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점성토에 근입된 앵커들의 상향 인발시 흡입효과

Suction Effect during Pullout of Anchors in Clay

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Abstract

Laboratory model test results for uplift capacity of a circular plate anchors embedded in saturated clayey soils have been presented. Clayey soils used in this study are kaolinite and montmorillonite. Suction effects on the ultimate uplift capacity of plate anchors with respect to various embedment depths of anchor have been considered.

요　지

포화 점성토에 근입된 원형 평판 앵커의 상향 저항력에 대한 실내 모형 시험결과를 제시하였다. 오랜 기간동안 연약 포화점토에 근입되어 있는 평판 앵커에 상향력이 작용할 때 평판 앵커위의 흙은 압축되어지는 반면에 아래의 흙은 응력이 경감된다. 그래서 앵커평판위는 과잉간극수압이 증가하고 앵커평판 아래는 감소한다. 본 연구에 사용된 점성토들은 고령토와 먼트몰로나이트이다. 점토광물의 형태, 함수비 그리고 유동지수 및 근입깊이에 따른 얕은 원형 평판앵커에 대한 진흙 흡입력의 변위성을 평가하였고 평판 앵커의 근입심도에 따른 극한 상향 저항력에 미치는 흡입효과에 대해서 연구하였다.

1. Introduction

In many instances, earth anchors are used in ocean sediment to construct mooring systems during exploration and utilizatlon of ocean resources. During the past two decades, a number of papers have been pubilished which relate to the ultimate and allowable uplift capacity of shallow and deep plate anchors embedded in sand and saturated clayed soils. A comprehensive summary of those studies was provided by DAS (1990). However, several facts in the anchor uplift are not yet clearly understood and need further investigation. When an uplifting force is applied to the anchor ,the soil above the anchor plate is compressed while the soil below is relieved from stress. Hence there will be an increase in the pore water presseure above the anchor while a decrease pore water pressure is encountered below it (Fig. 1). The difference results in a suction force which is commonly referred to as the mud suction force in a general sense (Vesic, 1971; Baba, Gulhati and

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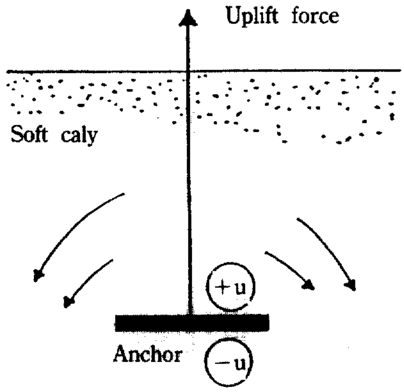

Fig. 1. Mud suction force

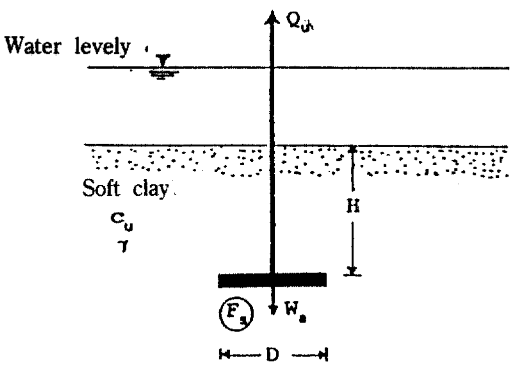

Fig. 2. Circular anchor embedded in soft clay

Datta,1989).

The purpose of this paper is to present some recent laboratory model test results conducted to evaluate the nature of variation of the mud suction force for shallow circular plate anchors with (a) type of clay mineral, (b) moisture content and thus liquidity index, and (c) depth of embedment.

2. Ultimate Uplift Capacity Parameters

Figure 2 shows a circular plate anchor of diameter D embedded in a soft saturated clay at a depth H below the ground surface. The undrained shear strength of the clay is C_u. If the embedment ratio H/D is relatively small, at ultimate load the failure surface located in soil above the anchor plate extends to the ground surface. This is referred to as shallow anchor condition. However, at large embedment ratios, the failure in soil takes

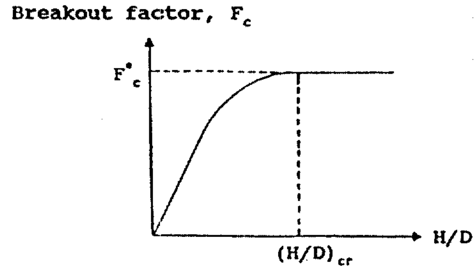

Fig. 3. Nature of variation of breakout factor with embedment ratio

place around the anchor and it does not extend to the ground surface. This is called deep anchor condition.

The ultimate uplift capacity of anchors can be expressed as

$$Q_u = Q_n + W_a + F_s \tag{1}$$

where Q_u and Q_n = gross and net ultimate uplift capacity, respectively; W_a = effective self weight of the anchor; and F_s = mud suction force.

The net ultimate uplift capacity in a c-ϕ soil can be given by the relation (Vesic, 1971)

$$Q_n = A(C_u F_c + \gamma H \ F_q) \tag{2}$$

where A = area of the anchor plate; C_u = undrained shear strength; F_c, F_q = breakout factors, which is a nondimensional quantity; γ = unit weight of soil; and H = depth of embedment.

For undrained condition, $\phi = 0$, $C = C_u$ and the value of F_q is equal to one. Thus the preceding relation can be expressed as

$$Q_n = A(C_u F_c + \gamma H) \tag{3}$$

For square or circular anchors, the breakout factor increases with H/D up to about 9 (shallow anchor condition) and remains constant thereafter (deep anchor condition). The embedment ratio H/D at which Fc reaches the maximum value (F^*) may be referred to as the critical embedment ratio, $(H/D)_{cr}$, as shown in Fig. 3. For square or circular anchors (Das, 1978)

$$(H/D)_{cr} \approx 0.107 C_u + 2.5 \leq 7 \tag{4}$$

where C_u is in KN/m^2.

In soft saturated clays, the mud suction force

Table 1. Properties of the Clay Soils

Clay	Item	Quantity
Kaolinite	Grain size:	
	percent passing No. 200 U.S.	
	sieve (0.075 mm opening)	96
	percent finer than 0.002 mm	67
	Atterberg limits:	
	Liquid limit (%)	63
	Plasticity index (%)	27
montmorillonite	Grain size:	
	Percent passing No. 200 U.S.	
	sieve (0.075 mm opening)	98
	Percent finer than 0.002 mm	72
	Atterberg limits:	
	Liquid limit (%)	406
	Plasticity index (%)	372

can be a large part of the gross ultimate uplift capacity. However, in most investigations reported so far, the mud suction force has been eliminated by venting the bottom of the anchor plate.

3. Laboratory Model Tests

Laboratory model tests were conducted using a plate anchor made of plexiglas. The diameter, D, of the model anchor was 51 mm, and the anchor plate was 13 mm thick. For the present study, two types of clay soil (kaolinite and montnorillonite) were used. The grainsize distribution and Atterberg limits for these two clays are given in Table 1.

In order to conduct the model tests, a given soil was mixed with a required amount of water. To achieve uniform moisture distribution the moist soil was placed in several plastic bags and stored in a moist curing room for several days before use.

For a given soil, two series of tests were conducted one series in which the bottom of the anchor plate was vented and the other series in which it was not vented; Figure 4 shows a schematic diagram of the model test arrangement in the laboratory in which the bottom of the plate anchor was vented, thus eliminating the mud suction force.

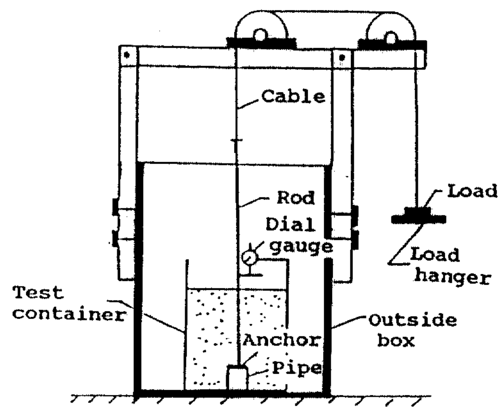

Fig. 4. Schematic diagram of the model test arrangement with the bottom of the anchor plate vented

The test were conducted in a tank measuring 350 mm in diameter and 500 mm height. In conducting a test, the model anchor was placed over a Plexiglas pipe. A rigid shaft having a diameter of 6 mm was attached to the model anchor. Moist soil from the plastic bags was placed in the test tank and compacted in 25-mm thick layers up to the desired height. The test was performed about 24 hours later after installation of anchor in compacted clay. The top of the rigid shaft was attached to a cable which passed over two pulleys fixed to a rigid frame. A load hanger was attached to the other end of the cable on which step loads could be placed. The anchor movement corresponding to a given load was measured by a dial gauge. For each loading step, additional load was added when the upward displacement of anchor was very small. For a given test the loading was continued until complete pullout occurred. The undrained shear strength of the soil, Cu, was measured at the end of each pullout test by a hand vane shear test device. When tests were conducted with the bottom of the anchor plate not vented, the model anchor was placed on compacted clay and the Plexiglas tube as shown in Fig. 4 was not used. Table 2 gives the details for the series of test conducted under this program.

4. Model Test Results

The net ultimate uplift capacities (Q_n) of the

Table 2. Properties of the Clay Soils (H/D=1, 2, 3, 4, 5 and degree of saturation S_r=100 %)

Test series	Average w (%)	Average C_u (KN/m^2)	Unit weight γ (KN/m^3)	Remarks
KV-1	45.4	4.6	17.2	Kaolinite
KV-2	49.3	4.0	16.9	No Suction
KV-3	52.74	2.9	16.6	
KWV-1	45.38	4.6	17.2	Kaolinite
KWV-2	49.27	4.0	16.9	Suction
KWV-3	52.74	2.9	16.6	
MV-1	311.56	3.7	11.48	Montmorillonite
MV-2	337.0	3.2	11.37	No suction
MV-3	364.51	2.3	11.26	
MWV-1	311.56	3.7	11.48	Montmorillonite
MWV-2	337.0	3.2	11.37	Suction
MWV-3	364.51	2.3	11.26	

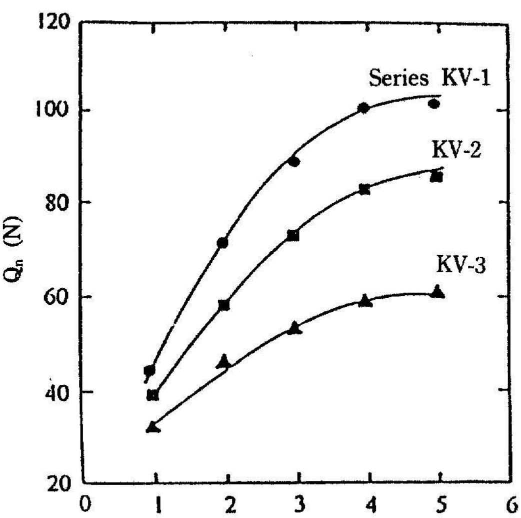

Fig. 5. Variation of Q_n with H/D in kaolinite

anchor determined from the results of test series KV and MV are given in Figs. 5 and 6. From these plots it can be seen that, for a given clay and embedment ratio (H/D), the net load increased with the decrease in the moisture content of the soil (i.e. with the increase of the undrained shear strength, C_u). Also, for any given clay the magnitude of Q_n increased with the increase in the embedment ratio. By using the experimental values of Q_n and Eq. (3), the magnitudes of the

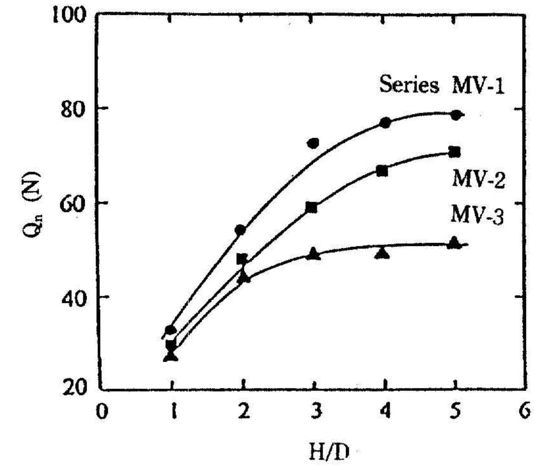

Fig. 6. Variation of Q_n with H/D in montmorillonite

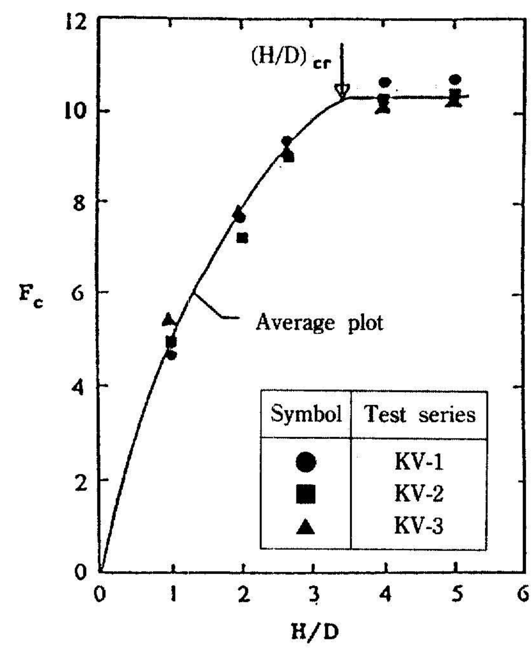

Fig. 7. Variation of F_c with H/D in kaolinite

breakout factor (F_c) were calculated and are plotted against their corresponding values of H/D in Figs. 7 and 8. Fig. 7 and Fig. 8 show the test results conducted in Kaolinite and Montmorillonite, respectively. From these plots it can be seen that:

1. For a given test series the magnitude of F_c increases with H/D to a maximum value and remains practically constant thereafter.

2. For tests in Kaolinite (Fig. 7), although there

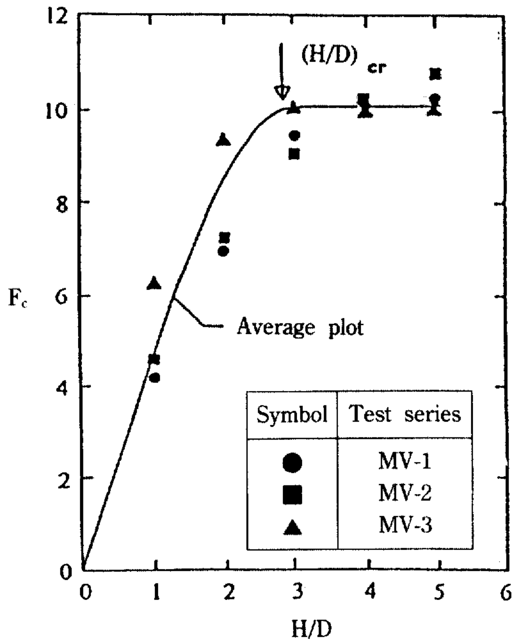

Fig. 8. Variation of F_c with H/D in montmorillonite

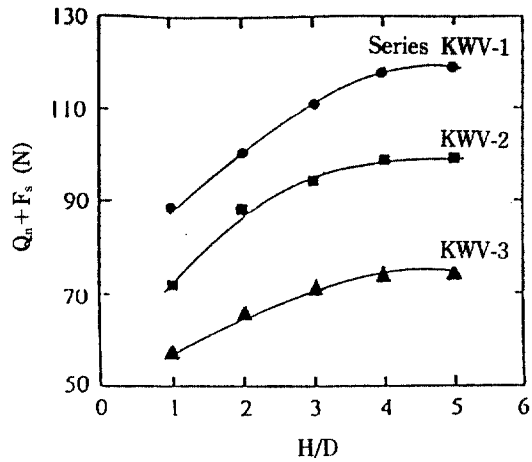

Fig. 9. Variation of Q_n+F_s with H/D in kaolinite

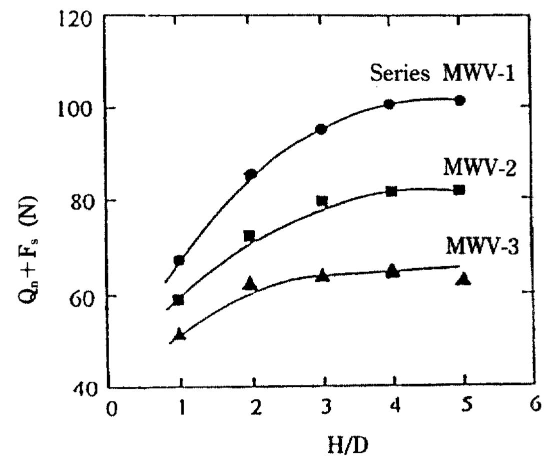

Fig. 10. Variation of Q_n+F_s with H/D in montmorillonite

is some scatter in the experimental results, the variation of F_c with H/D can be approximated by a single average curve (shown in Fig. 7). For the average curve the maximum value of F_c is about 10 and $(H/D)_{cr}$ is about 3.25. This value of $(H/D)_{cr}$ is slightly higher than that obtained using Eq. (3), $(H/D)_{cr}$ should be between 2.8 to 3.

3. For tests in montmorillonite (Fig. 8), the scattering is somewhat higher than that for the tests in kaolinate. This may be due to difficulty in working with this type of clay. Using similar rationale, an average curve for the variaticn of F_c with H/D has been plotted . For the average plot, the maxium value of F_c is about 10 and $(H/D)_{cr}$ is about 2.8.

Figures 9 and 10 show the variation of $Q_u-W_s=Q_n+F_s$ obtained from test series KWV and MWV. The trends are generally similar to those in Figure 5 and 6. Using the experimental values of Figures 5 & 9, and Figures 6 & 10, the variation of mud suction force for a given type of clay, C_u, and H/D can be calculated. One way to observe the effect of mud suction on the net ultimate uplift capacity is to plot the variation of F_s/Q_n versus H/D, as shown in Figure 11. Based on this figure the following general conclusions can be drawn:

1. The ratio of F_s/Q_n decreases with the increase in the embedment ratio. This is true for both clays and all values of undrained cohesion.

2. Although there is some scatter, as can be expected from this type of test, the variation of F_s/Q_n with H/D can be approximated by a single average curve. Based on the average curve, at H/D=1 the magnitude of F_s/Q_n is about 1 at H/D=1 and decreases to about 0.2 at H/D=5.

3. The average plot of F_s/Q_n versus H/D is not necessarily a function of the type and amount of

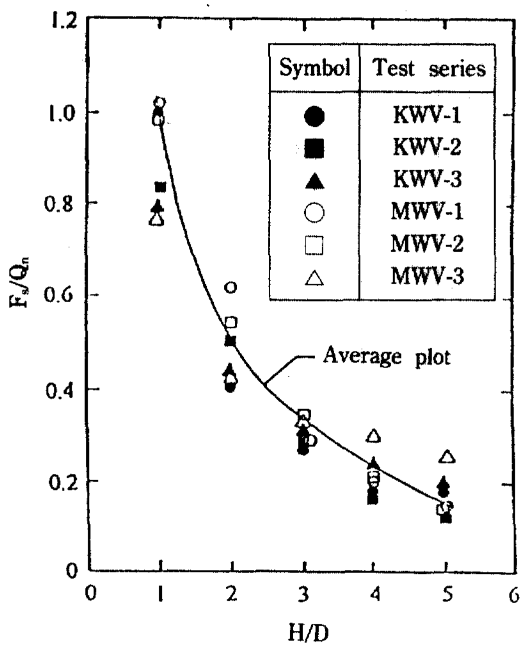

Fig. 11. Variation of F_s/Q_n with H/D

clay minerals. It is also not a function of the liquidity index(LI) which is defined as

$$LI=\frac{w-PL}{LL-PL} \quad (5)$$

where w = moisture content, LL = liquid limit, and PL = plastic limit. For the present tests in kaolinite, the magnitude of the liquidity index varied from 35% to 62%; and, for tests in montmorillonite, it varied from 75% to 89%.

5. Conclusions

The results of a number of laboratory model tests for the ultimate uplift capacity of a circular plate anchors embedded in saturated kaolinite and montmorillonite have been presented. The liquidity index for kaolinite varied from 35% to 62%, and for montmorillonite varied from 75% to 89%. The pullout tests were conducted with and without venting the bottom of the anchor plate for an embedment ratio varying from one to five. Based on the test results the following conclusions can be drawn:

1. For a given clay and moisture content, the ratio of the mud suction force to the net ultimate uplift capacity generally decreases with the increase in embedment ratio.

2. Within the limits of the model tests, the ratio of F_s/Q_n decreases from about one at H/D = 1 to about 0.2 at H/D = 5. This is true irrespective of the amount and type of clay mineral.

3. More tests of this type are needed for a better quantitative assessment of the mud suction force.

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