FIELD TESTS WITH REGARD TO PILE DRIVING AND BEARING CAPACITY OF PILED FOUNDATIONS

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SYNOPSIS

The paper consists of two parts. In Part I the effect of pile driving on the surrounding soil is discussed on the basis of observation of the pore water pressure in the soil, and the earth pressure and pore water pressure on the pile surfaces. Part II describes the result of load tests on a single pile, group pile, and spread footing foundations. The piles were steel piles 30 cm in diameter and 5.55 m in embedded length. With the group pile foundation the piles were placed in a square array spaced 90 cm center to center. Three of the group piles and the single pile were instrumented. In order to investigate group action, the measurement was made on the axial loads in the piles, earth pressure on the pile surface, the base reaction at the bottom of pile cap, and the movements of the subsoil. The bearing capacity of the group pile foundation is compared with the single pile and spread footing foundations.

INTRODUCTION

In design of pile foundations driven into soft clay, there are some problems for which no adequate solutions have been found in common engineering practice.

When piles are driven into saturated clay, it is commonly observed that the clay around the pile is remolded and the skin friction decreases temporarily during the process of pile driving. However, the clay would eventually be consolidated under the overburden pressure and the compressive stresses produced by the penetration of the pile, and the strength of the clay may be largely regained. The final strength, therefore, depends on the magnitude of the pressures developed in the clay surrounding the pile. This problem has been studied previously by H. B. Seed¹⁾ for single piles. However, the effect of pile driving on the surrounding soil obviously depends on the soil conditions and the pile arrangement. We need, therefore, to have more experience with various conditions.

When load is applied on a group of piles having a rigid cap, it has been generally recognized that the piles behave differently from the individual pile, since the piles in a group are seldom placed so far apart that they do not influence each other. A group

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of piles in a clay soil usually give less desirable performance than an equal number of individual piles.

One of the most outstanding approaches to the group action of piles has been conducted by T. Whitaker²⁾. His experiments with model piles have shed some light on this problem by showing that for a wide range of loading the corner piles in a group take the largest and the center the smallest proportion of load, and that there is a critical value of pile spacing at which the mode of failure changes from the block failure to the local failure.

In model tests, the effect of pile driving on the soil properties cannot be properly evaluated, and also, there is a limitation to instrumentation both on piles and in the soil in order to investigate precisely the behaviour of the group action. No field loading tests on the group pile foundation have been published so far, in such a manner that the group action is reliably determined.

A research group, headed by Prof. H. Nagai, conducted a comprehensive experimental work on the pile foundation, performing a field test on a nearly full-sized group pile foundation in a fairly homogenous soft clay, as well as on a single pile foundation and spread footing foundations.

The testing program covered vibration tests, lateral load tests and vertical load tests. The piles were driven in July 1961, then series of the vibration tests and lateral load tests were carried out on the test foundations until January, 1962, in which maximum deflections at the pile caps were limited to less than 3 mm. From March, 1962, the vertical load was applied on the foundation until failure occurred.

The results of all the tests have been reported by the research group.³⁾ This paper describes the test data concerning the effect of pile driving on the clay, and the result of the vertical load tests.

TEST SITE SUBSOIL CONDITIONS

The primary consideration for selecting the test site was that the soil profile should have a fairly uniform clay stratum of sufficient thickness to facilitate data analysis. After a careful study of several locations, the test site was selected at 1-chome, Otemachi, Tokyo.

Three borings and six Dutch cone penetration tests were made within the test area. The subsoil stratigraphy, together with a typical result of field and laboratory tests on 75 mm piston samples from one of the bore holes, is shown in Fig. 1.

The soil profile at this site consists of about 1.5 m of sandy silt with gravel, and a layer of slightly organic silty clay, containing shells, which extends to a depth of about 13.5 m. Below this depth gravel and dense sand layers extend to bedrock.

Before the test foundations were constructed, the top soil was excavated down to a depth of 1.5 m, so that the foundations would be supported by the relatively homogeneous clay. As the ground water table was about 50 cm higher than the foundation level, a drainage pit was dug in a corner of the excavated area.

The clay is highly plastic, and the plasticity index tends to increase with depth. The natural water content varies between 80% and 130%, and it nearly equals the liquid

Fig. 1 Boring log and soil properties

limit in the upper half of the stratum.

According to the observation of piezometers, installed prior to pile driving, the hydrostatic pore water pressure distribution exists to a depth of about 8 m, but at greater depths the pore water is smaller than the hydrostatic pressure, approaching zero near the gravel layer. This distribution of the pore water pressure has been caused by pumping of ground water in the underlying sand layers, and resulted the increase in the effective stress of the lower part of the clay stratum.

The shear strength of the clay increases with depth, about 0.25 kg/cm² at the foundation level and about 0.4 kg/cm² at the proposed pile tip. The sensitivity is so high that remolded samples for the unconfined compression test failed under their own weights.

The consolidation tests showed that the clay was slightly overconsolidated.

DESCRIPTION OF TEST PILES AND INSTRUMENTATION

All the test piles were of steel pipes with closed ends, 30 cm in diameter, 0.32 cm in

Fig. 2 Arrangement of test piles

Fig. 3 Test piles

wall thickness and 5.55 m in embedded length. The arrangement of the piles is shown in Fig. 2. For the group pile foundation nine piles were arranged in a square array, 90 cm (three times the pile diameter) center to center.

The instrumented piles were piles Nos. 1 to 3 of the group pile foundation which were to be placed at side, center and corner, respectively, and the isolated pile No. 4. They were split longitudinally, instrumented on the inside, then reassembled to form watertight joints (Fig. 3).

On piles Nos. 1 and 2 were installed earth pressure cells, pore water pressure cells and wire strain gages. Only wire strain gages were mounted on piles Nos. 3 and 4.

The earth pressure cells for measuring the soil pressure acting on the pile surfaces during driving and load testing must operate for as small a deflection as possible in order

Fig. 4 Arrangement of gages on piles

to minimize the effect of soil arching. From this point of view the magnet-type pressure cell, with a diaphragm 3 cm in diameter was selected and mounted on the pile surface with metal-to-metal contact. The measuring range of the cell can be changed from 1 kg/cm² to 3 kg/cm², and the maximum deflection is 0.001 mm, with an accuracy of 1% for each range. The pore water pressure cells are of the same type as the earth pressure cells, except for the porous disk outside the diaphragm.

The earth pressure cells were installed at 8 elevations on both sides of the piles, and the pore water pressure cells were located near the earth pressure cells as shown in Fig. 4. Wire strain gages with polyesel base were mounted on the inside diametrically opposite to eliminate the effect of away from the longitudinal joints, at 60 cm intervals on piles Nos. 1 and 2, and at 30 cm intervals on piles Nos. 3 and 4.

PART I PILE DRIVING AND ITS EFFECT ON THE SURROUNDING SOIL

In order to safeguard the gages on the instrumented piles, all the piles were driven statically using a winch system. At a rate of driving of approximately 10 cm/min, the pile driving was completed in five days, following the sequence shown in Fig. 2.

Before driving the piles, the piezometers had been installed at the locations shown in Fig. 2. The piezometers consisted of 5 cm steel pipe with pore water pressure cells

Fig. 5 Distribution of total normal stress (effective stress plus pore water pressure) and pore water pressure induced during pile driving

of the wire strain gage type mounted at depths of 1 m, 3 m, 5 m, 8 m and 12 m from the foundation level. Piezometer records prior to pile driving showed essentially a hydrostatic pore water pressure distribution, except at the lowest one, as stated before.

These piezometers, together with the earth pressure cells and pore water pressure cells on the surface of piles Nos. 1 and 2 were employed to study the effect of pile driving on remolding and reconsolidation of the surrounding soil.

Measurements were made at a partial pile penetration (3 m) and full penetration (5.55 m) for each pile, and they were continued for the whole test period of about 9 months.

Fig. 5 gives the total normal stresses (the effective stress plus pore water pressure) and the pore water pressures at the surface of the instrumented piles Nos. 1 and 2 during their penetration. The data show that both the total normal stress and the pore water pressure tend to increase with depth. It is noticeable that the total normal stress on the pile surface is nearly equal to the pore water pressure, i.e., the effective stress normal to the pile surface vanishes near the pile during and immediately after driving.

Fig. 6. Distribution of total and pore water pressure acting on the surface of piles Nos. 1 and 2

If one assumes that the maximum principal stress is perpendicular to the pile surface and that dilatancy of the clay is negligible, zero effective stress on the pile surface indicates that the shearing stresses do not exist and the clay become liquefied during the pile driving. It is conceivable that, in the clay at the test site whose natural water content equals the liquid limit, the shearing strength may nearly vanish due to re-molding.

Fig. 6 shows the distribution of the pore water pressure and the total normal stress produced at the surface of piles Nos. 1 and 2 while a certain pile is being driven. In Fig. 7 the pore water pressure induced at given depths immediately after full penetration of a pile are plotted against the horizontal distance from the pile being driven. The pore water pressure is very small from the ground surface down to a depth of 1 meter, but at greater depths it tends to increase with depth, reaching peak values at elevations somewhat shallower than the tip of pile that is being driven. At depths below the pile tip in some distances from the pile, even negative pore water pressures

Fig. 7 Distribution of induced pore water pressure

Fig. 8 Changes with time of total and pore water pressure acting on pile No. 2

are recorded. The excess pore water pressure decreases sharply as the distance increases, nearly vanishing at about 2 m from the pile.

The clay adjacent to the pile surface is completely remolded due to pile driving, but the clay at some distance from the pile is still in a state of elastic equilibrium and therefore, exerts residual compressive stress in addition to the earth pressure at rest due to the overburden pressure. The total compressive stress will be released with the progress of consolidation of the clay. Fig. 8 shows how the total stress and the pore water pressure at pile No. 2, as an example, changes with time. The data, together with other piezometer records show that the excess pore water pressure vanished in

Fig. 9 Distribution of total stress and pore water pressure acting on the surface of pile Nos. 1 and 2, immediately and three weeks after driving

two to three weeks after the last pile was driven, indicating the completion of the consolidation process.

In Fig. 9 are plotted against depth the pore water pressure and the total normal stress acting on the pile surface measured three weeks after pile driving, when the pore water pressure has evidently reached the hydrostatic pressure. The data taken immediately following pile driving are superposed on Fig. 9 for comparison. It should be noticed that the normal stress after consolidation exceeds by about 0.2 to 0.3 kg/cm^2 the overburden pressure shown in dotted line, which probably is due to the residual compression stress produced by pile penetration. These values showed little change for nine months during the test period.

PART II LOAD TESTS ON SINGLE-PILE, GROUP PILE AND SPREAD FOOTING FOUNDATIONS

TEST FOUNDATIONS

Concrete pile caps were placed over the single pile and the group piles* about one month after pile driving. For the purpose of comparison, two spread foundations were

* In the multi-pile foundation the dead weight was doubled by placing concrete over the pile cap about two months after the first concrete placing, for the necessity of a vibration test in the program.

also constructed for the testing program, the dimensions of which were identical with those of the pile caps. The vertical load test was conducted* only on the following three foundations:

- a single pile foundation
- a group pile foundation
- a spread footing foundation

The layout of the test foundations, as well as their dimensions are shown in Fig. 10.

Fig. 11 Arrangement of measuring devices for group pile foundation

* The vertical load test was carried out after the vibration test and lateral load test which had been performed for about 6 months. Therefore, the vertical load test reported here, started about 7 months after the construction of the test foundations for a test period of about a month.

The instrumented piles were extended up to the surface of the pile caps to provide outlet for the cables of the gages, and to enable inspection of the inside of the piles and circulation of warm dry air through the pile to protect the wire strain gages during the test period.

To measure the soil reaction at the bottom of the pile caps, three earth pressure cells for the group pile foundation and one for the single pile foundation were mounted as shown in Fig. 11. These pressure cells were of the wire strain gage type with a diaphragm 5 cm in diameter and the measuring range of 0–1 kg/cm².

To facilitate observation of the ground surface movement, the testing area was back-filled with sand to a depth of 50 cm, the surface being covered with thin lean cement mortar. It has been confirmed by measurements of the earth pressure cells at the bottom of the pile caps that the weight of the sand mat exerted negligible influence on the consolidation of the caly stratum.

TEST PROCEDURE AND MEASUREMENTS

Each of the test foundations was loaded by means of a hydraulic jack against a beam which was supported by pile rows placed at a distance of 8 m. Load was applied principally in increments, repeating loading and unloading to increasing loads until failure occurred. To allow measurements a pause of sufficient duration was made after loading and unloading. The test procedure for each of the foundations was as follows :

The single pile foundation: For the single pile foundation, measurements were made on the settlement of the pile cap, movement of the ground surface around the foundation, loads on the pile shaft and the soil reaction beneath the cap.

For the measurement of the ground surface movement, short piles were placed as bench marks at distances of 55 cm, 105 cm, 155 cm from the center of the pile, and the vertical and horizontal displacements of these bench marks were measured by dial gages accurate to 0.001 cm. The settlement of the pile cap was measured at four corners by the same type dial gages. The gages were attached to an independently supported rigid frame. The load was observed by a proving ring of 30 ton-capacity.

Loading and unloading were made at a speed of 3 t/min in increments up to 24 t, pausing 20 min both at loading and unloading, except 15 min for the last load. When load was raised to 24 t, the settlement proceeded too rapidly to measure its amount for more than 15 min.

Settlement readings were taken at every 2 t during loading and unloading, and also at elapsed times of 0, 1, 3, 5, 10, 20 min after loading and unloading. The axial strains of the pile and soil reaction beneath the pile cap were measured immediately and 15 min after loading and unloading. Some errors might have been introduced since the reading of all the gages took about 2 to 3 min.

After failure was observed the further penetration of the foundation was caused by jacking with a constant penetration rate of 3 mm/min and observations were made at adequate intervals.

The group pile foundation: For the group pile foundation, measurements were made

of the settlement of the pile cap, the vertical and horizontal movements of the subsoil and the ground surface, the shaft loads of piles Nos. 1 to 3, the earth pressures and pore water pressures on the surfaces of piles Nos. 1 and 2; and the soil reaction at the bottom of the pile cap. The layout of the measuring devices is shown in Fig. 11.

For measurement of the vertical displacement of the subsoil, three reference points were installed at depths of 2.5 m, 5 m, and 7.5 m from the foundation level within a perimeter of pile group, and the other three points at depths of 5 m and 7.5 m outside the pile group. The reference point consisted of an inner rod with a conical point having a sleeve 20 cm in length at the lower end and a casing of 5 cm in outer diameter to eliminate the friction through the stratum. To install the device, the rod, together with the casing, was driven into a desired depth, then the casing was withdrawn about 50 cm. The dial gage reading at the top of the inner rod would indicate the vertical displacement of the soil at a depth of the reference point in position.

The horizontal displacement of the subsoil outside the pile group was determined by measuring inclination of flexible plastic pipes placed at three locations indicated in Fig. 11 by means of the inclinometer. To afford a sufficient flexibility these pipes, 5 cm in inner diameter, consisted of 50 cm units with connections of tire tubes, a thin brass rod running through the inside to guide the inclinometer. The construction of the inclinometer is similar to the design by the Swedish Geotechnical Institute.⁴⁾

The settlement of the pile cap and the ground surface movements were measured in a similar manner as in the single pile foundation. The applied load was read by a hydraulic pressure gage attached to the jack.

Loading and unloading were made in increments to 30 t, 60 t, 90 t and 143 t, resting 45 min both at loading and unloading for measurements. Since a noticeable settlement occurred at 143 t, the 120 t cycle was repeated. An examination of the test data showed that the strain gages had been read with inadequate magnification; for this reason, the tests for the 60 t cycle were repeated twice, then the program proceeded to the 143 t, 170 t and 173 t cycle in the same way as before.

Settlement of the pile cap was recorded at every 20 t during loading and unloading and at elapsed times of 1, 3, 5, 10, 20, 45, min after loading and unloading. The measurements of soil movements, pile loads, pressures on piles and base reactions were made immediately and 35 min after loading and unloading. It took about five minutes to complete the readings of all the gages. Although evident failure had appeared already under the previous loading, the further loading was given after a three-day intermission until the settlement reached about 50 cm, in which the loading procedure was not the same as before, but measurements were continued at frequent intervals.

The spread footing foundation: For the spread footing foundation, loading and unloading were repeated with a 1-ton increment up to 18 t. A pause of 10 min was given both at loading and unloading, and the settlement of the foundation was measured at the four corners. Readings were taken at every ton during loading and unloading, and at elapsed times of 1, 3, 5, 10 min. After release of the 18-ton load, the load was increased again at a constant penetration rate of about 1 mm/min until

failure was reached.

LOAD-SETTLEMENT CURVES AND SOIL REACTION AT BOTTOM OF THE PILE CAPS

The load-settlement curves for the three test foundations are shown in Figs. 12, 13, and 14.

The ultimate bearing capacity of the spread foundation is evaluated from Fig. 12 as 20 t in total or 24.7 t/m^2 (27.4 t/m^2 including its own weight). The average shearing strength of the clay within the failure zone is estimated at 0.37 kg/cm^2 , assuming $N_c = 1.3 \times 5.7 = 7.4^4$. Comparing this value with the soil test data, some consolidation effect on the soil beneath the footing would be appreciated.

The settlement of the single pile foundation is very small for loads under 21 t, but it tends to increase noticeably at the 21 t-load, then a well-defined failure occurs at 24 t (Fig. 13).

From measurements of soil reaction at the bottom of the pile cap as shown in Fig. 16 (a), it is evident that an increase of 3 t over 21 t was carried mostly by the base of the

Fig. 12 Load-settlement curve for the spread foundation

Fig. 13 Load-settlement curve for the single pile foundation

pile cap. This will also be recognized from measurement of the pile load as will be stated later.

The settlement of the group pile foundation is considerably larger than that of the single pile at the same average pile load, within a customary working load. Curves linking points at each loading in Figs. 13 and 14, with some modification for the repeating cycles in Fig. 14, are shown together in Fig. 15 for comparison, in which the load scale for the group pile foundation is taken 9 times that for the single pile foundation.

At a load of 120 t, an indication of increase in settlement first appears, and when the load is increased to 143 t, a considerable amount of settlement occurs and a noticeable increase in the soil reaction at the bottom of the pile cap is associated with it as shown in Fig. 16 (b). As the load increases further, the settlement proceeds approximately along a straight line with a steeper slope and also the base reaction increases gradually. Thus, there is no sharp critical load as can be seen in the test results of the single pile foundation. This indicates that failure in the pile group takes place progressively from pile to pile.

Fig. 17 shows how the load applied on the pile foundations can be distributed between the pile or piles and the pile cap on the basis of measurements of the soil pressures

Fig. 14 Load-settlement curve for the group pile foundation

at the bottom of the pile caps. It is only a crude estimate because, although no significant tilting of the foundations took place until failure approached, the number of the measuring points at the base was limited. However, it could be said that most of the load is carried by the piles for loads below the critical load, the base reaction being negligibly small, and the ultimate bearing values at a considerable penetration of the foundations are only $0.5\text{--}0.6\text{ kg/cm}^2$ which are only about 20% of that of the spread foundation without pile. Relationships between the soil reaction and the foundation settlement are shown by dotted lines in Fig. 15. The low bearing capacity of the pile cap may result from the following: remolding of the clay during pile driving, less consolidation effect associated with the fact that only a small portion of the weight of the concrete cap is transmitted to the base, and overlapping of additional stress caused by the load of the pile shaft.

SHEARING RESISTANCE OF THE SOIL AROUND THE SINGLE PILE

Load distributions along the pile shaft of the single pile foundation at loading and unloading are shown in Fig. 18, and Fig. 19(a) shows in proportion to the maximum shearing strength how the shearing stress along the shaft develops as the load in-

Fig. 15 Comparison of load-settlement curve between the single and group pile foundations

creases. It is evident that the skin friction reaches its maximum along the shaft except the uppermost part at a load of 21 t and there is no more appreciable increase in the shaft load when the load is increased from 21 t to 24 t. The maximum shearing resistance acting on the pile surface were calculated assuming that failure occurred at the interface between the soil and the pile. They are plotted in Fig. 19(b), in which the shear strengths obtained from the field cone tests and laboratory tests conducted before pile driving are superposed for comparison. According to our experiences with pulling-out tests of 30 cm steel piles at different places of similar soil conditions, the actual slide occurred about 3 cm away from the pile surface. If it is so in this case, the values in Fig. 19 should be reduced by about 10%. Although some discrepancy exists between the test results for the undisturbed soil, the calculated shearing resistances fall between them, except for the highest and lowest parts of the pile.

Fig. 16 Soil reaction at the bottom of the pile cap
 (a) For the single pile foundation
 (b) For the group pile foundation

Fig. 17 Loads carried by pile and pile cap
 (a) For the single pile foundation
 (b) For the group pile foundation

Fig. 18 Distribution of the axial load in the single pile

GROUP ACTION AND PROGRESSIVE FAILURE IN THE GROUP PILE FOUNDATION

The group action or the progressive failure of the group pile foundation will be more vividly pictured by investigating the results of measurements of soil movement, soil pressures at the pile surfaces and load carrying proportion among piles in various stages of loading.

The vertical displacements of the soil at various depths are plotted against the settlement of the foundation in Fig. 20.

Differences between the readings at adjacent reference points along depth would indicate the amount of the contraction of the soil between them. Proportion of the contraction of the soil below the foundation area is shown for various loading stages in Fig. 21. The horizontal displacement measured at 30 cm from a perimeter of the pile group, as an example, is also shown in this figure.

The horizontal movement of soil around piles would produce the pressure on the surface of the piles. The pressure distributions along the pile shaft of piles Nos. 1 and 2 are shown in Fig. 22.

Before showing the results of load distribution along the piles, it will be necessary to add some comments on their measurements. For the first test cycle up to a load of 143 t, the measurement on the wire strain gages was made with inadequate amplification. After finding this, the test was repeated in a normal measurement system and proceeded to the further loading stage, but during this test cycle it was evident that the foundation had been subjected to a loading history for loads lower than the 143 t load. Moreover, since some zero drift was observed on the gages during the test period, the data analysis was made on the basis of the increment or decrement in the strain during each loading or unloading.

Fig. 19 Comparison of the shearing resistance with soil test data

Fig. 23 is for the subsequent test cycle, and shows the distribution of the shaft loads which are represented as the increment during the loading plus the decrement during the previous unloading, and the distribution of the decrements in the shaft loads during each unloading.

The movement of the reference point at a depth of 7.5 m within the foundation area is only less than 2 mm, while the settlement of the foundation becomes 200 mm under the load of about 200 t as seen in Fig. 20. It is evident, therefore, that most of the soil movement is limited in depths shallower than 7.5 m and the deeper stratum has little influence on the behavior of the foundation under loading. The behavior of the soil above that level changes its feature with load.

The 60-t load At a load of 60 t, more than 60% of the total settlement of 3 mm is attributed to the constriction of soils at or immediately below the pile tips, from 5 m to 7.5 m (Fig. 21). Therefore, the soils within the perimeter of the pile group are not so strongly stressed and no significant earth pressure acts on pile surfaces. As the soil

Fig. 20 Vertical displacements in subsoil and the ground surface

within the pile group moves nearly en masse with the piles, most of the friction is mobilized along the perimeter of the pile and a considerable amount of load is transmitted to the pile tips. The applied load is so divided among piles that the corner piles are mostly stressed, the side piles next and the center pile least as previous investigators have already pointed out.

The 120-t load At a load of 120 t, there are some indications that failure is incipient at the pile tips: the reference points Nos. 4 and 5 at depths 5 m and 7.5 m, respectively, placed outside the pile group show a tendency of upward movement, but no such tendency at No. 6. The stressed zone inside the pile group extends to the upper part, and an increase in the earth pressure at pile surfaces appears to some extent both at

Fig. 21 Distribution of compression zone in the ground

the pile tips and immediately below the bottom of the pile cap.

The 143-t load It has been previously suggested that a fairly distinct failure has occurred at a load of 143 t and a sudden increase in the base reaction has taken place. It will be seen from Fig. 21 that the stressed zone moves upwards. The skin friction at the corner pile is supposed to reach the ultimate shearing resistance, except for a length near the bottom of the pile cap. The earth pressures produced at the pile surface increases greatly, except for the part close to the bottom of the pile cap. The phenomena which appear near the bottom of the pile cap in the measurements of the shaft load and the earth pressure show that the movement of the soil beneath the bottom is still restrained by the friction between soil and concrete. With regard to the pile No. 1, the earth pressure produced is larger on the inside face, while the pressure at the center pile is nearly symmetric. This indicates a tendency of the outward soil movement from the center.

The upward soil movement is observed even at the reference point at a depth of 7.5 m placed as far as 2.85 m from the edge of the pile group.

Summarizing these observations the authors consider that at this load distinct failure occurred around the pile group in a large scale.

Fig. 22 Distribution of the earth pressured acting on the surface of pile Nos. 1 and 2

Fig. 23 Distribution of the shaft loads of pile Nos. 1 to 3

The 160-t and greater load As load increases over 143 t, the phenomena observed under the previous load become more pronounced. Although the contribution of the side and center piles increases in some extent as load increases, the severe inequality still exists in loading distribution among piles. The shearing resistance along the shaft of pile No. 3 is about 0.4 kg/cm^2 on the average, except for the uppermost part, which is nearly equal to the value obtained with the single pile or the original shear strength of the clay.

CONCLUSIONS

The following conclusions may be made concerning pile driving and the behavior of the pile foundation :

(1) When a pile is driven in a sensitive clay the clay around the pile liquefies during driving. The range of influence was about 2 m (about 6 times the pile diameter) and the excess pore water pressure in the clay dissipated in about 3 weeks after pile driving. After the completion of consolidation the effective stress perpendicular to the pile surface was larger by 0.2 to 0.3 kg/cm^2 than the stress at rest. This would be caused by displacement of soil due to pile penetration.

(2) The single pile foundation had the well-defined failure load, at which the skin friction almost simultaneously reached the shearing strength along the pile shaft. With the group pile foundation the load was carried mostly by the corner pile, least by the center pile and failure proceeded progressively. The skin friction of both the single pile and the corner pile in the pile group at failure was approximately equal to the strength of the original soil. The average bearing capacity per pile of the group pile foundation was about 75–80% of the bearing capacity of the single pile foundation.

(3) The load applied on the pile foundation was mostly carried by pile or piles until failure occurred and the bearing capacity of the base of the pile caps was only about 20% of that for the spread foundation.

Since the characteristics mentioned above obviously depend on the soil condition, pile spacing and size of pile, the authors consider that more experience with various test conditions will be necessary before the results can be applied to practical problems.

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