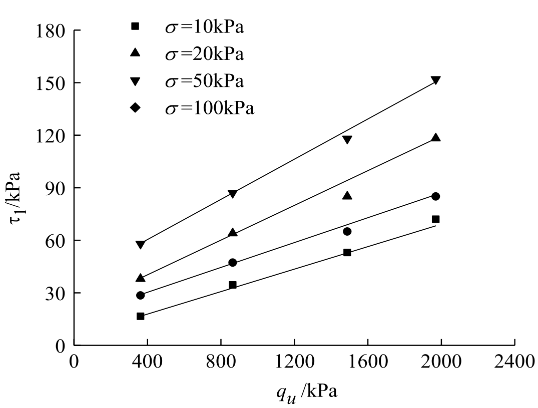
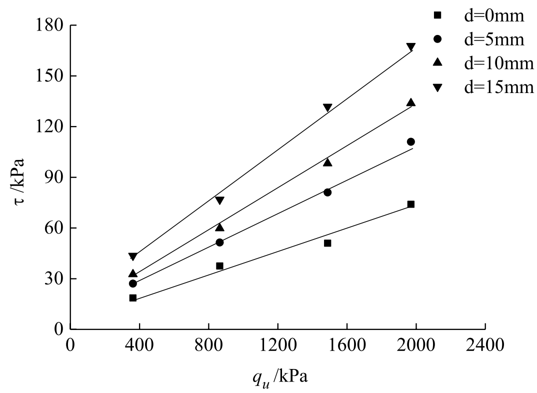


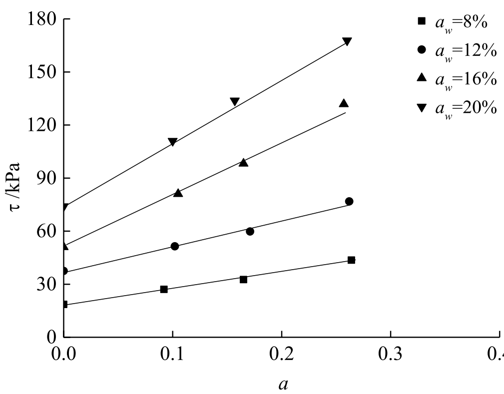
Fig. 15 Interface shear strength

5 Conclusions

Suction caisson foundations are often subjected to vertical pullout loads, but the uplift bearing capacity of caisson is very small. So, a new suction caisson foundation, gravitational reinforced composite suction caisson foundation, is proposed to solve this problem. The uplift bearing capacity of the new suction caisson foundation and the shearing characteristics of caisson-cement soil interface were studied



(a) τ–q_u



(b) τ–a

Fig. 16 Analysis of overall shear strength **a** τ–q_u, **b** τ–a

through the model test and pushout test. The main conclusions are as follows:

(1) The composition of the uplift bearing capacity of the new suction caisson foundation is made up of its self-gravity, cement soil, the frictional force of outside wall, and the reverse bearing capacity at the bottom of new caisson.
(2) The uplift bearing capacity of the new suction caisson foundation is much higher than the uplift bearing capacity of the traditional suction caisson foundation. And the uplift bearing capacity of new caisson increases with the increase of reinforcement scope. Meanwhile, the uplift bearing capacity of the new caisson increases with the increase of additional weight.
(3) It can be known that the strength of cement soil is higher, the overall shear strength is greater. And the ring rib diameter ratio is bigger, the overall shear strength is greater. The friction coefficient is bigger, the overall shear strength is greater.
(4) Combined with the uplift bearing characteristics of new caisson by model test and pushout test, the interface failure mode, overall shear strength, and the bearing capacity composition are proposed for analyzing the uplift bearing capacity of the new caisson. It can provide a reference for the engineering design of the gravitational reinforced composite suction caisson foundation under vertical load.

Table5 Comparison of results

a_w	d	τ_1	τ_2	a	τ^a	τ^b	$F^a{}_{ext}$	$F^b{}_{ext}$	*error*
8/%	0	16.6	98.0	0	16.6	18.6	0.86	0.96	0.12
	5	16.6	98.0	0.092	24.1	27.1	1.25	1.40	0.12
	10	16.6	98.0	0.161	29.6	32.6	1.53	1.69	0.10
	15	16.6	98.0	0.258	37.6	43.6	1.95	2.26	0.16
12/%	0	34.5	180.8	0.0	34.5	37.5	1.79	1.94	0.09
	5	34.5	180.8	0.102	49.4	51.4	2.56	2.66	0.04
	10	34.5	180.8	0.171	59.3	59.8	3.07	3.10	0.01
	15	34.5	180.8	0.262	72.8	76.8	3.77	3.98	0.05
16/%	0	53.0	282.0	0	53.0	51.0	2.75	2.64	-0.04
	5	53.0	282.0	0.105	77.0	81.0	3.99	4.20	0.05
	10	53.0	282.0	0.158	89.2	98.2	4.62	5.09	0.10
	15	53.0	282.0	0.257	111.8	131.8	5.79	6.83	0.18
20/%	0	72.0	402.0	0.0	72.0	74.0	3.73	3.83	0.03
	5	72.0	402.0	0.1	105.0	111.0	5.44	5.75	0.06
	10	72.0	402.0	0.157	123.8	133.8	6.41	6.93	0.08
	15	72.0	402.0	0.26	157.8	167.8	8.18	8.69	0.06

τ^a is the calculated value; τ^b is the whole shear strength obtained by pushout test; $F^a{}_{ext}$ is the calculated value; $F^b{}_{ext}$ is the pushout force

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