NEGATIVE SKIN FRICTION ACTING ON STEEL PIPE PILE IN CLAY

LE FROTTEMENT NEGATIF SUR PIEUX-TUYAUX EN ACIER DANS L'ARGILE

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SYNOPSIS Measurement of skin friction on four kinds of steel pipe piles was carried out for more than two years at a thick alluvial stratum where consolidation of 8 cm per year had been observed due to a decrease in pore water pressure at an underlying sand stratum. Several factors which influence the negative skin friction, i.e., elapsed time, position of the neutral point, magnitude of skin friction, and load carried to the bottom end of the pile are discussed for the following cases: friction and point bearing piles, open point and closed point pipe piles, and vertical and battered piles. A method of estimating the magnitude of negative skin friction is discussed and proposed.

INTRODUCTION

Judging from the fact that it is recently often demanded to use long piles in soft soils where settlement is supposed to occur, the negative skin friction due to settlement of the surrounding soil has to be taken into account in the design of piles. Several measurements and proposals have been made, but neither the mechanism nor the method for estimating the negative skin friction have not been clarified yet. The Steel Pipe Pile Committee of the Japanese Society of Soil Mechanics and Foundation Engineering carried out long term in-situ measurements of the negative skin friction and conducted extensive studies on its behavior.

Several different treatments have been employed in previous papers. The existence of the neutral point that was a boundary between the positive and the negative skin friction was sometimes taken into account (Plomp and Mierlo 1948, Ahu and Habib 1960), and sometimes neglected (Zeevaet 1959, Johannessen and Bjerrum 1965). The decrease in shear stress on piles due to relaxation phenomena was sometimes taken into account (Bessyo 1960) and sometimes was not.
In-situ measurements using four kinds of steel pipe piles have been carried out to clarify the phenomena of the negative skin friction for more than two years at a site in Tokyo where a 43 meter thick alluvium is underlain by diluvial sand. The ground surface at the site has settled at a rate of 15 cm per year.

SITE CONDITIONS

The site of the experiment which is in the premises of Takenaka Technical Research Laboratory is located at Fukagawa district about 3 km west of downtown Tokyo. The thick alluvium was deposited in a buried valley which had been formed by an old river. The test site measures 20 m by 30 m and no structures on piles exist within a 50 m radius except a few light weight wooden buildings. Soil conditions obtained from four borings prior to the experiment are shown in Fig. 1. A silty sand layer 7 meter thick underlies a 2 m fill. Under the silty sand layer, a very soft silt stratum containing sandy silt layers extends to a depth of G. L. - 39 m, and is underlain by 4 meter thick hard silt which rests on a diluvial sand layer at G. L. - 43 m. The water level in the top fill is at about G. L. - 1.5 m.

At the test site the ground surface settles at a rate of roughly 15 cm per year due to a decrease in water head in the diluvial sand layer which in turn is caused by pumping of ground water for industrial purposes. Decrease in thickness of the thick alluvial deposit is about 8 cm a year measured from a bench mark at G. L. - 55 m in the site.

PILE TYPE, INSTRUMENTATION AND MEASUREMENT

The objective of the experiment is to study the behavior of the negative skin friction acting on single piles, i.e., the magnitude of the friction, its distribution, time effect, influence of relative settlement velocity between soil and pile, and location of the neutral point. The test piles were set apart more than 10 m to minimize interference from adjacent piles. Four kinds of steel pipe piles, as shown in Table - I, were employed as test piles considering

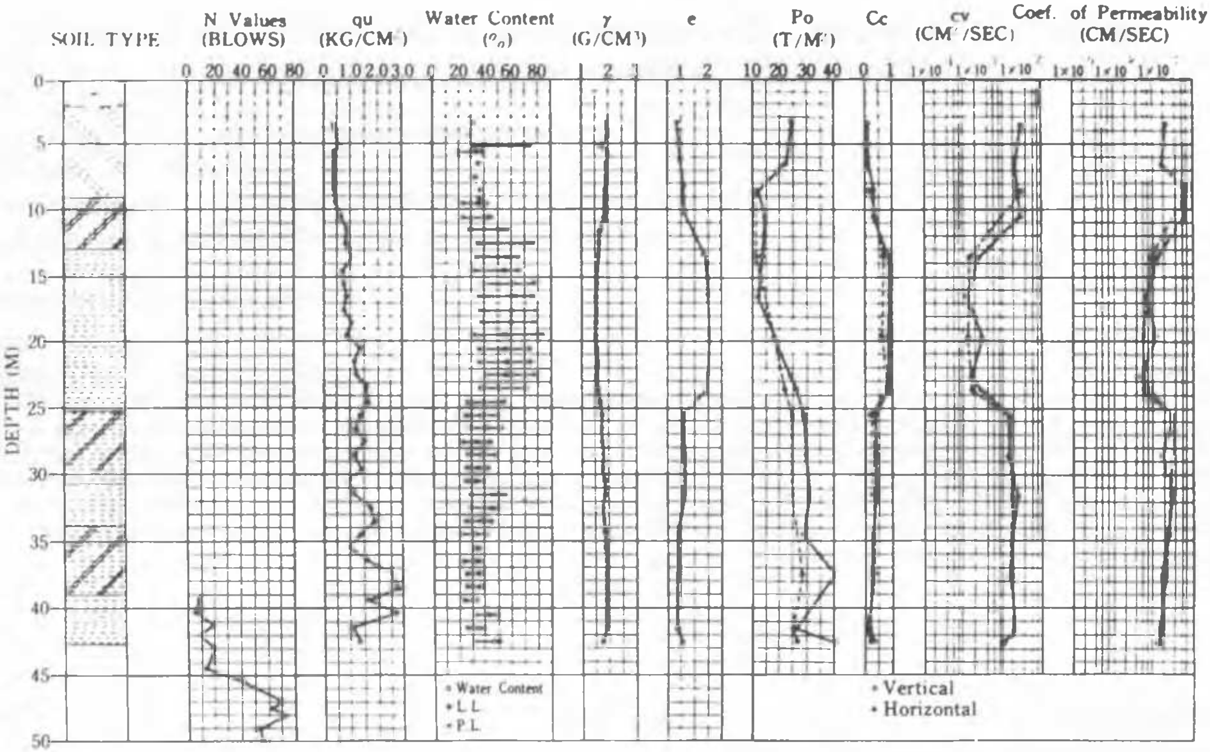


FIG. 1. Soil Conditions and Properties of Soil.

the following factors: i) difference between point bearing pile and friction pile, ii) difference between closed point pile and open point pile, iii) difference between vertical pile and battered pile. The battered pile was restrained at the top by a concrete mat which was supported by two battered prestressed concrete piles spread out 120 deg. in plan. No load was applied on the pile top so that the measured stresses in the piles would show only an effect of negative skin friction.

Various devices were installed in the test piles and in the surrounding soil as can be seen in Table - II. Differential transformer type pickups were employed for most of the devices because of their reliability in long-term measurements in the ground from both electrical and mechanical view points. Measurements were made at every 6 m depth considering soil conditions and the distribution of the negative skin friction as shown in Fig. 2.

The test piles, above mentioned, were driven during May-June, 1964, and main series of measurements were made for about two years until April, 1966, with additional measurements for another year. They were driven by a 11.6 t.m. Diesel hammer, and their total number of blows for each pile is as shown in Fig. 2. As can be seen in the figure, the point of each test pile was not sufficiently penetrated into the dense sand layer to avoid damage to the measuring instruments. The results of the measurements on the negative skin friction are shown in Figs. 3-6 and Table III - V.

Additional borings were made after the two-year main series of measurements 20 cm - 30 cm from Pile-oE43 and Pile-cE43, and inside of Pile-oE43 as shown in Fig. 8.

Table I List of Steel Pipe Test Piles

Type of Test Piles	Dimension (mm)	Length (m)	Symbols
Point Closed Vertical Point Bearing Pile	Diameter = 609.6 Thickness = 9.5	43	cE43
* Point Closed Battered Point Bearing Pile (Angle = 8 Deg.)	"	43	cB43
Point Opened Vertical Point Bearing Pile	"	43	oE43
Point Closed Vertical Friction Pile	"	31	cF31

* Pile top is fixed, using a concrete cap, by the additional two battered precast concrete piles.

SETTLEMENT OF SOIL AND PILE

The rate of the ground settlement at the surface and at each depth is given in Table-III beginning in

Table II. List of Measurement and Apparatus

Object of Measurement		Items of Measurement	Apparatus
Pile	Stress During Piling	Dynamic Strain in Pile During Piling	Wire Strain Gauge (Electric Wire and Transister Wire)
	Negative Skin Friction	Stress in Pile	Differential Transformer Type Strain Gauge
		Reactional Pressure at Pile Point	Differential Transformer Type Pressure Gauge
		Earth Pressure on Pile	Differential Transformer Type Earth Pressure Gauge
		Pore Pressure on Pile	Differential Transformer Type Pore Pressure Gauge
Ground		Settlement of Ground in Each Depth	Double Tubes with Special Point and Water Level
		Pore Pressure of Ground in Each Depth	Sliding Resistance Type Pore Pressure Gauge

June 1964 just after the test piles were driven. The ground settlement was fairly large until four months after the piles were driven and was gradually decreasing. The rate of the ground settlement decreased uniformly at the upper portion of the alluvial silt layer, but showed some influence of seasons at depth below G. L. -20 m. This tendency can be attributed to seasonal changes in the water head in the diluvial sand layer. As can be seen in Fig. 3, unbalanced water heads between the surface water level and in the sand layer undoubtedly causes ground settlement in this area. The zones above and below the 26-m depth appear to show different trends in pore pressure distribution probably due to the presence of a permeable layer at the 26-m depth. The dissipation of pore water pressure and the ground settlement show reasonable agreement except in minor details due to the frequent fluctuations in pore pressure. Settlement of piles, on the other hand, is shown in Table-IV for two years until April, 1966. As shown in Fig. 7, the amount of penetration of pile point in the ground is 1 mm during the period from 490 days to 672 days after pile driving, although a 15 mm penetration occurred at the beginning. During the period Pile-cB43 also showed 1 mm penetration, but the open point pile, on the

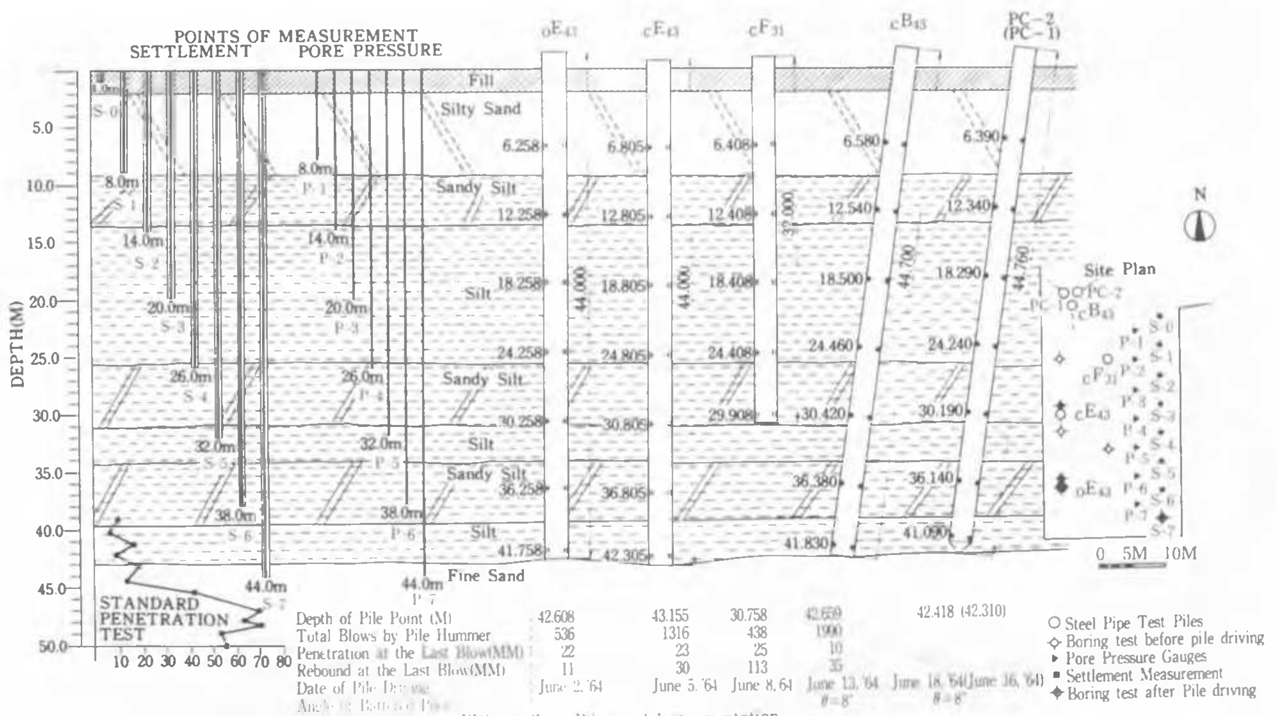


FIG. 2 Test Piles and Instrumentation.

Table III. Ground Settlement in Each Term

Depth of Measurement (M)	1st Term Jun. '64-Oct. '64		2nd Term Oct. '64-Apr. '65		3rd Term Apr. '65-Oct. '65		4th Term Oct. '65-Apr. '66		5th Term Apr. '66-Oct. '67	
	Settlement (CM)	Rate of Settlement (CM/Yr)	Settlement (CM)	Rate of Settlement (CM/Yr)	Settlement (CM)	Rate of Settlement (CM/Yr)	Settlement (CM)	Rate of Settlement (CM/Yr)	Settlement (CM)	Rate of Settlement (CM/Yr)
1.0 (So)	4.0	12.0	4.0	8.0	2.6	5.2	2.3	4.6	4.3	2.9
9.0 (S1)	3.5	10.5	3.6	7.2	2.6	5.2	2.1	4.2	4.3	2.9
14.0 (S2)	3.2	9.6	3.4	6.8	2.2	4.4	1.9	3.8	3.7	2.5
20.0 (S3)	2.3	6.9	2.5	5.0	2.0	4.0	1.9	3.8	1.7	1.1
26.0 (S4)	1.5	4.5	2.2	4.4	1.0	2.0	1.0	2.0	1.2	0.8
32.0 (S5)	1.2	3.6	0.8	1.6	0.3	0.6	0.5	1.0	0.5	0.3
38.0 (S6)	0.4	1.2	0.6	1.2	-0.1	-0.2	0.5	1.0	0.5	0.3
44.0 (S7)	0.25	0.75	0.3	0.6	-0.2	-0.4	0.2	0.4	0.1	0.07

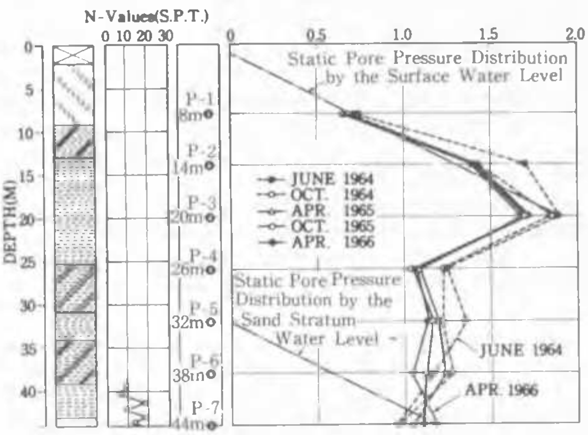


FIG. 3. Time v.s. In-Situ Pore Pressure.

other hand, showed 5 mm penetration and the friction pile 7 mm. It must be noted that the pile point has nearly ceased to penetrate in the soil in case of the closed end bearing pile, but continues to penetrate in case of the open point pile and the friction pile. It is also noted, as shown in Table-IV, that the single pile, even a point bearing pile, when it is

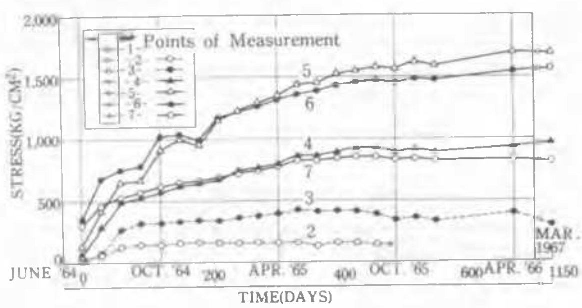
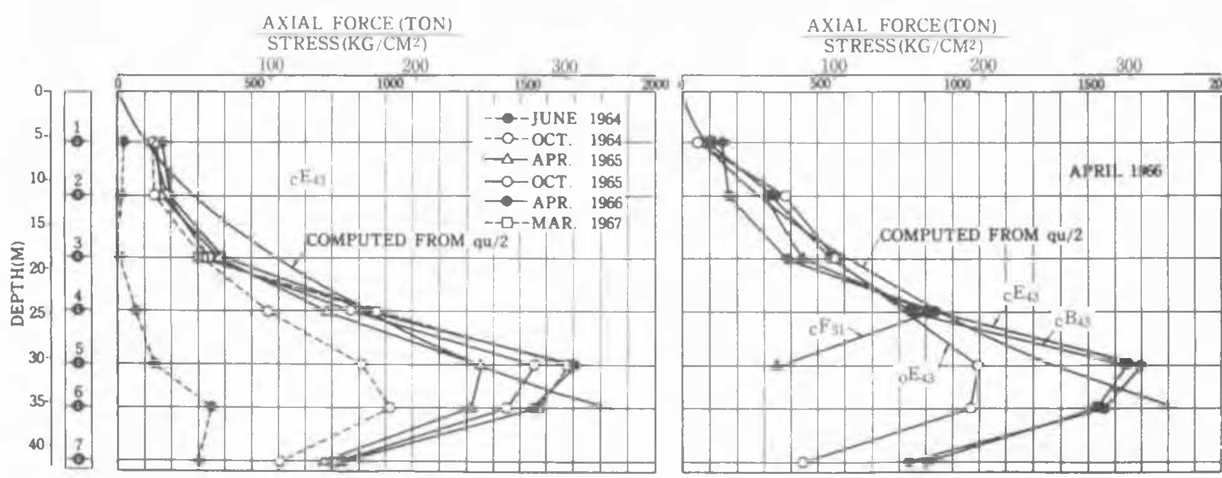


FIG. 5. Time v.s. Stress of Pile- $_CE_{43}$



(a) Time v.s. Stress Distribution on Pile- $_CE_{43}$

(b) Stress Distribution of each Test Pile (672 Days after Driving).

FIG. 4. Measured Stress Distribution on Steel Pipe Pile.

Table-IV Settlement of Test Piles (June 1964-Apr. 1966)

	Settlement at Pile Top (CM)	Relative Heaving of Pile Top to Soil (CM)	* Penetration of Pile Top to Bearing Layer (CM)
cE43	5.4	6.4	2.25
oE43	4.8	7.0	2.0
cB43	4.4	7.4	1.15
cF31	7.6	4.2	3.4

* Subtract an amount of the ground settlement at pile top and a shrinkage of pile from the settlement at pile top.

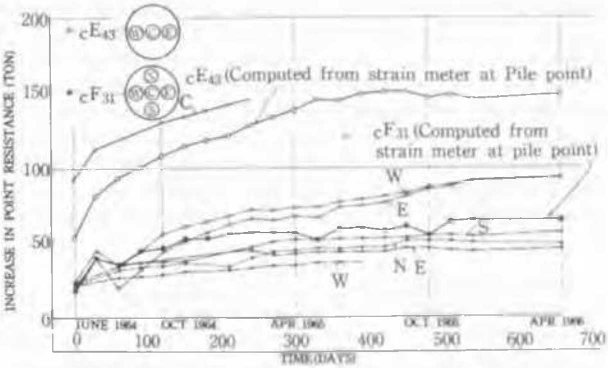


FIG. 6. Time v.s. Pile Point Reaction of Piles-$_cE_{43}$, $_cF_{31}$: omparison between Values by Pile Stresses and Pile Point Pressures.

driven through more than 40 m thick soft soil settles 4 to 5 cm at the pile top due to negative skin friction alone. Amount of settlement of both piles and ground in each depth at 124 days, 490 days and 672 days after piles were driven is shown in Fig. 7. As the negative skin friction occurs where the soil settles more than the pile and the positive skin friction occurs where the pile settles more than the soil, Fig. 7 shows the existence of a neutral point which is defined as the point at which the soil and the pile move together.

CHANGES IN SOIL PROPERTIES BEFORE AND AFTER PILE DRIVING

The soil properties before pile driving are compared in Fig. 8 with those obtained 20 to 30 cm outside the pile surface after the two-year main series of measurements. Little significant changes can be seen in the properties of soil. As far as the soil inside the open point pipe pile is concerned, however, significant decrease in strength can be seen in Fig. 8.

RESULTS OF MEASUREMENTS ON NEGATIVE SKIN FRICTION

Results of measurements of negative skin friction are shown in Fig. 4 - Fig. 6. An increase in stresses on Pile-cE43 due to the negative skin friction at various times are shown in Fig. 4 (a) and Fig. 5, and stress distribution in each test pile two years after pile driving is shown in Fig. 4 (b) After the two-year period, as little significant changes can be seen in stresses in piles, Fig. 4 (b) can be considered roughly to give maximum values of stresses as shown in Fig. 4 (a) and Fig. 5 (due to an increase in water head in the diluvial sand layer. a slight decreasing trend can be seen in the figures, but a slight increasing trend can be seen for Piles-oE43 and cF31 as shown in Fig. 9). It can generally be derived from these results of measurements that the measured

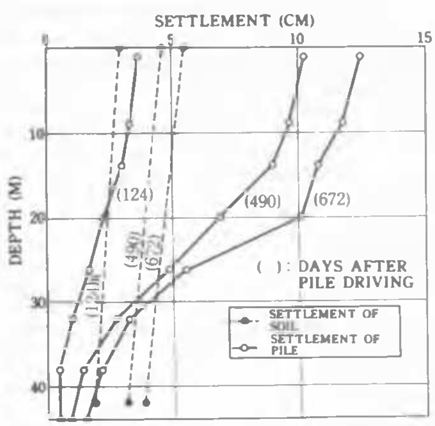
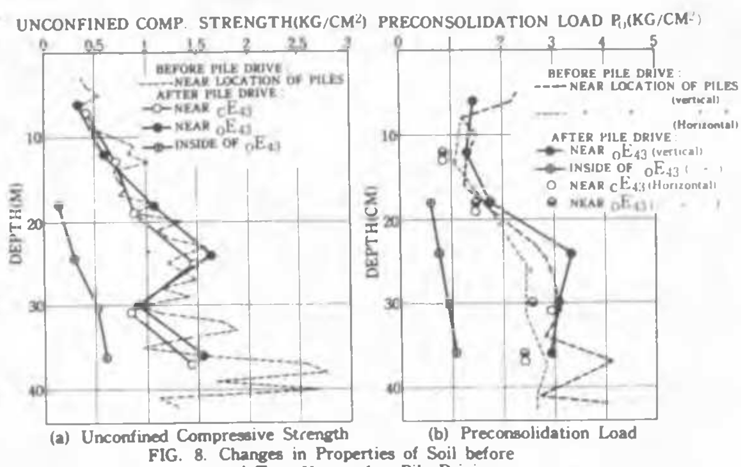


FIG. 7. Time v.s. Comparison between Settlement of Pile and Soil. (Pile-$_cE_{43}$)



(a) Unconfined Compressive Strength (b) Preconsolidation Load

FIG. 8. Changes in Properties of Soil before and Two Years after Pile Driving. (Include Inside of Opened Pipe Pile)

Table - V Negative Skin Friction 672 Days after Pile Driving

	oE43		cE43		cB43		cF31	
	τ_a (KG/CM²)	Force (TON)	τ_a (KG/CM²)	Force (TON)	τ_a (KG/CM²)	Force (TON)	τ_a (KG/CM²)	Force (TON)
Max. negative skinfriction near the neutral point	984	176	1.964	302	1,420	254	905	162
Negative skin friction at the pile point	396	71	831	149	673	121	359	64

stresses in piles show good agreement with the stresses computed on the basis of fully mobilized shear strength(qu/2), and that a clear existence of the neutral point is recognized where the stresses in piles due to settlement of surrounding soil become maximum. The rate of increase of the skin friction is most significant in the 1st term (See Table-III), and 2nd term next and so on Stresses near pile top and pile point become stable fairly early. The negative skin friction near the neutral point, on the other hand, still increases even in the 4th term. This can be described for all the test piles including the friction pile whose pile point is within soft soil. It may be described that the negative skin friction increases for a long time near the neutral point where relative velocity between soil and pile is relatively small.

Amount of measured negative skin friction in each test pile increases in the order of the friction pile, open point vertical pile, closed point battered pile, and closed point vertical pile. It is quite interesting that the relative locations of the neutral point of each test pile are roughly the same regardless of the type of pile. It is also noted that the maximum negative skin friction on each test pile, as listed in Table-V, shows fairly large values. Variation with time of stresses in pile near the neutral point is shown in Fig. 9. Variation with time of the transmitted stress to the pile point is shown in Fig. 6. The figure also contains measured reaction at the pile point. The point reaction derived from measured strains near the pile point may be considered reasonable with reference to the results of reaction measurements even though central pressure gauges of both piles were damaged(Fig. 6.).

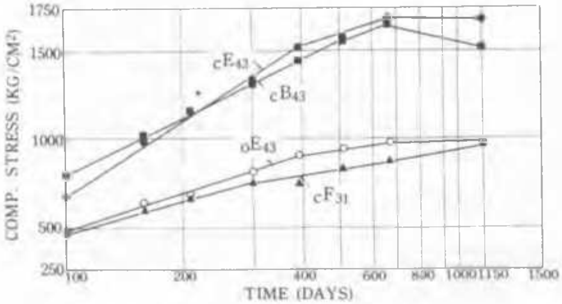


FIG. 9. Time v.s. Stresses near the Neutral Point of Skin Friction.

DISCUSSIONS ON THE NEUTRAL POINT

The portion of the pile where negative or positive skin friction is acting can be determined from the relative settlement between soil and pile as shown in Fig. 7, or from the measured stress distribution on pile as shown in Fig. 10. Fig. 10 shows clearly that the neutral points do exist, and that the location of the neutral points after two years determined from the stress distribution in the pile shows good agreement with that determined from the relative displacement. As can be seen in Fig. 11, however, the agreement is poor in early times of the measurement. This may be due to the fact that the measurement of the ground settlement was made 10 to 20 m away from the pile, and that the rate of settlement of the soil differs considerably depending on the distance from the pile during the early stage of settlement. On the other hand, the maximum axial stress in the pile can

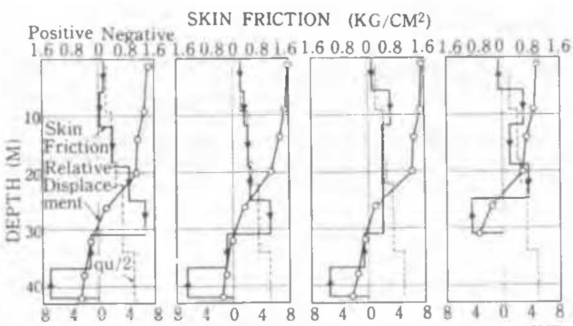


FIG. 10. Distribution of Relative Displacement between Soil and Pile, and Skin Friction.

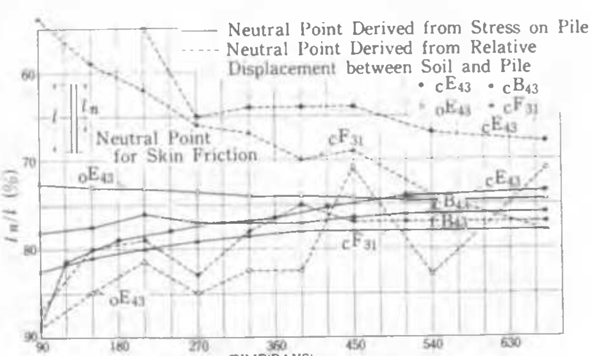


FIG. 11. Time v.s. Neutral Point Derived from Stresses on Pile and the Relative Settlement between Soil and Pile.

be determined fairly accurately by drawing a curved line over the measured points of stresses. Fig. 11 shows variation with time of the location of the neutral points which is derived from both the relative displacement between soil and pile and the stress distribution in the pile, the latter shown in full line being more reliable than the former. The following observations may be made: i) The location of the neutral point on pile changes more or less at early stage but gradually converges to a certain fixed point with time. The neutral point appears to move upwards with time. ii) The relative position of the neutral points(l_n/l) converges to a narrow range from 0.73 to 0.78, where l is the length of pile in the com pressible soil, and l_n is the distance from pile top to the neutral point. iii) As the positive skin friction increases with increasing negative skin friction, the load transmitted to the pile point shows little increase after certain period(See Fig. 6.). iv) Fairly long time must be allowed before the location of the neutral point is determined from the relative displacement between soil and pile. It is conceivable that the above-mentioned observations regarding the neutral point may be extended to other cases, because the test data include various types of piles and bearing strata, i.e., a friction pile whose pile point is in the soft ground, a point bearing pile whose pile point is in a hard layer(even though penetration was not sufficient), and open point and closed point piles.

NEGATIVE SKIN FRICTION ACTING ON PILE

Negative skin friction per unit area τ_a obtained from the results of measurements is somewhat smaller than qu/2 in case of the open point pipe pile with a few exceptions, and larger in case of the closed point vertical and batter piles near the neutral point, and larger or smaller in case of the friction pile(See Fig. 10). The relation, mentioned above, shows rough general agreement with some scattering as shown in Fig. 12. The following formula had been proposed by Johannessen and Bjerrum(1965) to compute τ_a on piles in clay: $\tau_a = \sigma_h' \tan\phi_a' = \sigma_v' K \tan\phi_a'$ . . . . (1)

The above formula can be considered suitable for the analysis of the negative skin friction phenomena, as the friction is governed by the final shear strength which the surrounding soil eventually gains after it adopt itself to a given situation. The values of the effective vertical stress, σ_v', computed on the basis of measured pore water pressure are plotted against the negative skin friction in Fig. 13, in which oblique lines indicate several values of K $\tan\phi_a'$. It is seen in the figure that the average values of K $\tan\phi_a'$ are about 0.35 in the closed point end bearing piles, 0.3 for the friction pile and 0.2 in the open point pile, although the data show a considerable scatter.

Negative skin friction and relative velocity between pile and soil: The ratio of τ_a to qu/2 is plotted in Fig. 14 against the relative velocity between soil and Piles-cE43 and cF31. The plot shows no tendency for τ_a to decrease with a decrease in the relative displacement velocity.

Load transmitted to the point of pile: Judging from the measured stresses in the piles, one will be on the safe side if one assumes that the distribution of the positive skin friction is symmetrical to that of the negative skin friction about the neutral point. This can be considered from the fact that the load transmitted to the pile point as well as position of the neutral point after some time in spite of an increase in the negative skin friction as shown in Figs. 6 and 11, and that the shear strength of lower portion of clay is generally stronger than that of upper part.

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions may be made on the basis of the measurements of the negative skin friction acting on single piles in clay:

1) Except in special cases, a neutral point exists where the axial stress in a pile is a maximum. The axial force due to the negative skin friction is transmitted to the pile point while it is being diminished by positive skin friction acting on the pile below the neutral point.
2) The observed ratio of the depth to the neutral point to the length of each pile in compressible strata fell within a range between 0.73 to 0.78, regardless of the variation in the manner in which the pile point was supported.
3) The total negative friction on the closed point piles could be estimated fairly accurately by using qu/2 as the average skin friction. In view of the physical nature of the negative skin friction, however,

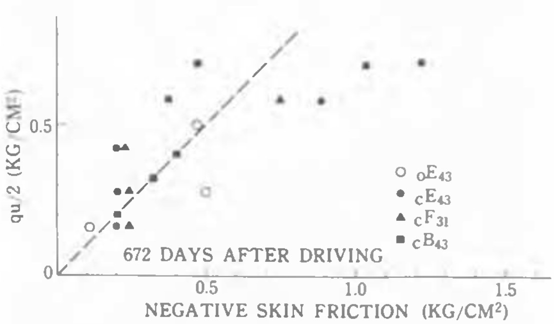


FIG. 12. Comparison between Negative Skin Friction and qu/2.

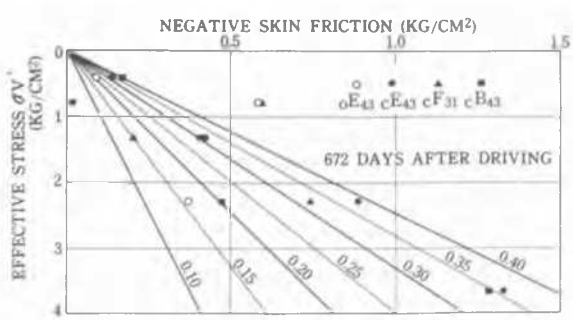


FIG. 13. Relation between Negative Skin Friction at 672 Days after Pile Driving and Effective Vertical Stress of Soil.

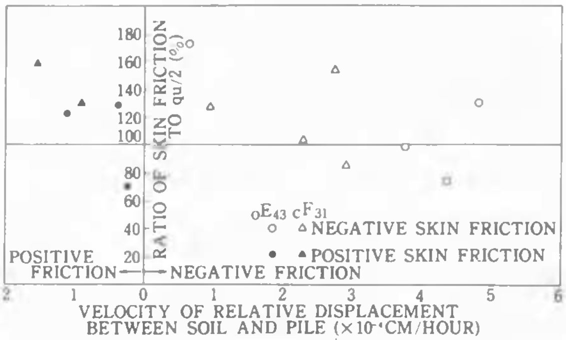


FIG. 14. Relation between $\tau a/qu/2$ and Velocity of Relative Settlement between Soil and Pile.

an expression in terms of the effective stress is considered more appropriate than qu/2. Such an expression has been proposed by Johannessen et al. as follows:

$$\tau_a = K \tan\phi_a' \cdot \sigma_v' \quad \text{......(1)}$$

The values of K $\tan\phi_a'$ determined from the test data were from 0.2 to 0.35. This seems to be comparable with the representative value for C_u/p for alluvial clays in this area. In case of the open point pile, on the other hand, the total negative friction was approximately 60% of that for the closed point piles.

4) Concerning the relationship between the negative friction and the relative displacement velocity of piles and soil, a considerable length of time was required before the negative friction was fully developed where the velocity was small. The negative friction showed no tendency to decrease with a decrease in the relative displacement velocity.

5) The observed distribution of the axial force in each pile was approximately symmetrical about the neutral point. Thus one may make a conservative estimate of that part of the negative friction which is transmitted to the pile point by taking the axial force at a distance $2l_n - l$ from the pile top, where l_n denotes the distance between the pile top and the neutral point and l is the length of the pile which penetrates the compressible strata.

From the point of view of design of pile foundations, the negative friction on single piles may be taken into account as follows: (1) check the required cross section of the pile against the total negative friction acting on the pile above the neutral point, F_{Nmax} , and (2) check the bearing capacity of the soil at the pile point against that part of the negative friction which is transmitted to the pile point, F_{NP} , namely,

$$F_{Nmax} = \eta \cdot \psi \cdot \alpha \int_0^{\beta l} \sigma_{vz}' \, dz \quad \text{............(2)}$$

$$F_{NP} = \eta \cdot \psi \cdot \alpha \int_0^{(2\beta-1)l} \sigma_{vz}' \, dz \quad \text{..(3)}$$

where
- η : coefficient which depends on the type of the pile point (1.0 for the closed point piles and 0.6 for open point pile in this investigation)
- α : K $\tan\phi_a'$ (0.3 to 0.35 for the closed point piles in this investigation)
- $\beta = l_n/l > 1/2$, relative depth of the neutral point (0.73 to 0.78 in this investigation)
- l_n: distance between pile top and neutral point.
- l : length of pile in compressible strata.
- ψ: perimeter of pile

The characteristics of the coefficients, η , α and β , should be clarified by further research.

ACKNOWLEDGMENTS

The experiments reported herein were carried out as a portion of the work being conducted by the Steel Pipe Pile Committee of the Japanese Society of Soil Mechanics and Foundation Engineering under the sponsorship of the Kouzai Club (the steel manufacturers club). The support provided for this experi ment is gratefully acknowledged. The authors are deeply indebted to the cooperation of Dr. Y. Ohsaki and other members of the committee.

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