THE BEARING CAPACITY OF A PILE DRIVEN INTO
SOIL AND ITS NEW MEASURING METHOD

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SYNOPSIS

This is a report of theoretical and experimental research on the settlement and the
bearing capacity of pile. A formula concerning the settlement of a friction pile is
introduced with the theory deduced from a rheological standpoint. Then applying
the formula, a new measuring method of the critical bearing capacity of pile is derived.
The bearing capacity thus derived well agrees with the results of pile tests both in
laboratory and field. This method is known to be applicable not only to the friction
pile but also to other types of pile supported in soil.

INTRODUCTION

There have been various methods to determine the critical bearing capacity of the
pile driven into soil. But, strictly speaking, the definition and the meaning of the
value thus determined are somewhat obscure. In this paper, after defining the critical
bearing capacity of the pile as the maximum load which the pile can bear permanently
without any failure, some characteristics of the settlement of a pile, the meaning and
the determining method of the critical bearing capacity and some investigations on the
pile tests are presented.

THEORETICAL CONSIDERATION ON THE MEASURING PRINCIPLE
OF THE BEARING CAPACITY OF FRICITION PILE DRIVEN
INTO CLAYEY GROUND

Creep character of friction pile: In this consideration, bearing capacity of friction
pile driven into clayey soil is assumed to be produced only by skin friction on the
surface of the pile, because the point bearing capacity of such pile is said to be negligibly
small compared with the bearing capacity due to skin friction.

In another theoretical research on the rheological property of clay, the authors derived
the following formula for the creep strain assuming the mechanical model of clay as
shown in Fig. 1.

If

\[ 0 < \gamma < \frac{\tau - \tau_0}{2B_2G_2} (2B_2 - 1) + \frac{\tau}{G_1} \]

then

\[ \gamma = \frac{\tau}{G_1} + \frac{\tau - \tau_0}{G_2} + \frac{\tau - \tau_0}{B_2G_2} \log A_b B_2G_2 t \]

(1)

where \( \gamma \) is shearing strain of clay, \( \tau \) is applied shearing stress, \( \tau_0 \) is lower yield value of

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clay (Fig. 1), $G_1$ and $G_2$ are moduli of rigidity of the mechanical model, $A_2$ and $B_2$ are rheological constants of the dashpot $\eta_2$ (Fig. 1), which has the special structural viscosity derived by the authors, and $t$ denotes time. Equation (1) is valid when the applied stress $\tau$ is less than the upper yield value which means the critical strength below which no failure can be caused by applied stress.

In the clayey ground around the pile loaded with $P$, the shearing strain $\gamma$ is given by

$$\gamma = \frac{du}{dr} \quad \text{...........................................(2)},$$

where $r$ is the radius vector from the axis of the pile (Fig. 2), and $u$ is the vertical deformation of clay.

Shearing stress at $r=r$ caused by the load $P$ is obtained as

$$\tau = -\tau_a\left(\frac{a}{r}\right) \quad \text{...........................................(3)},$$

where $\tau_a$ is the shearing stress acting in downward direction on the surface of the pile whose radius and length are $a$ and $l$, respectively.

As creep develops in the region where the applied stress is larger than the lower yield value $\tau_o$, the limiting radius of this region $b$ is given by
\[ b = a \frac{\tau_a}{\tau_0} \] ............(4).

Substituting Eqs. (2) and (3) into Eq. (1), we get
\[
\frac{du}{dr} = -\frac{\tau_a}{\gamma} a \left( \frac{\alpha a}{\gamma} - \tau_0 \right) \left( \frac{1}{G_1} + \frac{1}{B_2 G_2} \log A_2 B_2 G t \right) ............(5).
\]

Integrating Eq. (5) under the condition \( u \equiv 0 \) at \( r = b \), we obtain
\[
u = \frac{a \tau_a}{G_1} \log \frac{b}{\gamma} + \left( \frac{a \tau_a}{G_1} \log \frac{b}{\gamma} - \tau_0 (b - \gamma) \right) \left( \frac{1}{G_2} + \frac{1}{B_2 G_2} \log A_2 B_2 G_2 \right) ............(6).
\]

Besides, \( \tau_a \) is expressed as
\[
\tau_a = \frac{P}{2 \pi a d} ............(7).
\]

Substituting \( a \) instead of \( r \) in Eq. (6), and applying Eq. (7) for \( \tau_a \), the settlement of pile \( u_a \) is given as
\[
u_a = \frac{P}{2 \pi a d} \log \frac{b}{a} + \left( \frac{P}{2 \pi a d} \log \frac{b}{a} - \tau_0 (b - a) \right) \left( \frac{1}{G_1} + \frac{1}{B_2 G_2} \log A_2 B_2 G_2 \right) ............(8).
\]

According to Eq. (8), the settlement of the friction pile is proportional to the logarithm of time, if the applied load is constant.

**Fundamental theory of the load-controlled test of friction pile**: In a load-controlled test, as the load is increased stepwise by equal load increments at uniform time intervals, the stress in the clayey ground around the pile increases according to Eq. (3). In this case, final strain \( \gamma_n \) developed at the end of the \( n \)th loading stage can be represented* by the following equation unless the stress exerted in the ground exceeds the upper yield value:
\[
\gamma_n = n \tau_i \left( \frac{1}{G_1} + \frac{1}{G_2} \right) - \frac{\tau_i}{G_2} \left( -l + (-l)^2 + (-l)^3 + \cdots + (-l)^n \right) ............(9),
\]

where \( \tau_i \) is the stress exerted by each load increment, \( \tau_i \) denotes a loading interval of each stage and
\[
l = \frac{1}{B_2} \log A_2 B_2 G_2 \tau_i ............(10).
\]

The relation between \( \gamma_n \) and applied stress \( \tau \) (= \( n \tau_i \)) expressed by Eq. (9), is proved** to be a straight line on a logarithmic paper for various values of \( l \ (-1 < l < 0) \).

Since the linearity of \( \log \tau \sim \log \gamma_n \) curve holds only for the stress below upper yield value, the stress corresponding to the bent point from which the straight line deviates should be equal to the upper yield value.

Substituting Eqs. (2) and (3) into Eq. (9) and assuming \( u \equiv 0 \) at \( r = b \), the settlement of the pile at the end of the \( n \)th loading stage is obtained as
\[
u_{an} = \frac{nP_i}{2 \pi a d} \log \frac{b}{a} - \frac{P_i}{2 \pi a d} \frac{1}{G_2} \log \frac{b}{a} \left( -1 + (-l)^2 + (-l)^3 + \cdots + (-l)^n \right) ............(11),
\]

* Details of this reduction will be reported in another paper.
where \( P_t \) is the load increment at each loading stage. Hence the total load \( P \) at the \( n \)th loading stage is given by

\[
P = nP_t \tag{12}.
\]

For the same reason as stated above, the critical bearing capacity of the friction pile defined by making the maximum shearing stress in the ground equal to the upper yield value can be measured as the load corresponding to the first bent of the load—settlement curve obtained by such a load-controlled pile test, when the settlement of pile \( u_{\text{cm}} \) and the total load when \( P \) is plotted in logarithmic scale.

**MODEL TESTS**

*Model piles and clay samples:* Model piles were of cylindrical form having a length of 50 cm. and various diameters of 1, 2, 3 and 4 cm., and were attached with conical tips with angle of 30 deg. Each model pile made of brass was penetrated by small drop hammer into clay sample in each vessel. Disturbed clay samples obtained from Osaka alluvial stratum were placed 70 cm. thick by compaction with a small rammer into 5 cylindrical steel vessels 55 cm. in diameter and 85 cm. deep. The results of the physical tests of the clay are as follows: specific gravity is 2.65, L.L. is 54.0 percent, P.L. is 29.3 percent, cohesive strength measured by a small vane tester is 0.07 kg./cm². This cohesive strength was the value obtained under the equilibrium state after about 3 months of curing in the vessel (Fig. 3).

![Fig. 3.](image)

*Creep settlement of model pile:* Figure 4 shows the relations between the settlement of pile and time which were obtained under loadings of various constant loads on the pile of 4 cm. in diameter for 100 min. It is worth noticing, in these curves, that i) the settlement \( u_a \) increases proportionally to the logarithm of time \( t \), if the load is smaller than 5 kg., ii) the slopes of these lines increase with the increase in applied load, iii) if the load is larger than 5 kg., the curve rises concave upwards and suggests occurrence of failure in future. The features indicated in (i) and (ii) can be interpreted by Eq. (8), and more obviously represented in Fig. 5, which is obtained by plotting \( du_a/d (\log t) \) as ordinate and applied load as abscissa. The linear part of the line shown in Fig. 5 has two critical points. One is the intersecting point of the line with abscissa and this point (=0.2 kg.) gives the lower critical value of the load below which no creep settlement can develop. The other one is the bent point of the line, and the ab-
scissa of this point (=5.0 kg.) gives the upper yield value of the load or the critical bearing capacity of the pile below which no failure settlement is expected permanently.

**Load-controlled tests:** As a new method to determine the critical bearing capacity of friction pile, load-controlled tests were performed on the model piles. On this test, the load was applied by equal load increment at uniform time intervals, and settlement was measured at each end of the loading. The results obtained are plotted on a logarithmic paper as shown in Fig. 6, wherein the critical bearing capacity of the pile can be measured as the load corresponding to the bent point of each curve. It is worth noticing that, as shown in Fig. 6, the critical bearing capacity measured is independent of the
rate of load increment for each pile and is just equal to the upper yield value obtained as the bent point in Fig. 5. This is evident because the load value corresponding to the bent point of the settlement rate-load curve (Fig. 5 is an example for the pile of 4 cm. diameter) coincides very well with the critical bearing capacity obtained by the curve shown in Fig. 6 for each diameter of model pile.

*Relation between critical bearing capacity of pile and number of blow of drop hammer:* Each pile is penetrated as deep as 40 cm. into the clay by dropping a weight of 0.4 kg. from the height of 20 cm. above the top of the pile.

If we denote the number of blow of the drop hammer necessary to penetrate final 1 cm. as $1/s$, the relation between $1/s$ and the critical bearing capacity $P_c$ was obtained experimentally as shown in Fig. 7. As this figure seems to be a straight line, the equation of this line can be represented as

$$P_c = c \frac{1}{s}$$

$c$: a constant).

If we equate $P_c \cdot s$ or the work done by the penetrating resistance of a pile for a single blow to the effective energy given by the weight $W$ ($W=0.4$ kg.) dropped from the height $H$ ($H=20$ cm.), the above equation may be expressed as

$$P_c \cdot s = \frac{1}{30} W \cdot H.$$

**FIELD TESTS**

*Test piles and soil profile:* Soil profile of the ground where the pile tests were performed is shown in Fig. 8. Piles tested were cylindrical reinforced concrete piles of 35 cm. in diameter. The piles were driven more than 3 m. apart with a double acting steam hammer. As the piles were driven 7.5 m. and 9.0 m. into the ground through the sandy layer, the point bearing resistance arose in the soft clay which is an alluvial clay layer normally consolidated and has the unconfined compressive strength of 0.6 kg./cm². In this case, however, skin friction developed not only in the clayey layer but also in the sandy layer.
Tests on the pile of 9 m. in depth: Load-controlled tests in which load was added by 10 tons at every 100 minute stepwise were performed twice on the pile. The first time (say No. 1) was one day after the pile was driven, and the second time (No. 2) two weeks later. From the measurements of the creep settlement at every loading stage, the relation between the load and the rate of settlement caused by the load are obtained as shown in Fig. 9, from which critical bearing capacities are measured to be 38 tons for test No. 1 and 47 tons for No. 2. Figure 10 shows the load-settlement curves plotted on a semi-logarithmic paper in which settlements are measured at the end of every loading stage. From these figures, loads corresponding to the bent points or the critical bearing capacities are measured to be 38 tons for No. 1 and 47 tons for No. 2. These values satisfactorily agree with those obtained by Fig. 9.

From these tests, the following may be suggested: (1) The determining method of the critical bearing capacity of the pile by the load-settlement curves of load-control-
led test is applicable not only for clayey ground but also for sandy ground*, and (2) the bearing capacity of the pile tested here increases with the lapse of time.

Tests on the pile of 7.5 m. in depth: In order to investigate the time effect on the critical bearing capacity of the pile, the load-controlled test with the same loading rate as stated in the preceding paragraphs were performed on the pile of 7.5 m. in depth to measure the critical bearing capacities at 1, 4, 17 and 35 days after the pile was driven. The settlement-load curves at different time obtained by the tests are plotted in Fig. 11 and respective critical bearing capacities are measured as loads corresponding to their bent points. Plotting these values against elapsed time, the tendency of the increase of the critical bearing capacity with the lapse of time can be observed as shown in Fig. 12.

* This feature was also recognized by authors' other experiment.
Tests on the pile whose tip rests in a firm soil layer: The principle of the above stated method to determine the critical bearing capacity by means of the bent point of load-settlement curve is based on the character that the shearing deformation increases in proportion to the logarithm of time particularly in the case when the exerting stress in the soil is less than the upper yield value and the settlement is caused only by the shearing deformation. Therefore, this method is valid not only for the settlement due to skin stress around the pile but also for that due to the bearing stress under the tip of pile.

From this standpoint, the critical bearing capacity of a pile which is supported by skin friction and point bearing was tested.

The soil profile where the pile is driven consisted of a loose sandy layer 7 m. deep from the ground surface, a soft clay layer 20.5 m. thick, and a well compressed silty sand layer laid below the clay layer. A reinforced concrete pile of 50 cm. in diameter was driven to the depth of 30 m. from the ground surface. As the pile rested in the firm silty sand layer, respectable bearing resistance was expected under the tip of pile.

The critical bearing capacity of this pile was measured by two methods. Measured value according to Bullen's method\(^1\) was 313 tons which was the value of the load corresponding to the settlement of 10 percent of the diameter of the pile. On the other hand, the measured value by the authors' method was 310 tons, which is the load corresponding to the bent point of the curve shown in Fig. 13.

CONCLUSION

In this paper, the behaviour of creep settlement of a friction pile and the measuring method of critical bearing capacity are studied theoretically. To substantiate the
Theoretical consideration, some laboratory and field tests were performed.

The summary of the research is as follows: (1) The settlement of the friction pile can be determined by the theory which was obtained by the authors for the shearing creep of clay. (2) The creep settlement of the friction pile due to a constant load is directly proportional to the logarithm of the elapsed time, so far as the load is less than the critical bearing capacity. (3) The ratio of the settlement to the logarithm of time increases in proportion to the load applied. (4) If the load exceeds the critical bearing capacity, the settlement increases more than the ratio expected by (3) and this tendency suggests the detrimental settlement in future. (5) The critical bearing capacity of the friction pile can be determined by the load corresponding to the first bent point of the load-settlement curve plotted on the logarithmic paper. In this case, the curve is obtained by the load-controlled test whose load is added in equal increments at uniform time intervals. (6) The critical bearing capacity obtained by this method is independent of the rate of load increase. (7) From the test results on model piles, it is found that the product of the critical bearing capacity and the final penetrating length of the pile by one blow of the drop hammer is directly proportional to the potential energy of the drop hammer measured from the top of the pile. (8) The critical bearing capacity of the pile driven in the ground was observed to increase in accordance with the time elapsed. (9) The method to determine the critical bearing capacity as stated in (5) may be applicable not only to the friction pile driven into clayey ground but also to that into sandy ground. (10) The method stated in (5) may be applicable not only for the case of the friction pile but also for the method to determine the critical bearing capacity of other types of foundations such as a point bearing pile or a footing foundation, so far as the settlement of the foundation is caused by the shearing creep deformation in the soil under the foundations.

REFERENCES


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