# Creep Effect of Shallow Plate Anchor in Soft Clay

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## 요 지

판앵커들은 각종 해양공사와 해양구조물의 유지관리작업에 빈번히 사용되고 있다. 판앵커가 연약한 점토에 근입되었을 때 지속적인 허용하중을 받게 되면 Creep현상이 발생될 수도 있다. 본 논문은 얕은 원형앵커가 순극한 상향인발력보다 작은 순하중을 지속적으로 받았을 때 시간이 경과함에 따라 발생되는 크리프효과를 연구한 실내모델실험 결과를 기술하였다. 실내모델시험 결과를 바탕으로 순하중과 상향변위율 및 경과시간사이의 상관관계식을 제시하였다.

## Abstract

Plate anchors are often used for various types of offshore construction and maintenance works. When the plates are embedded in soft clay and subjected to sustanined allowable loads, creep may develop.

This paper presents some results from laboratory model test designed to study the creep effect that develops with time for a shallow circular anchor subjected to sustained net loads that are less than the net ultimate uplift capacity. Based on the model test results, relationships among the net load, the rate of strain, and time are developed.

Keywords : Creep, Plate Anchor, Soft Clay, Short Term, Long Term

## 1. Introduction

Plate anchors are occasionally used for various types of offshore construction and maintenance works. A number of laboratory and field test results are available in the literature for the estimation of the net short-term ultimate uplift capcity of plate anchors embedded in clayey soil(e.g., Ali, 1968 ; Kupferman, 1971 ; Adams and Hayes, 1967 ; Bhatnagar,

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1969 ; Das,1978, 1980 ; and Vesic, 1971).

Fig.1 shows a circular plate anchor of diameter D embedded in a soft saturated clay at a depth H. The short−term ultimate uplift capacity ($\phi$=0 concept, where $\phi$ is the soil friction angle) of the anchor plate can be given by the following relationship :

$$Q_u = Q_o + W_a + F_s \quad (1)$$

where $Q_u$=gross short−term ultimate uplift capacity, $Q_o$=net short−term ultimate uplift capacity, $W_a$=effective self−weight of the anchor, and $F_s$=mud suction force.

The mud suction force is primarily a function of the undrained cohesion and the coefficient of permeability of the clay. The short−term net ultimate uplift capacity $Q_o$ can be expressed as(Vesic, 1971)

$$Q_o = A(\gamma H + F_c\ c_u) \quad (2)$$

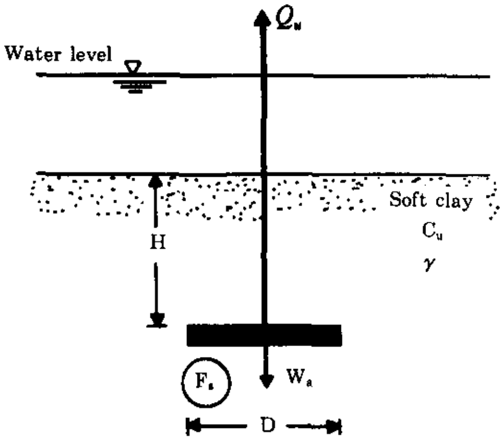


Fig. 1 Circular plate anchor embedded in soft saturated clay

where A=area of the anchor plate, $\gamma$=saturated unit weight, $F_c$=breakout factor, and $c_u$=undrained cohesion.

The breakout factor $F_c$ is a function of $c_u$ and the embedment ratio H/D. For a given clay, the breakout factor increases with the embedment ratio and reaches a maximum value(i.e., $F_c = F^*_c$) at $H/D = (H/D)_{cr}$. For $H/D \geq (H/D)_{cr}$, the magnitude of $F_c (= F^*_c)$ remains constant. According to Das(1990),

$$F_c \simeq n\left(\frac{H}{D}\right) \leq F^*_c = 9 \quad (3)$$

where n varies between 2 and 5.9 depending on the magnitude of the undrained cohesion $c_u$. For circular anchors, the variation of the critical embedment ratio $(H/D)_{cr}$ can be given by an empirical equation(Das, 1978) as

$$\left(\frac{H}{D}\right)_{cr} = 0.107c_u + 2.5 \leq 7 \quad (4)$$

where the unit for $c_u$ is in $kN/m^2$.

Anchors with embedment ratios of $H/D \leq (H/D)_{cr}$ are referred to as shallow anchors. For shallow anchors at ultimate load, the failuere surface in soil extends to the ground surface as shown in Fig. 2 ; however, for anchors with H/D greater than $(H/D)_{cr}$, the failure in soil occurs around the anchor and does not extend to the ground surface. These are referred to as deep anchors.

When a plate anchor embedded in soft saturated clay is subjected to a sustained net load $Q<Q_u$, the anchor is likely to exhibit some creep(upward movement with time). The creep rate is a function of $Q/Q_u$, $c_u$, H/D, and also the type and amount of clay minerals present in the soil. This is an important factor in the design of anchors in many offshore projects. Currently, published literature related to the evaluation of the creep rate of plate anchors embedded in soft clay is relatively scarce. This paper provides the results of a recent laboratory model study designed to evaluate creep of a circular plate anchor embedded in a soft saturated clay. The model tests were conducted in one type of clay using only one model anchor.

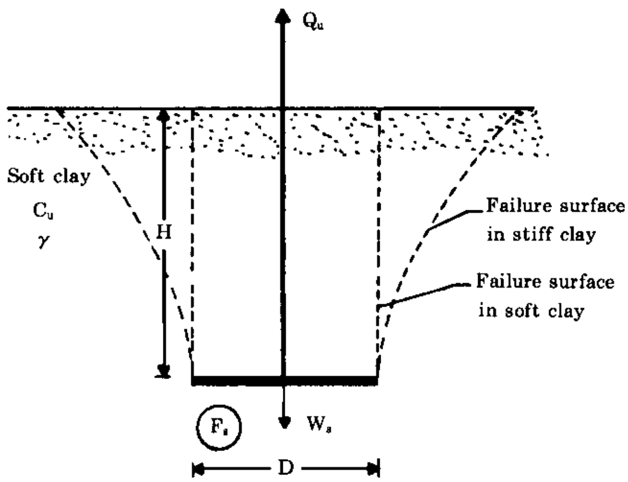


Fig. 2 Failure surface in clay above shallow anchor plate

## 2. Laboratory Model Tests

Laboratory model tests were conducted by using a plate anchor made of Plexiglas. The model anchor had a diameter D of 50mm and a thickness of 13mm. For the present study, a clayey soil was collected from the field. The grain-size distribution of the soil, determined by sieve and hydrometer analysis, showed that it had about 98% passing No. 200 sieve (0.075mm opening) and about 25% finer than 0.002mm. Other physical properties of the soil are given in Table 1.

Table 1. Physical Properties of Clay

| Item | Quantity |
|---|---|
| Grain size : | |
| Percent finer than No. 200 U.S. sieve(0.075mm opening) | 98 |
| Precent finer than 0.002mm | 25 |
| Specific gravity of soil solids | 2.74 |
| Atterberg limits : | |
| Liquid limit(%) | 43 |
| Plastic limit(%) | 23 |
| Plasticity index(%) | 20 |

In order to conduct a model test, the soil collected from the field was pulverized in the laboratory. It was then mixed with water. To achieve uniform moisture distribution, the soil was then placed in several plastic bags and stored in moist curing room for several days before use. Fig. 3 shows a schematic diagram of the model test arrangement in the laboratory. The tests were conducted in a tank measuring 350mm in diameter and 500mm in height. For conducting a test, the model anchor was placed over a Plexiglas pipe. This was done to vent the bottom of the anchor plate to eliminate the mud suction force. A rigid shaft with a diameter of 6mm was attached to the model anchor. Moist soil was placed in the test tank and compacted in 25mm−thick layers up to the desired height. The top of the rigid shaft was attached to a cable that passed over two pulleys affixed to a rigid frame. A load hanger was attached to the other end of the cable, on which step loads could be placed. A dial gauge was used to measure the anchor movement corresponding to a given load. The undrained shear strength of the soil $c_u$ was measured at the end of each

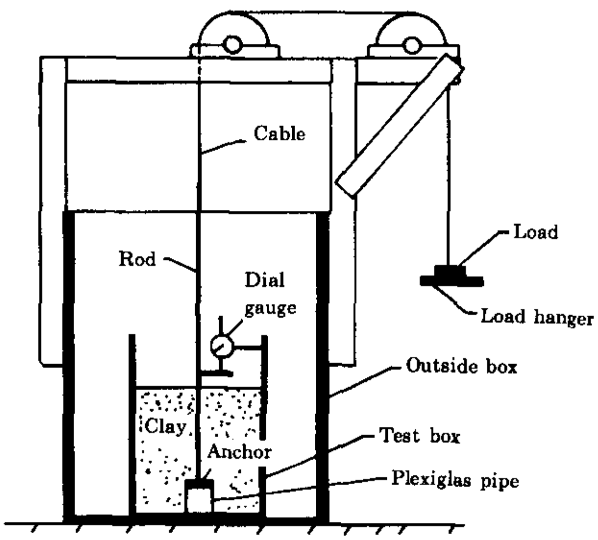


Fig. 3 Schematic diagram of laboratory model test arrangement

test by a hand vane shear test device. At least three measurements were done for each test conducted. The average value of $c_u$ was about $7kN/m^2$ with a variation of $\pm 6\%$. Two types of tests were conducted in the laboratory.

a. Short-term pullout tests were conducted to estimate the net short-term ultimate uplift capacity $Q_o$ and thus the variation of the breakout factor $F_c$ with the embedment ratio H/D. For any given test, load was applied in steps until failure occurred.

b. Creep tests were conducted to monitor the rate of anchor uplift movement, $\delta$, with time for various sustained loads Q and depths of embedment.

Table 2 provides details of the short-term pullout tests and creep tests conducted under this program as well as the average properties of the compacted clay during the tests.

Table 2. Details of Laboratory Tests

| Test type | Test details | Comments |
|---|---|---|
| Short-term pullout | $H/D=1, 2, 3, 4, 5$ | To determine net ultimate uplift capacity |
| Creep tests | $H/D=1, 2, 3, 4,$ $Q/Q_o=0.35, 0.5, 0.75$ | To determine creep rate |
| Note : Anchor : D=50mm. Soil : $c_u \simeq 7kN/m^2$, $\gamma=18.2kN/m^3$, degree of saturation=99%, moisture content $\simeq$37% | | |

## 3. Model Test Results

### 3.1 Short-Term Pullout Tests

The net ultimate uplift capacities determined from the short-term pullout tests are shown in Fig.4. From Eq. 2, the relationship for breakout factor can be derived as

$$F_c=\frac{1}{c_u}\left(\frac{Q_o}{A}-\gamma H\right) \tag{5}$$

Using Eq. 5 and the net ultimate uplift capacities shown in Fig. 4, the variation of the breakout factor versus the embedment ratio can be obtained as shown in Fig. 5. The figure shows that magnitude of F increases with H/D up to a maximum value $F=F^* \simeq 9$ at $H/D=(H/D)_{cr} \simeq 4$ to 5. According to Eq. 4, the critical embedment ratio for $c_u=7kN/m^2$ is about 3.3. However, some deviations for a given set of experimental values from an empirical relationship can always be expected. According to the present test results, the tests conducted at H/D=1, 2, 3, and 4 are for shallow anchor condition.

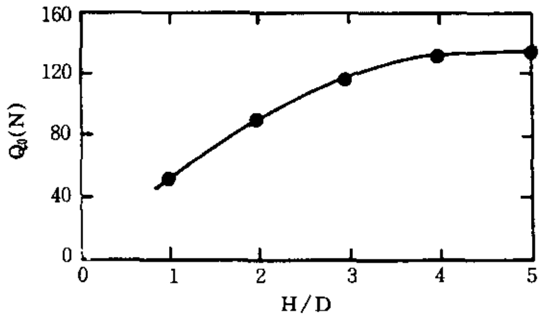


Fig. 4 Net ultimate uplift capacity versus various embedment ratios

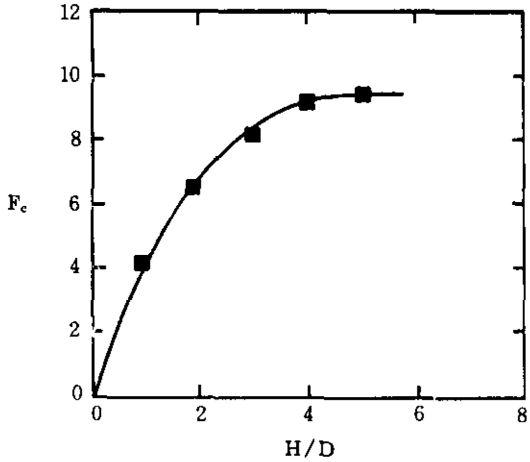


Fig. 5 Variation of breakout factor versus embedment ratio

## 3.2 Creep Tests

The creep tests were performed at H/D=1, 2, 3 and 4 for $Q/Q_o$=35%, 50% and 75%. Fig.6 shows typical creep test results at H/D=2. For any given value of $Q/Q_o$, the upward displacement of the anchor increases rapidly with time up to a certain point, after which it becomes more or less stable. Fig.7 shows a typical nature of the plot of $\delta$ versus t. The figure shows that for $t \leq t_c$ the upward movement of the anchor plate owing to the sustained load is relatively rapid and can be referred to as the primary creep. However, for $t > t_c$ the anchor movement slows down and this zone can be called the secondary creep zone.

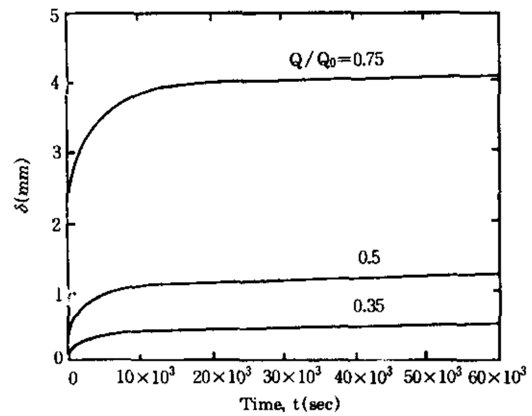


Fig. 6 Plot of displacement $\delta$ versus time for H/D=2

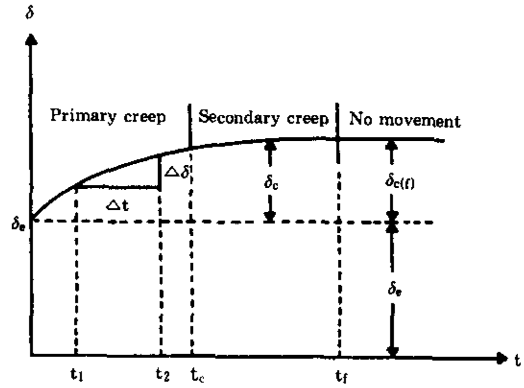


Fig. 7 Typical plot of $\delta$ versus t

At $t \leq t_f$ the upward movement of the anchor stops. As shown in Fig.7, the rate of strain $\dot{\epsilon}$ at time $t=(t_1+t_2)/2$ can be approximated as

$$\dot{\in} \simeq \frac{\Delta\,(\delta/H)}{\Delta\, t} = \left(\frac{\delta_2 - \delta_1}{t_2 - t_1}\right)\frac{1}{H} \tag{6}$$

By using the relationship described in Eq. 6, the rates of strain for all tests were calculated and plotted against time(log−log plots) in Figs. 8−11. Based on the average experimental plots, it appears that, for any given H/D,

$$\dot{\in} = \alpha_1 t^{-\beta_1} \text{ for } t \leq t_c \tag{7}$$

and

$$\dot{\in} = \alpha_2 t^{-\beta_2} \text{ for } t \geq t_c \tag{8}$$

where $\alpha_1$, $\alpha_2$, $\beta_1$ and $\beta_2$ are functions of H/D.

The time $t_c$(see Fig. 7 for definition) appears to increase with $Q/Q_o$. However, for a given value of $Q/Q_o$, $t_c$ does not change substantially as H/D changes.

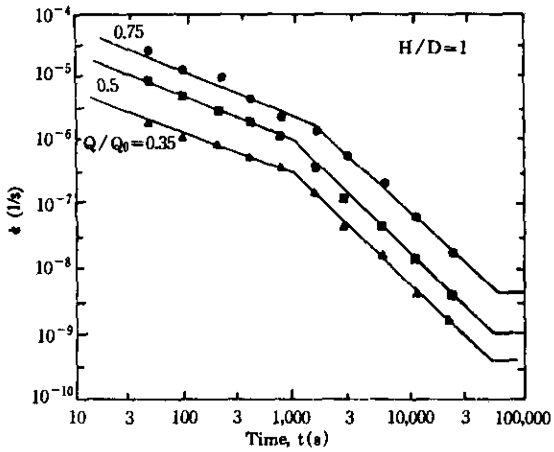


Fig. 8 Plot of ∈ versus t for H/D=1

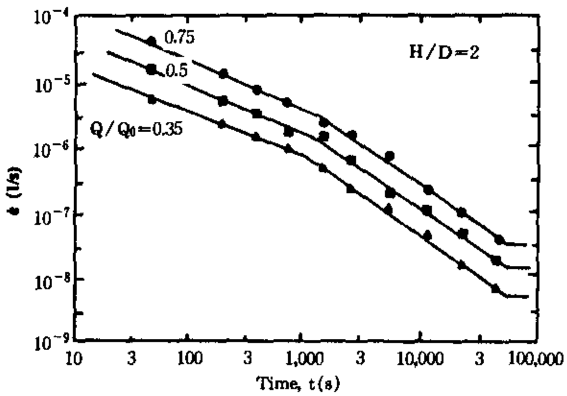


Fig. 9 Plot of ∈ versus t for H/D=2

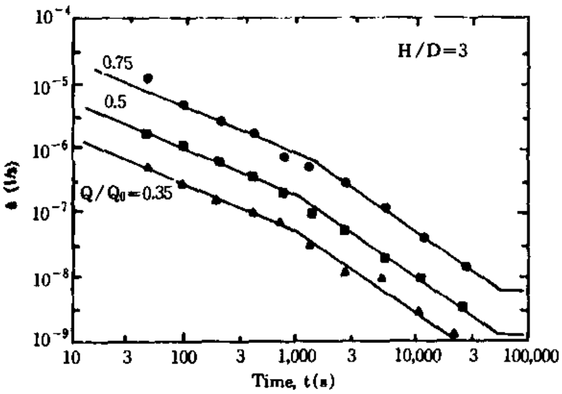


Fig. 10 Plot of ∈ versus t for H/D=3

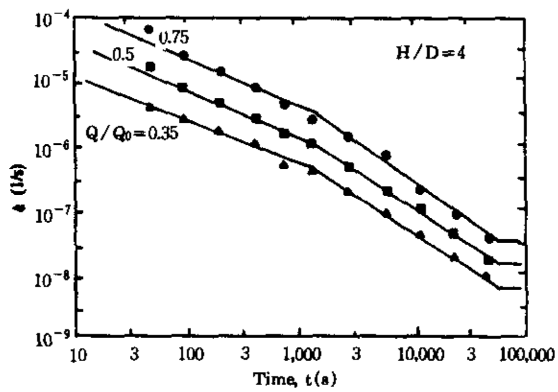


Fig. 11 Plot of ∈ versus t for H/D=4

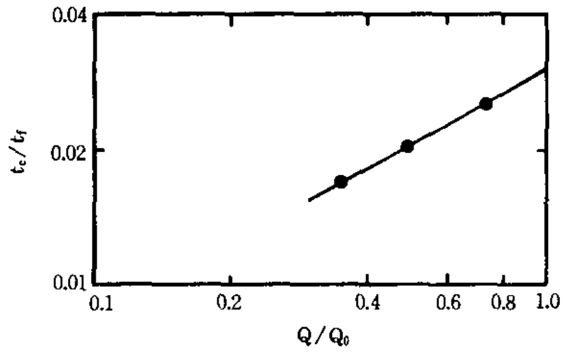


Fig. 12 Variation of $t_c/t_f$ versus $Q/Q_0$

Fig.12 shows a plot of the average values of $t_c/t_f$($t_f$=time when the creep stops, as shown in Fig. 7) versus their corresponding $Q/Q_o$. Each creep test was conducted for a 24-hr period ; however, in all cases the creep stopped after 16hrs. Hence, for the present tests, $t_f$=57,600 s. Based on Fig.12,

$$\frac{t_c}{t_f}=0.0285\left(\frac{Q}{Q_o}\right)^{0.52} \tag{9}$$

Obervations of Figs. 8~11 show that the magnitude of $\beta_1$ is practically the same irrespective of H/D and $Q/Q_o$. For the present tests,

$$\beta_1 \simeq 0.405 \text{ for t in s} \tag{10}$$

Again, for a given value of H/D the magnitude of $\beta_2$ is approximately the same irrespective of $Q/Q_o$ ; however, it decreases as H/D increases. The average values of $\beta_2$ determined from Figs. 8~11 are plotted against their corresponding H/D values in Fig.13.

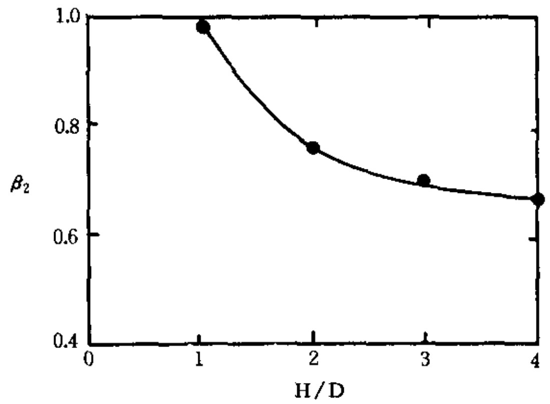


Fig. 13 Variation of $\beta_2$ versus H/D

The value of $\beta_2$ decreases from about 1 at H/D=1 to about 0.7 at H/D=4(t in s) and appears to remain practically constant thereafter. The total uplift of an anchor can thus be given as

$$\delta=\delta_e+\delta_{c(f)}=\delta_e+H\left(\alpha_1\int_0^{t_c} t^{-0.405}dt+\alpha_2\int_{t_f}^{t_c} t^{-\beta_2}dt\right) \tag{11}$$

where $\delta_e$=elastic movement and $\delta_{c(f)}$=total creep movement(Fig.7).

For the present tests, $\delta_{c(f)}/\delta_e$ varied between 0.75 to 1.25. Hence the movement caused by creep constitutes a large part of the total uplift of the anchor.

## 4. Conclusions

A number of laboratory model tests to estimate the short-term net ultimate uplift capacity of a circular anchor embedded in soft saturated clay were conducted. These tests were further supplemented by a number of creep tests in which the anchor was subjected to sustained loads which were less than the net ultimate uplift capacity. The creep tests were conducted at embedment ratios which represent shallow anchor condition. Based on these model test results, the following general conclusions can be drawn :

1. For short-term uplift, the critical embedment ratio$(H/D)_{cr}$ is approximately the same as that predicted by Das(1978).
2. In creep tests, if the net uplift load Q is less than or equal to 75% of the net ultimate uplift capacity $Q_o$, complete pullout does not occur owing to creep.
3. For $Q/Q_o \leq 75\%$, the rate of strain owing to creep may be divided into two parts-primary creep and secondary creep. The rate of strain for these two stages of creep can be expressed by Eq.s 7 and 8. It is important to realize that $\beta_1$ and $\beta_2$ may vary with the type of clayey soil.

### Notation

$\alpha_1$, $\alpha_2$, $\beta_1$, $\beta_2$ creep factors and functions of H/D

$\gamma$ saturated unit weight

$\delta$ anchor uplift movement

$\delta_e$ elastic movement

$\delta_{c(f)}$ total creep movement

$\phi$ soil friction angle

$\dot{\epsilon}$ rate of strain

A area of anchor plate

$c_u$ undrained cohesion

$F_c$ breakout factor

$F_s$ mud suction force

$Q_u$ gross short – term ultimate uplift capacity

$Q_o$ net short – term ultimate uplift capacity

t time

$t_c$ time at the end of primary creep

$t_f$ time at the end of secondary creep

$W_a$ effective self–weight of the anchor

## References


1. Adams, J.I., and Hayes, D.C.(1976), "The uplift capacity of shallow foundations", *Ontario Hydro Res, Qtrly.*, Vol.19, No.1, pp.1～13.
2. Ali, M.S.(1968), "Pullout resistance of anchor plates and anchor piles in soft bentonite clay", MS thesis, Duke Univ, Durham, NC, USA.
3. Bhatnagar, R.S.(1969), "Pullout resistance of anchors in silty clay", MS thesis, Duke Univ., Durham, NC, USA.
4. Das, B.M.(1978), "Model tests for uplift capacity of foundations in clay", *Soils and Foundations*, Vol. 18, No. 2, pp.17～24.
5. Das, B.M.(1980), "A procedure for estimation of ultimate uplift capacity of foundations in clay", *Soils and Foundations*, Vol.20, No.1, pp.77～82.
6. Das, B.M.(1990), *Earth anchors*, Elsevier, Amsterdam.
7. Kupferman, M.(1971), "The vertical holding capacity of marine anchors in clay subjected to static and cyclic loading", MS thesis, Univ. of Massachusetts, Amherst, USA.
8. Vesic, A.S.(1971), "Breakout resistance of objects embedded in ocean bottom", *J. Soil Mech. Found. Div.*, ASCE. Vol. 97, No. SM9, pp.1183～1205.


(received on Jan. 24, 1996)