International Conference on Case Histories in Geotechnical Engineering
(1984) - First International Conference on Case Histories in Geotechnical Engineering

07 May 1984, 11:30 am - 6:00 pm

Non-Negative Skin Friction Piles in Layered Soil

S. L. Lee
National University of Singapore

G. P. Karunaratne
National University of Singapore

S. Sithichaikasem
Siam Engineering Consultants Co. Ltd., Bangkok, Thailand

Non-Negative Skin Friction Piles in Layered Soil

S. L. Lee, Professor and G. P. Karunaratne, Senior Lecturer

Department of Civil Engineering, National University of Singapore

S. Sithichaikasem

Siam Engineering Consultants Co. Ltd., Bangkok, Thailand

SYNOPSIS A project in which a special type of steel pipe piles which reduce negative skin friction is described. The piles are driven in a layered soil which is subjected to subsidence due to pumping. The design concept in assessing the pile length, carrying capacity, settlement, construction control and load tests under these special conditions are discussed.

INTRODUCTION

In determining the bearing capacity of piles in layered soils the limiting point resistance and limiting shaft resistance are estimated based on pile diameter, depth of penetration, thickness and density of the bearing soil, strength of overlying and underlying soil and the method of installation of piles (Meyerhof, 1976). In subsiding soil an additional load due to negative skin friction has to be considered. The design of the foundation system for a highrise structure in subsiding layered soil is described in the following.

SITE CONDITION

The proposed Shangri-La Hotel on the bank of Chao Phraya River in Bangkok consists of a 27-storey tower block, a podium, a 11-storey car park and a 3-storey car park as shown in Fig. 1. The soft Bangkok clay from the surface down to about El-13 m rests on the alternating layers of stiff clay and sand. Fig. 2 shows a typical profile of the subsurface soil across the site along with the basic soil properties.

Fig. 1 Model of the Hotel Complex

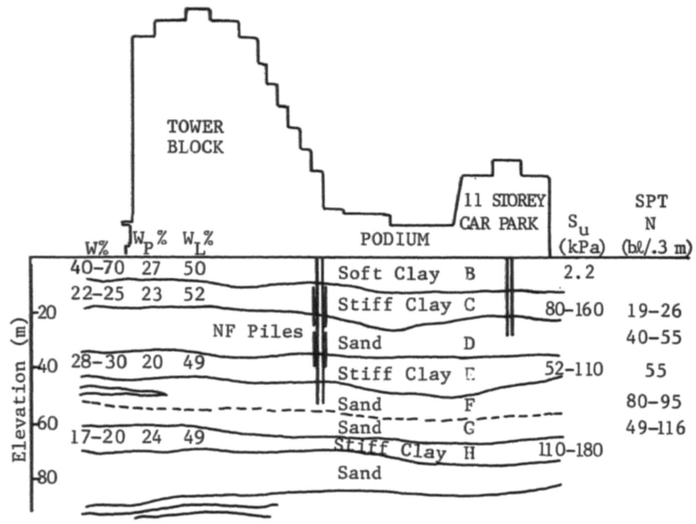

Fig. 2 Soil Profile and Depths of Pile under the Tower Block and Car Park

Pumping of water from the sand layers has reduced piezometric head in the stiff clay and sand layers. The resulting compression of clay has caused ground subsidence as much as 10 cm/year in Bangkok City (Premchitt, 1979). The annual compression is estimated to be more than 1 mm/m thickness in the soft clay layer B and about 0.65 mm/m to 0.97 mm/m in the stiff clay layers C and E. Foundations designed to rest on a given sand layer will therefore be subject to settlement due to subsidence and negative skin friction, depending on the length of the piles. The sand in layers D, F and G is medium dense to dense. The stiff clay layer H has an overconsolidation ratio of 1.5 whereas the clay in layers B, C and E is either normally consolidated or undergoing consolidation.

FOUNDATION SYSTEM

Internal column loads of the proposed hotel complex average about 3000 t for the tower block, 1200 t for the car park and 480 t for the podium. If all piles are placed on the first sand layer D then heavy columns will be subjected to large settlements. In the adopted foundation system, the piles for the tower block extended to the second layer F, and those for the podium and the car park were terminated in the first sand layer D. Construction joints were proposed between the tower block and the adjacent low rise structures. A small strip of the podium area running alongside the tower block towards the river is also supported on long piles to reduce the differential settlement between the tower block and the low rise strip in order to eliminate the necessity of having a long construction joint through the public area.

For the podium and the car parks 0.54 m square precast concrete piles were used. These piles are subjected to negative skin friction from the soft clay layer B and the stiff clay layer C.

The piles of the tower block are subject to negative skin friction from clay layers B and C, and the sand layer D, due to the settlement of soil relative to the pile. The compression of the underlying clay will be small because of the over-consolidated nature of stiff clay layer H. Only the long piles for the tower block are considered in this paper.

BEARING CAPACITY AND ALLOWABLE LOAD OF PILES

The point resistance q and skin friction f in t/sq.m may be calculated using SPT N values (Meyerhof, 1976) as

$$q = 4\ N\ D/B <= 40\ N \qquad (1)$$

$$f = \bar{N}/5 \qquad (2)$$

where $\bar{N}$ = average standard penetration resistance within embedded length of pile, N = average standard penetration value near the pile point, D = depth of penetration in sand and B = diameter of pile. The application of overburden correction to SPT N value for q gave more realistic estimates. A limiting point resistance of about 1100 ton/sq.m was used for driven piles in sand.

Allowable loads Qan and Qas under negative skin friction and short term loading conditions respectively are calculated by

$$Qan = (Qu - Qpa)/Fn - Qn \qquad (3)$$

$$Qas = Qu/Fs \qquad (4)$$

where Qu = ultimate load, Qpa = positive skin friction above the neutral point, Qn = negative skin friction, Fn = 1.5 and Fs = 3 respectively are the factors of safety with and without negative skin friction adopted in the design.

NON-NEGATIVE SKIN FRICTION (NF) PILES

The negative skin friction acting on a pil depends on the shear strength of clayey soils adhesion factor, downdrag reduction factor o the consolidating clay (Poulos and Davies 1980) and the area of the pile-soil interfac above the neutral point. The negative ski friction on 1 m to 1.5 m diameter concret piles founded in the sand layer F was estimate to be as much as 40% to 60% of the norma working load of the piles.

To reduce negative skin friction, steel pip piles with non-negative skin friction (NF layers, manufactured by Nippon Kokan Kaisha Japan, were adopted for the tower block. Thes piles consist of tubular steel sections coate with a special bitumen slip layer which i protected by a polyethylene layer (Fukuya 1982).

A steel pipe pile composed of two bitume coated NF sections, 15 m and 14 m in lengt welded to one 11 m long uncoated (black section at the top and about 12 m to 17.5 long uncoated (black) section at the bottom wa used for the tower block. The lowest uncoate section was driven into the second sand layer to develop positive friction from stiff cla layer E and sand layer F. Fig. 3 shows th soil properties and pile dimensions of typical steel pipe pile used in the project.

Under negative skin friction condition th shear stress between bitumen and steel is muc smaller. For an annual settlement of 3 cm t 10 cm of soil around the pile and for a 0.15 c thick bitumen layer the design unit ski friction at a ground temperature of 30° C, wit a factor of safety of 3, can be calculated a 0.1 ton/sq.m.

Fig. 4 shows the allowable load for variou diameters of steel pipe piles, composed of fou sections as shown in Fig. 3. Allowable stee stress is limited to 0.3 times the yield stres

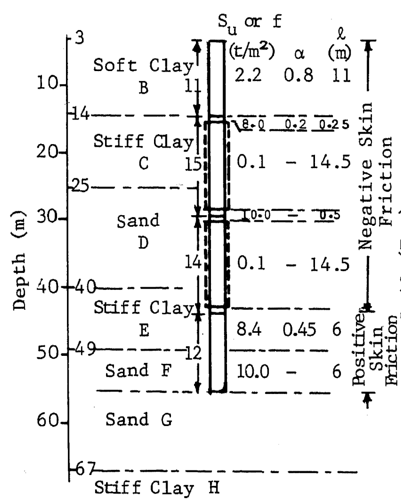

Fig. 3 NF Piles under the Tower Block

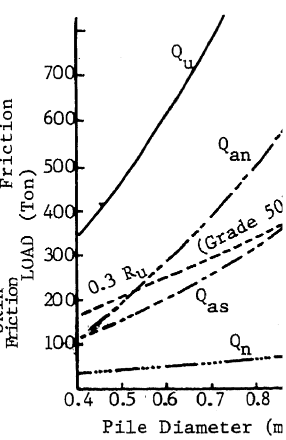

Fig. 4 Capacity of NF Piles

of steel. The most economical pile section was found to be between 0.5 m to 0.6 m in diameter. Piles of diameter 0.609 m and 0.509 m with allowable loads of 205 ton and 150 ton respectively were used for the tower block with two NF sections. Piles with steel of Grade 50 were skirted with the NF slip layer only where stiff clay and sand came in contact with the piles. The contribution of soft clay layer B to negative skin friction was very small.

SETTLEMENT

The tower block is estimated to have a consolidation settlement of 12 mm to 16 mm from the compression of stiff clay in layer H. Under the car park where 0.54 m concrete piles were used a consolidation settlement of about 65 mm is expected due to structural loading. The subsidence of the upper sand layer is estimated to be about 10 mm/year.

LOAD TEST

Fig. 5 shows the distribution of forces on a pile under driving, test loading and long term negative skin friction conditions. At the neutral point the long term axial load will be the sum of the allowable load Qan and the negative skin friction Qn. This axial load when multiplied by the factor of safety Fn gives the long term ultimate load of the pile. The positive skin friction Qpa above the neutral point is then added to obtain the ultimate test load Qt.

The load test was designed for the greater of Qt given by

$$Qt = Fs\ Qas \qquad (5)$$

$$Qt = Fn(Qan + Qn) + Qpa \qquad (6)$$

where symbols are as defined for Eqs. 3 and 4.

The failure test load on a 0.609 m diameter steel pile of the tower block is estimated to be about 540 ton derived from 160 ton of

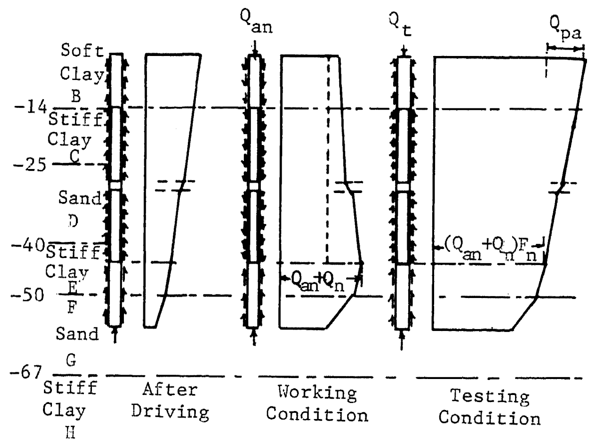

Fig. 5 Forces on NF Pile

positive friction, an estimated negative skin friction of 50 ton and an allowable load of 205 ton, with a factor of safety of 1.5.

Instrumented Load Test

A 0.609 m diameter steel pile was instrumented at nine cross sections to monitor the axial load along the pile during a load test. The ultimate load of the instrumented pile at test conditions is about 550 ton for a pile top settlement of 10% of the pile diameter (Fig. 6). For a unit skin friction in sand of 10 ton/sq.m and an average undrained shear strength of 8 ton/sq.m in stiff clay E the pile tip load at failure would lead to a point resistance of about 1000 ton/sq.m.

Therefore, when an allowable load of 205 ton and a negative friction load of 50 ton act on a 0.609 m diameter steel NF pile the axial load at the neutral point would be 255 ton. The factors of safety in accordance with Eqs 3 and 4 are found to be 1.6 and 2.7 respectively.

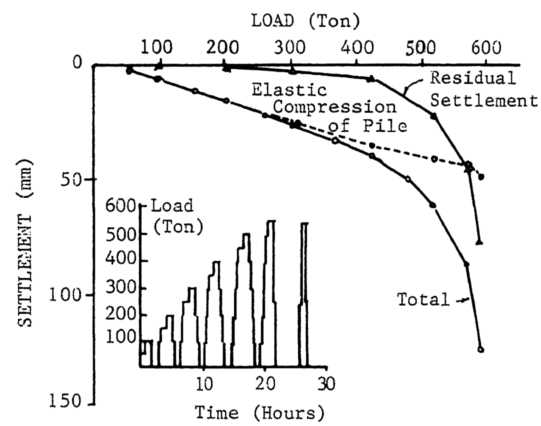

Fig. 6 Load Settlement Curve of NF Pile

CONSTRUCTION

The open ended steel pipe pile was auger-pressed through the clay layers B and C, and the sand layer D, augering the soil inside the pipe without unduly loosening the sand outside. Below the sand layer D, the pile was driven with a K45 hammer with a drop height of about 2.5 m until a set of 1 mm per blow is achieved between El.-55 m and El.-58 m.

To develop maximum tip bearing pressure all piles were driven to a penetration of about 10 diameters in sand F, while ensuring that approximately 10 diameters of sand existed between the pile tip and the stiff clay H. The latter condition was met by ensuring that no pile penetrated below El.-58 m.

Determination or Working Load and Constructional Control

At every pile location driving data such as penetration per blow and drop height provided the exact elevation of the upper surface of the second sand layer F which varied between El.-48 m and El.-52m. Within the sand layer F thin seams and lenses of clay occurred which affected the pile set. It was, therefore, necessary to determine the working load of individual piles from driving data so that the total working capacity of piles could be checked and deficiencies corrected.

The results of six test loaded piles are shown in Table I. Piles No. 4, 5 and 6 were driven with a 17.5 m extended lower section to develop adequate positive skin friction below neutral point at locations where the second sand layer F occurred below El.-49 m. Close agreement can be seen between the load test results and the predicted values from Gate's formula (Olsen and Flaate, 1967)

$$Qug = 13(EWH)^{1/2} \log_{10}(10/S) - 83 \qquad (7)$$

where E = 1, W = weight of ram in ton, H = fall of ram in inches and S = final set in inches per blow for tests conducted within about 14 days after pile installation, such as test piles No. 1, 2 and 3.

Table I: Load Test Results of NF Piles

Test No	Number of Days after Driving	Tip Elevation (m)	Set (mm/blow)	K45 Hammer fall (m)	Ultimate Load (ton) from Test	Ultimate Load (ton) from Eq.(7)
1	11	-54.1	0.85	2.6	550	605
2	14	-58.1	3.1	2.5	440	435
3	13	-55	0.72	3.0	650	680
4	8	-57.9	0.8	2.4	590	585
5	36	-57.9	4.8	2.0	550	335
6	15	-57.9	4.17	2.6	530	415

Piles No. 5 and 6 were tested 36 days and 15 days after driving respectively whereas pile No. 4 was tested only after 8 days. For both piles No. 5 and 6 the failure load was more than the predicted load showing a gain in strength in stiff clay amounting to about 105 ton per pile with the 17.5 m long lowest section. Thus for all similar piles the allowable load Qag was evaluated using pile driving data and Eq. 8.

$$Qag = (Qug - Qpa + Qe)/Fn - Qn \qquad (8)$$

where Qug is evaluated using Eq. 7, and Qe = gain in shear strength in stiff clay E amounting approximately to one third the friction from the stiff clay layer E. Qpa, Qn and Fn are as defined for Eqs. 3 and 4.

CONCLUSIONS

Pumping of water from sand layers of the substrata of Bangkok city causes ground subsidence. The piles driven to sand layers are, therefore, subject to negative skin friction. In this project to overcome excessive differential settlement, piles for the tower block were taken down to the second sand layer which has much smaller subsidence rates. To reduce the consequential increase in negative skin friction on long piles steel NF piles were used.

An instrumented NF pile of 0.609 m diameter tested after driving down to El.-55 m showed that it can take a working load composed of 205 ton of allowable load and 50 ton of negative skin friction yielding a factor of safety of about 1.6. A factor of safety of 2.7 was observed under short term loading conditions.

Installation of NF piles was carried out by 'auger-pressing' the piles down to the bottom of the first sand layer. Thereafter piles were driven through the next stiff clay and sand layers until a set of 1 mm per blow was achieved or the middle of the sand layer was reached. Augered clay was left inside the pile. A driven length of 15 m for a 0.609 m diameter pipe pile was sufficient to form an effective plug of soil so as to benefit from the gross bearing area of the pile.

Allowable load for each pile was evaluated using driving data and Gate's dynamic formula with corrections for gain in shear strength in the clay strata. Increase in strength in stiff clay due to consolidation around pile shaft was taken into account. The extra number of piles required for the design load of the structure was thus evaluated based on the driving records.

ACKNOWLEDGEMENT

The authors wish to thank Shangri-La Hotel Co. Ltd., Bangkok, Thai Ohbayashi Corporation Limited, Bangkok, Nippon Kokan K.K., Tokyo, Okura and Co. Ltd., Tokyo and Leo Property Management Pte. Ltd., Singapore, for granting permission to present the data used in this paper.

REFERENCES

Fukuya, T., T. Todoroki and M. Kasuga, (1982), "Reduction of Negative Skin Friction with Steel Tube NF Pile, Proc. 7th Southeast Asian Geotech. Conf., pp. 333-347, Hong Kong.

Meyerhof, G.G. (1976), "Bearing Capacity and Settlement of Pile Foundations" Journal Geot. Eng. Div., ASCE, Vol. 102, GT 3, pp. 197-228.

Olsen, R.E. and K.S. Flaate (1967), "Pile-Driving Formulas for Friction Piles in Sands, Journal of Soil Mech. and Found. Div., ASCE, Vol. 93, pp. 279-296.

Poulos, H.G. and E.H. Davies (1980), "Pile Foundation Analysis and Design, John Wiley and Sons, New York, p. 397.

Premchitt, J. (1979), "Land Subsidence in Bangkok, Thailand: Results of Initial Investigation, 1978, Geot. Eng., Vol. 10, pp. 49-76.