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SUMMARY The use of steel pipe piles gradually becomes more popular in recent years. One of the major advantage of steel pipe is the high strength of pipe material itself. In design, maximum utilization of the steel strength is usually considered. However, the actual design capacity is often affected by the buckling strength of the pile during driving. In order to reduce driving resistance, open end pipe piles are often used.

Steel pipe piles derive its bearing capacity from both side friction and end bearing. For open-end piles, if a dense plug could be formed inside the pipe after driving, the pile will behave similar to a closed-end pile as far as end bearing is concerned. It is thus obvious whether a good plug is formed or not has enormous effect on the bearing value of the pile.

This paper describes a case record of driving open-end steel pipe piles through soft clay layer. Properties of the plug formed inside the pipe were examined. The bearing capacity of this type of piles was compared with closed-end pipe piles by full scale loading tests.

INTRODUCTION

Steel pipe piles have broadened their usage in the piling industry due to their relatively high strength of material and speed of installation. The bearing capacity of steel pipe piles is often considered to be contributed from two major factors, i.e. the frictional resistance from the pile shaft and the end-bearing resistance from the pile tip.

In early years, the end-bearing resistance of open-end pipe pile was estimated on the basis of fully activated cross-sectional area of the pile base. However, with increasing use of the pipe piles and availability of more load test data, it has been found that this design concept is not

correct for open-end pipe piles installed in soil medium. Rather, the magnitude of the end-bearing resistance depends more on the effective length of plugging developed within the pipe. Studies on the plugging effect of pipe piles have been carried out by SZÉCHY (1961), YAMAHARA (1964), KZOS & TEJCHMAN (1977), and KISHIDA & ISEMOTO (1977), etc.

Behavior of soil plug and its effect on the load transference have been evaluated by a number of investigators. However, due to lack of measurement records of the prototype piles in the field, explanation of the mechanisms involved appears to be inadequate. This paper presents some data on an investigation carried out on the formation of soil plugs in steel pipe piles driven through soft cohesive soil layers.

BEARING CAPACITY OF OPEN-END PIPE

For a closed-end pipe pile, the ultimate bearing capacity Q_u is generally considered to be composed of two major portions, namely the total outside skin friction, Q_f , and the end-bearing resistance, Q_p , as expressed by Eq. (1):

$$Q_u = Q_f + Q_p \quad \text{-----} \quad (1)$$

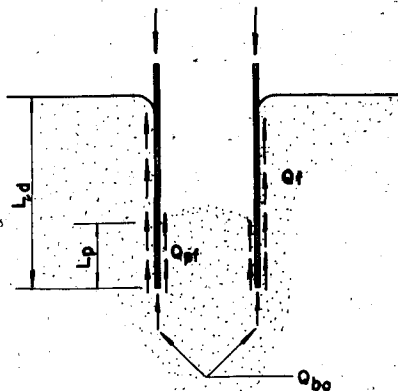
For open-end pipe pile, the bearing capacity will be

$$Q_u = Q_f + Q_{po} + Q_{pf} \quad \text{-----} \quad (2)$$

where

Q_{po} = end resistance of steel pipe section

Q_{pf} = inside skin friction of pile (Fig. 1).



- Q_f = OUTSIDE SKIN FRICTION
- Q_{pf} = INSIDE SKIN FRICTION
- Q_{po} = END RESISTANCE
- L_p = LENGTH OF SOIL PLUG
- L_d = PENETRATION LENGTH OF PILE FOR FRICTIONAL RESISTANCE

Fig. 1 Soil Plug Formed in Open-End Pipe Pile

Under most conditions, the sum of the last two terms in Eq. (2) is often smaller than the end-bearing resistance as obtained in Eq. (1) for closed-end pile. This means that the bearing capacity of open-end pipe pile is generally smaller than closed-end pile. Since the supporting capacity of an open-end pipe pile will be greatly dependant on the last two terms of Eq. (2), the magnitude of Q_{pf} will certainly affect the total quantity of piles to be used in a project. In other words, the proper formation of soil plug inside a pipe pile is a governing factor of the bearing capacity of open-end piles.

Theoretically, Q_{pf} is in proportion to the length L_p of the soil plug within the pile tube. In cohesive soils, Q_{pf} can be determined by Eq. (3) when the adhesion between the pipe wall and the soil plug is known:

$$Q_{pf} = \pi C_a d_i L_p \quad \text{-----} \quad (3)$$

where C_a is the average adhesion of the plug over the length of the plug, d_i is the internal diameter of the pipe pile.

In cohesionless soils, the theoretical equation derived by STEFANOFF and BOSHINOV (1977) can be formulated as follows:

$$Q_{pf} = \frac{\gamma d_i}{4k_g f} \left[\exp \left(\frac{4k_g \tan \delta}{d_i} L_p \right) - 1 \right] \quad \text{-----} \quad (4)$$

where

γ = unit weight of the soil plug

δ = angle of friction between pile wall and soil plug

k_g = horizontal earth pressure coefficient of soil plug

and

$$k_g = \frac{6k'}{4 + (1 + \sin \theta)(k' + 1)} \quad \text{-----} \quad (5)$$

k' is the earth pressure constant in relation to the frictional resistance of the soil, which is expressed as the following:

$$k' = \frac{1 + \sin^2 \phi - 2 \cos \phi \sqrt{\tan^2 \phi - \tan^2 \delta}}{\cos^2 \phi + 4 \tan^2 \delta} \quad \text{---} \quad (6)$$

ϕ is the angle of shearing resistance of the soil within the pile.

In case of very dense sand formation, the value of δ is very close to ϕ , then

$$k_g = \frac{3 \cos 2\phi}{2 \sin 2\phi + \sin \phi + 3} \quad \text{-----} \quad (7)$$

When the friction between the pile wall and the soil plug becomes small, the k_g value gradually reduces to k_a , the active earth pressure coefficient.

The resistance developed from the adhesion or friction around the inner

face of the pile, as it is determined from Eqs. (3) and (4), is in fact in proportion to the length of the soil plug. The longer the plug is developed, the greater the internal friction resistance Q_{pf} will be. However, the end-bearing capacity determined of open-end pile which is the sum of Q_{po} and Q_{pf} should be smaller than the end-bearing capacity Q_p of closed-end pile. A reduction factor must be applied to the end-bearing capacity if one were going to use the static pile capacity formula to estimate the end-bearing capacity of open-end pipe pile.

Japanese code (ARCH. INST. OF JAPAN, 1974) recommends a reduction factor η based on the penetration depth of pile into the bearing stratum (L_d):

$$\begin{aligned} \eta &= 0.16 L_d/d_i & \text{for} & \quad 2 \leq L_d/d_i \leq 5 \\ \eta &= 0.8 & \text{for} & \quad 5 \leq L_d/d_i \end{aligned} \quad \text{--- (8)}$$

However, the above formula does not clearly indicate the effect of plug, although it is related since the development of plug is a function of the penetration depth and the pile diameter. KYOS and TEJCHMAN (1977) recommended that reduction factors should be applied not merely to soil resistance under the pile base but also to the skin friction as well. The reduction factors which they recommended were 10% reduction on the end-bearing and 20% for the outside skin friction.

TEST PILES

For the construction of the Hsin-Ta Steam Power Plant in the Southern part of Taiwan, steel pipe piles were selected as the most suitable foundation system for the first two power generation units in view of the subsoil conditions at the site, availability of piles locally and speed of installation. The construction was started in late 1978. During the initial stage of construction, a series of full scale pile loading tests was carried out. For the purposes of evaluating formation of soil plugs in open-end pipe piles and comparison of bearing capacities of open-end with closed-end pipe piles, three different types of steel pipe piles were driven to different depths. The piles were all 50.8 cm in diameter and 9 mm in pipe wall thickness. The bottom ends of the pipe piles were either closed, open or with a cross welded to the tip. The piles were driven into the ground with a Kobe K-65 steam hammer.

The subsoil condition at the site can be briefly described as consisting of 2 to 4 m of loose hydraulic fill overlying a layer of very soft grey clay, 2 to 9 m in thickness. Underlying the soft clay extended to a depth of over 40 m is a layer of silty fine sand. The relative density of the sand increases with depth. At the depth of about 15 m below the ground surface, there is a 4 m thick layer of soft silty clay interstratified with the sand stratum. At depths below 25 m to 30 m, medium clay lenses were also found. A simplified soil log is presented in Fig. 2. To evaluate the

formation of soil plug inside the open-end steel pipe pile, measurements were taken of the length of the plug at various stages of pile driving.

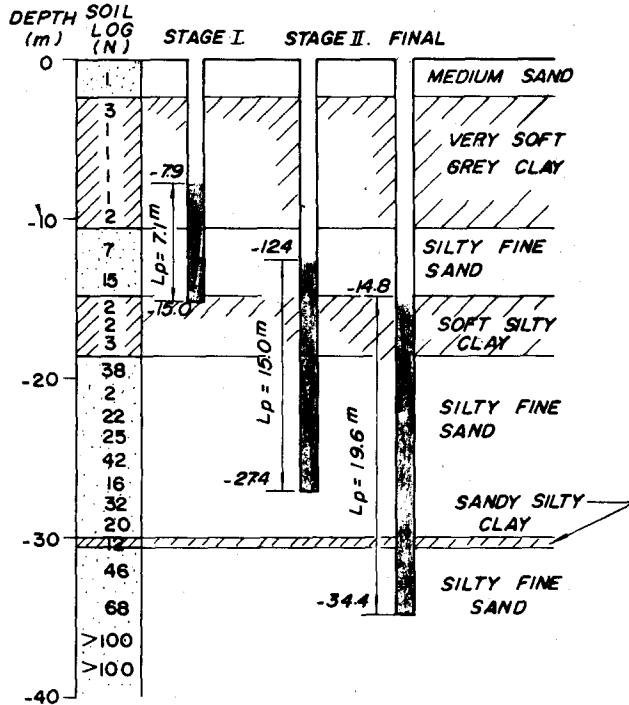


Fig. 2 Development of Plug during Pile Driving.

A typical example of the plug formation is shown in Fig. 2. The record of pile driving in number of blows per ft is presented in Fig. 3. Also shown in the same figure are the Standard Penetration Resistance values of the subsoils at a nearby location prior to pile driving. In one of the open-end pile driven to the depth of 34 m below the ground surface, Standard Penetration Tests and Split Spoon Sampling were carried out on the soil plug inside the pipe.

It is clearly indicated by the record of the borehole drilled through the soil plug within the pipe, Fig. 2 and 4, that the upper soft clayey soil had been squeezed out during pile driving. The length of soil plug formed is 7.1 m for 15.0 m of penetration, very little resistance was encountered during the first stage of driving. In the second stage, when the medium dense sand layer was encountered, the plug quickly formed and continuously increased in length when the penetration depth increased. The ratio between the length of soil plug (L_p) and the length of penetration (L_d) gradually increased from 0.47 at the first stage, to 0.55 at the second stage, and to 0.57 at the final settling level.

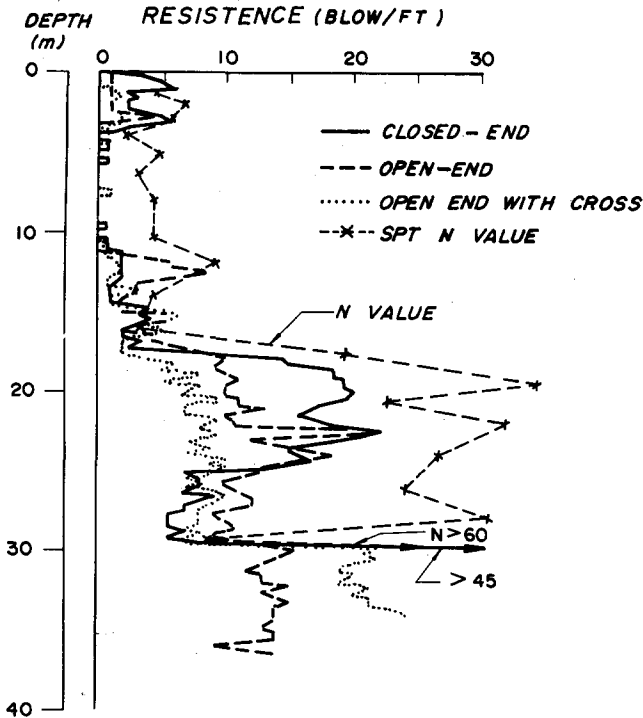


Fig. 3 Driving Resistance of Piles with Open-End, Crossed-Tip and Closed-End

Results of sampling and Standard Penetration Tests performed on the soil plug as shown in Fig. 4 indicate that there were no significant densification occurring in the soil plug as compared with the original soil. The upper part of the soil plug appeared to be looser than the original soil. The soil was more likely being pushed to the sides of the pipe instead of going into the pipe.

Comparing the behavior of closed-end pile with that of open-end and pile with crossed-end, it was interesting to learn from the driving records shown in Fig. 3 that a higher driving resistance was encountered for close-end pile than open-end pile while driving through sandy soil stratum, but it was reverse when driven through cohesive stratum. Pile with a cross at the tip had lowest resistance when it was driven through the upper sand layer. The resistance became greater than the open-end pile when the piles reached the dense sand layer at deep depth below 30 m.

Some of the results of pile loading tests which have been carried out on different lengths of piles are shown in Figs. 5 and 6. The ultimate

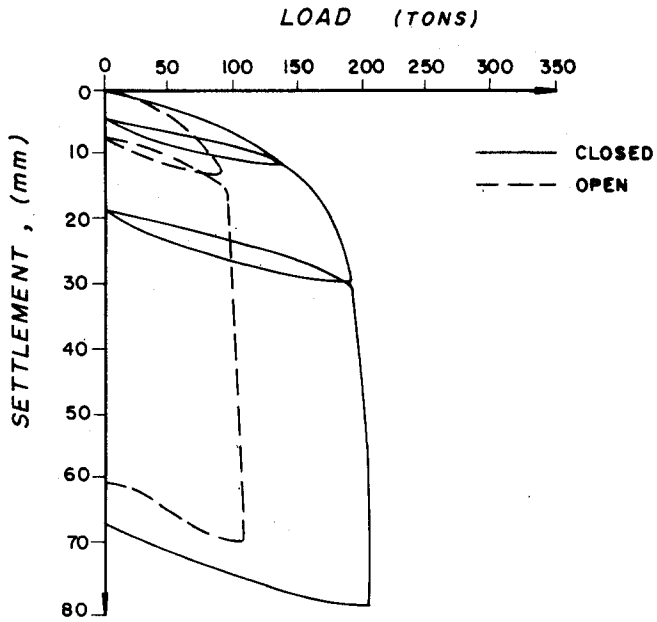


Fig. 5 Comparison of Ultimate Loads between Open and Closed-End Piles (21 m Long)

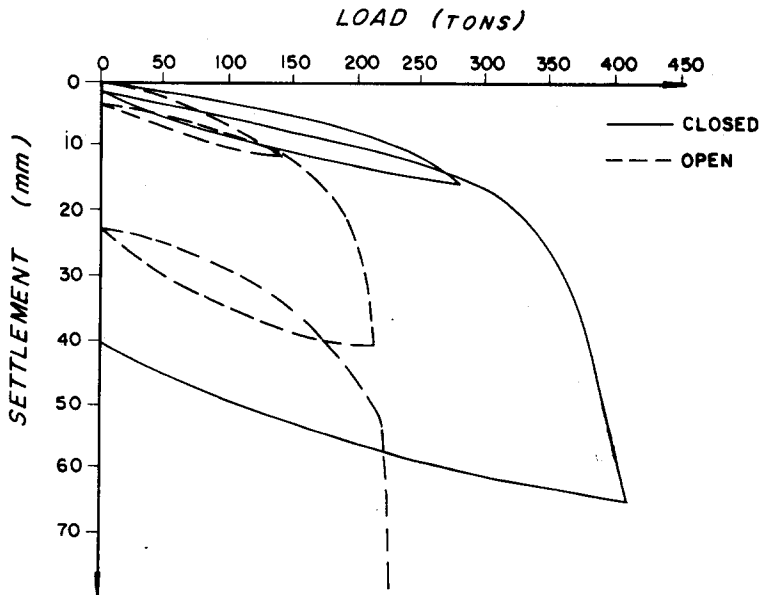


Fig. 6 Comparison of Ultimate Loads between Open and Closed-End Piles (32 m Long)

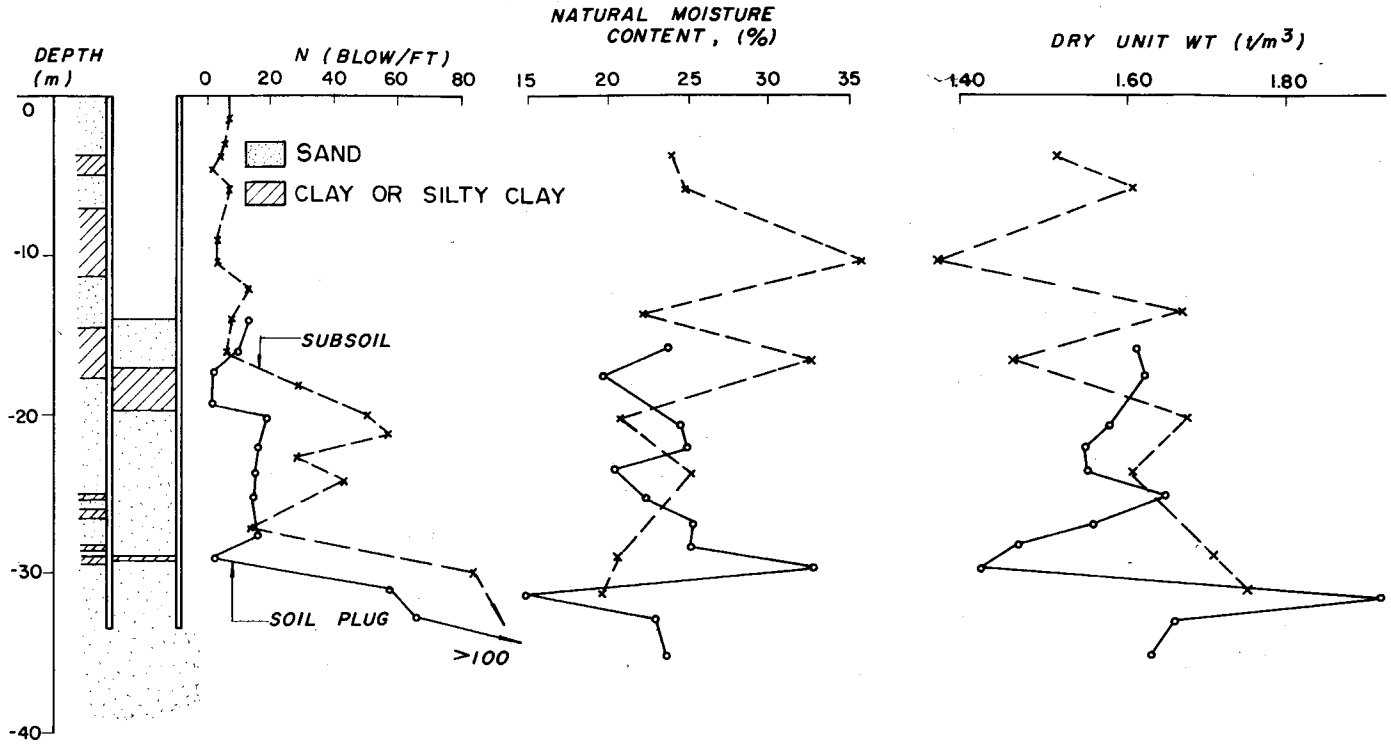


Fig. 4 Penetration Resistance of Subsoil and Soil Plug

bearing capacities of the open-end piles are only approximately 60% of the ones in closed-end condition. It is also clear that for the open-end piles, even the plug had been formed to a great length, the bearing capacity apparently did not reach the value as predicted by theoretical equations proposed by other investigators. Unfortunately due to practical restrictions encountered in the construction schedule, no loading tests were carried out on those piles with a closed-end.

Based on the pile loading test results, some back calculations were made by using Eqs. (3) and (4). It is found that the frictional resistance between the soil plug and the inside wall of the pipe pile is extremely small. The value of the δ angle is only in the order of 2 to 3 degrees. This may be caused by some lubricating effect of the remolded soft cohesive soil. This phenomenon suggests that not only a reduction factor should be used for calculating the end-bearing capacity of pipe piles in open-end condition, but a higher reduction factor have to be considered if the pile is driven through soft cohesive soil stratum.

Table I lists the estimated bearing capacities of the test piles by using Meyerhof's formula, (MEYERHOF, 1976), Japanese Building Standards (ARCH. INST. OF JAPAN, 1974) and DM-7 method (U.S. DEPARTMENT OF NAVY, 1974). The reduction factor as suggested by the Japanese Building Standards, i.e., Eq. (8) was used for estimation of the bearing capacities of the open-end piles. It is apparent that the reduction factors, as recommended by the Japanese Building Standards require modification to consider the actual plugging length and the condition of the ground. For pipes penetrating through soft clay layer and embedded in sand stratum, higher reduction factor would have to be considered.

Table I Ultimate Bearing Capacities of Steel Pipe Piles
(in tons)

Bearing Capacity	Closed-End				Open-End	
	Meyerhof	Jap. Stand.	DM-7	Test Pile	Jap. Stand.	Test Pile
21 m Piles						
End Bearing	176	132	127		104	
Skin Friction	20	20	38		20	
Total	196	152	165	180	124	100
32 m Piles						
End Bearing	336	252	186		202	
Skin Friction	117	117	237		117	
Total	453	369	423	360	319	205

It has been observed that during both the driving and loading tests of the open-end piles, no water was found inside the pipes, although the ground-water level at the site was found to be very high, existing at a depth of only 1.0 to 1.5 m below the ground surface. It is very likely that some uplift pressure has contributed to the reduction of the bearing capacity of these piles. However similar condition could also exist for the closed-end piles. More detailed study by incorporating instrumentations in the piles should be carried out in order to elucidate the mechanisms involved in the development of bearing resistance of the pipe piles.

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