Bearing Capacity of Piles in Layered Soils

Force Portante des Pieux dans les Sols Stratifiés

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SYNOPSIS Previous bearing capacity theory and semi-empirical methods of estimating ultimate pile loads in uniform soils have been extended to layered soils. The analyses are compared with the results of model and field tests of piles in non-uniform soils of two and three layers.

INTRODUCTION

In a previous paper (Meyerhof, 1976) the bearing capacity and settlement of pile foundations in sand and clay was discussed on the basis of recent investigations. It was shown that conventional bearing capacity theory is limited to short piles of less than about 15 to 20 pile diameters in sand and non-plastic silt. The bearing capacity of piles in non-uniform and layered soils was only treated briefly, and some simple relationships were proposed for preliminary bearing capacity estimates in such soils.

In order to study this problem further, extensive model studies have been made on piles penetrating through a weak stratum into dense sand. Additional tests were made on piles resting in a sand layer of various thicknesses underlain by a weak deposit to study the resistance to punching of the piles into the underlying soil. The results of these investigations are summarized and analysed below together with some published field data of piles in layered soils.

WEAK LAYER OVERLYING FIRM STRATUM

The two-layered soils for this test series consisted of a layer of soft remolded clay of low plasticity or loose well-graded sand overlying a dense sand stratum. Details of the properties and placing of the soils in the test bed were published for similar pile tests in homogeneous and some layered soils (Clark and Meyerhof, 1973, Sastry, 1976). Most experiments were made with 3 in. diameter instrumented steel piles, which were jacked into the soil at a constant rate of penetration of 0.25 in./min. and loaded at various depths at a rate of 0.025 in./min. Additionally, 1.4 in. diameter static cone penetration tests were made for comparison.

Point Resistance:- Typical test results of the variation of the ultimate unit point resistance with depth of penetration of the piles and cones for the cases of loose over dense sand, medium over dense sand and clay over dense sand (Sastry, 1976), as well as comparative tests for homogeneous loose and dense sands, are shown in Fig. 1(a). For these short piles the thickness of the upper layer is less than the critical depth for the dense sand bearing stratum in which the point resistance becomes constant. The corresponding test results for long piles, where the

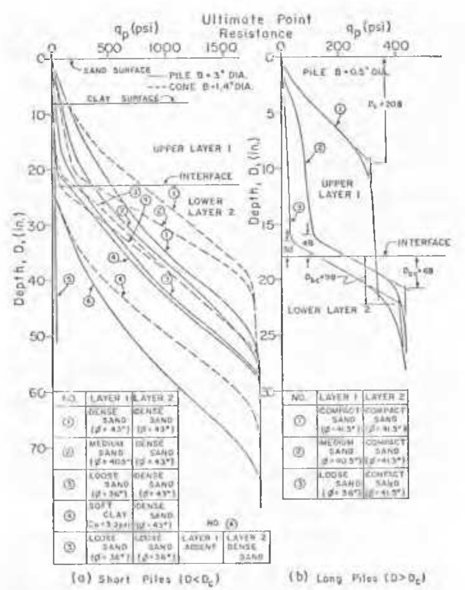

Fig. 1 Ultimate Point Resistance of Model Piles in Two-Layered Soils

thickness of upper layer is greater than the critical depth of the lower stratum, are presented in Fig. 1(b). Comparison of Figs. 1(a) and (b) indicates that the penetration curves have a similar shape. However, the unit point resistance, critical depth and scale effects are functions of the relative strengths of the two layers.

The ultimate bearing capacity Q_u of a pile may be expressed as

$$Q_u = Q_p + Q_s = q_p A_p + f_s A_s \qquad (1)$$

where Q_p = point resistance, Q_s = skin resistance, q_p = unit bearing capacity of pile point of area A_p, and f_s = average unit skin friction on shaft of surface area A_s. Further,

$$q_p = p_o N_q \leqslant q_\ell \qquad (2)$$

where p_o = effective overburden pressure at pile point, N_q = bearing capacity factor with respect to overburden pressure, and q_ℓ = limiting value of unit point resistance for $D/B \geqslant D_c/B$ where B = width of pile, D = depth and D_c = critical depth of penetration of pile (Meyerhof, 1976).

The pile test results above the critical depth (Fig. 1a) have been analysed in terms of Eq. 2, and the deduced values of N_q are shown in Fig. 2. It is found that N_q increases roughly linearly with depth, as would be expected theoretically (Meyerhof, 1963). Beyond a depth of about two-thirds of D_c, the value of N_q remains constant at a maximum of N_{qm} up to D_c when the unit point resistance reaches q_ℓ of the lower layer. Beyond D_c the value of N_q decreases and conventional bearing capacity theory can no longer be used (Kerisel, 1964; Vesic, 1967; Meyerhof, 1976).

The bearing capacity factor N_{qi} deduced from point resistance q_{pi} at the interface between the two layers is small for a friction angle $\phi_1 < 30^o$ (or $q_{\ell 1} < 15 t/ft^2$) in the upper layer, but N_{qi} increases rapidly for greater values of ϕ_1 (Fig. 3). Based on plastic theory an approximate upper bound value has been found

$$N_{qi} = q_{pi}/\gamma_1 H_1 \qquad (3a)$$

$$= N_{qo}/(1-\sin\phi_1) \leqslant N_{qm} \qquad (3b)$$

where H_1, γ_1 and ϕ_1 are thickness, unit weight and friction angle, respectively, of upper layer, and N_{qo} is bearing capacity factor at ground surface for soil of lower layer with friction angle ϕ_2. The values given by Eq. 3 are found to agree with the solution (Nagaoka, 1973) for a deep strip footing in layered soil when corrected for compressibility and using a shape factor for piles of

$$s_q = 1 + 1.5 \sin\phi \qquad (4)$$

A semi-empirical lower bound value of N_{qi} can be obtained by using the approximate relationship between the value of q_ℓ and ϕ (Meyerhof, 1974b) in

$$N_{qi} = N_{qo} + \Delta N_q \leqslant N_{qm} \qquad (5)$$

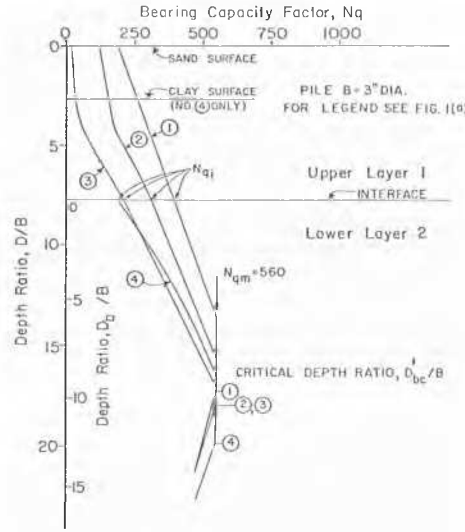

Fig. 2 Bearing Capacity Factor for Model Piles in Two-Layered Soils

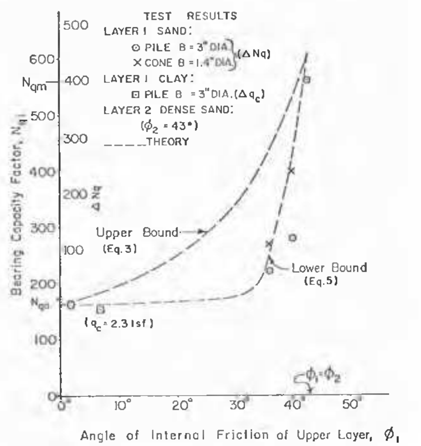

Fig. 3 Bearing Capacity Factor for Model Piles at Interface of Two-Layered Soils

where $\Delta N_q = f(\Delta q_{\ell 1})$ of limiting static cone resistance $q_{\ell 1}$ of upper layer. The observed values of N_{qi} lie close to those given by Eq. 5 due to the relatively compressible soil in the upper layer (Fig. 3). Moreover, this solution can also be used for the case of a clay layer overlying a sand stratum by taking the limiting value of $q_{\ell 1}$ of clay, as shown by the corresponding observed value of N_{qi} (Fig. 3).

The semi-empirical relationship between the bearing capacity factor N_{qi} for short driven piles and the friction angles ϕ_1 and ϕ_2 of the upper and lower cohesionless layer, respectively, is shown in Fig. 4 and has been interpolated by Eq. 5 within the limits of N_{qo} for a circular footing and N_{qm} for a short pile (Meyerhof, 1963). For piles with various depths of embedment D_b in the sand bearing stratum, the variation of the bearing capacity factor N_q with the ratio D_b/B is shown in Fig. 2. The corresponding critical depth ratio D'_{bc}/B is roughly 10 and is not much affected by thickness and strength of the upper layer (Meyerhof, 1956). Hence, approximately, for piles below the interface

$$N_q = N_{qi} + (N_{qm} - N_{qi})\, D_b/10B \leq N_{qm} \qquad (6)$$

where N_{qi} and N_{qm} are given in Fig. 4.

For long piles (Fig. 1(b)), the value of q_{pi}, the slope of the penetration curve and the critical depth in the bearing stratum are functions of the ratio $q_{\ell 1}/q_{\ell 2}$, in which $q_{\ell 1}$ and $q_{\ell 2}$ are limiting point resistances in upper and lower layers, respectively, as found previously (Puech et al, 1974). The presence of the bearing stratum is felt within about 3 to 4 times the diameter of model piles. Analysis of field data, which essentially have low values of $q_{\ell 1}/q_{\ell 2}$, indicated that, approximately, in the sand bearing stratum (Meyerhof, 1976)

$$q_p = q_{\ell 1} + \frac{(q_{\ell 2} - q_{\ell 1}) D_b}{10B} \leq q_{\ell 2} \qquad (7)$$

This relationship would be conservative for a ratio of $q_{\ell 1}/q_{\ell 2} \geq 0.2$.

Critical Depth and Scale Effects:- The present study and earlier data indicate the important influence of pile diameter on the bearing capacity of short and long piles. This effect is shown in Fig. 5 for the observed critical depths D_c in homogeneous sand and D_{bc} in two-layered soil, and the height h at which the presence of a firm layer is felt above the interface. In submerged sand the value of D_c is about 1.6 times that of a dry soil, as would be expected, and D_c decreases with friction angle and larger pile diameter. Thus, the critical bearing depth D_{bc} for full-scale piles in saturated two-layered soil is roughly 10B or two-thirds of that in homogeneous sand. This depth varies with the relative strength of the two layers and generally decreases with increasing ratio $q_{\ell 1}/q_{\ell 2}$. Further, the presence of a firm stratum is felt at approximately one pile diameter above the interface.

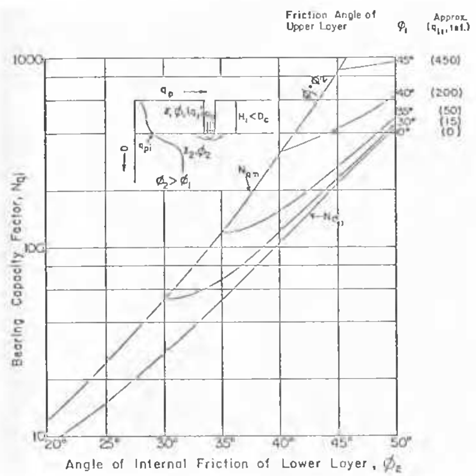

Fig. 4 Bearing Capacity Factor for Short Driven Piles at Interface of Two-Layered Cohesionless Soil

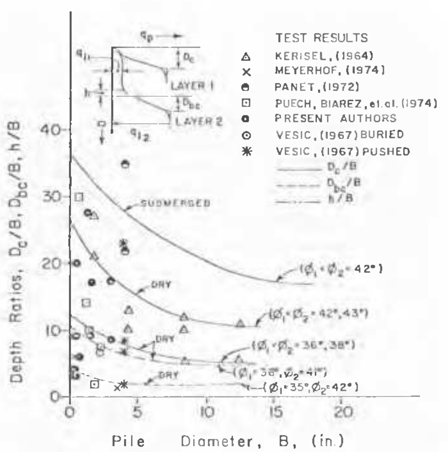

Fig. 5 Relation Between Critical Depth Ratios and Pile Diameter in Two-Layered Soils

Skin Friction:- The average ultimate unit skin friction f_s in homogeneous sand may be expressed by

$$f_s = K_s \bar{p}_o \tan\delta \leqslant f_\ell \qquad (8)$$

in which K_s = average coefficient of earth pressure on pile shaft, $\bar{p}_o$ = average effective overburden pressure along shaft, δ = angle of skin friction, and f_ℓ = limiting value of average unit skin friction for $D/B \geqslant D_c/B$, which is roughly the same as that for unit point resistance (Meyerhof, 1976).

Fig. 6 shows typical variations of the measured radial stress σ_r on the piles with depth for homogeneous and two-layered soils. The local ultimate unit skin friction f_z is directly related to σ_r so that Fig. 6 also representes the variation of f_z. The values of σ_r and f_z increase to a maximum at 2 to 3 times the pile diameter above the tip and in general have a roughly parabolic distribution with depth. Deduced values of K_s for large depths of embedment were 1.2 and 5 for homogeneous loose sand (ϕ = 36°) and dense sand (ϕ = 43°), respectively. The above-mentioned distribution of σ_r and magnitude of K_s are in fair agreement with earlier investigations (Vesic, 1967; Koizumi, 1971; Meyerhof, 1976). Comparison of curves (1), (2) and (3) in Fig. 6, shows that the maximum value of σ_r increases with the strength of the upper layer for shallow depths of pile embedment in the bearing stratum. Further, with increasing depth of embedment the maximum σ_r and average K_s approach the corresponding values of a very thick bearing stratum.

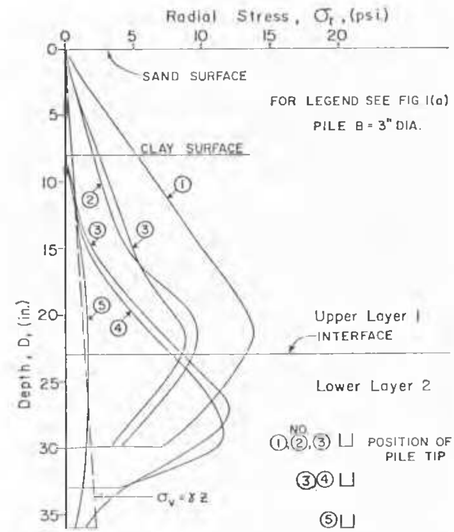

Fig. 6 Radial Stress on Model Piles in Two-Layered Soils

FIRM LAYER OVERLYING WEAK STRATUM

The punching resistance of the above-mentioned model piles and cones was studied for the cases of dense over compact sand, dense over loose sand and loose sand over clay, the soils having the same properties, as before. The thickness of the upper layer was less than the critical depth of the bearing stratum for the piles and approximately equal to the critical depth for the cones. Typical test results of the ultimate unit point resistance q_p of these short piles (Fig. 7a) show that the punching behaviour is considerably influenced by pile diameter, thickness and the relative strength of the two layers. Based on punching theory for cohesionless soil (Meyerhof, 1974a)

$$q_p = q_{02} + 2\gamma_1 (H')^2 (1+2D/H')\, sK_{ps} \tan\phi_1/B + \gamma_1 H_1 N_q \leqslant q_{\ell 1} \qquad (9)$$

where q_{02} = bearing capacity at ground surface for soil of lower layer, H_1 and H' = total thickness of upper layer and thickness between pile point and interface, respectively, K_{ps} = punching coefficient for strip footing depending on H_1/B and ϕ_1, and s = shape factor for pile (Eq. 4), and other symbols as before. The experimental punching coefficients are similar to those predicted by theory (Fig. 8)

Fig. 7(b) shows typical test results for long

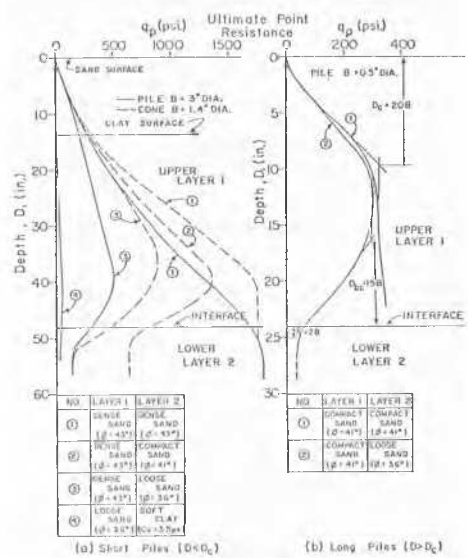

Fig. 7 Ultimate Point Resistance of Model Piles in Two-Layered Soils

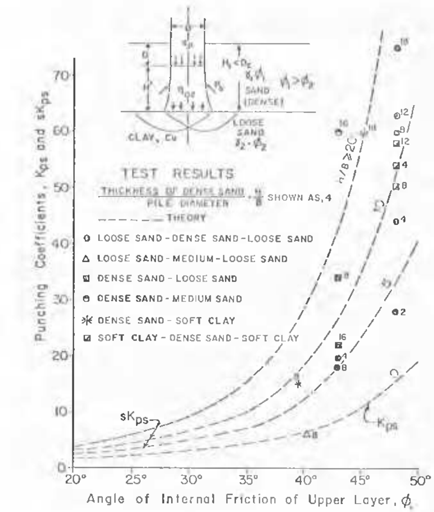

Fig. 8 Punching Coefficients for Short Piles in Cohesionless Layered Soils

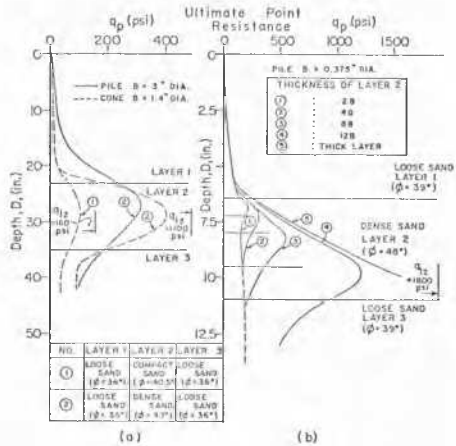

Fig. 9 Ultimate Point Resistance of Model Piles in Three-Layered Soils

piles and comparison with Fig. 1(b) indicates that the increase of punching resistance with H' approximately corresponds to the increase of point resistance for small values of D. Thus the linear semi-empirical relationship for full-scale piles in sand can be expressed by (Meyerhof, 1976)

$$q_p = q_{\ell 2} + \frac{(q_{\ell 1} - q_{\ell 2})H'}{10B} \leqslant q_{\ell 1} \qquad (10)$$

The radial stress σ_r and corresponding local skin friction f_z of the dense over loose sand tests indicate that at shallow depth of pile embedment in the lower layer the maximum σ_r in the upper stratum is practically unaffected. However with increasing depth of embedment in the weak stratum, the maximum σ_r and average K_s approach those of a weak layer.

THIN FIRM LAYER BETWEEN WEAK STRATA

Typical test results for the piles and cones in a thin compact or dense sand layer between loose sand layers are shown in Figs. 9(a) and (b), where the total soil thickness was less than the critical value of the firm layer. Due to the relatively small thickness H_2 of the dense bearing layer, the maximum unit point resistance q_p is much smaller than the corresponding limiting value $q_{\ell 2}$. Moreover, the magnitude and position of the maximum q_p depend on the thickness ratio H_2/B (Fig. 9b), and with increasing H_2/B the maximum q_p approaches $q_{\ell 2}$ for a very thick layer. Further, the variation of q_p with depth near the upper and lower boundaries of the bearing layer depend on H_2/B (Geuze, 1953).

The effect of the ratio H_2/B on q_p can be analysed by using reduction factors on the limiting case of a very thick bearing layer. Thus, for short piles (Eqs. 5 and 6) the reduction factors R_i and R_m at the upper interface and near the middle of a sand bearing layer, respectively, are defined as

$$R_i = N'_{qi}/N_{qi} \leqslant 1 \qquad (11)$$

and

$$R_m = N'_{qm}/N_{qm} \leqslant 1 \qquad (12)$$

in which primed symbols refer to a thin layer. The experimental R_i and R_m increase with H_2/B from less than 0.5 for $H_2/B = 2$ to unity at about $H_2/B \geqslant 8$ and 20, respectively (Fig. 10). The corresponding deduced punching coefficients sK_{ps} (Eq. 9) also increase roughly linearly with H_2/B of the sand bearing layer to a maximum value for about $H_2/B > 20$, and they are again similar to those predicted (Fig. 8).

Similarly, for long piles in a sand bearing layer with a thickness of less than twice the critical depth, or about $H_2/B \leqslant 20$, the analysis for a very thick layer can be modified by using a reduction factor $R_q \leqslant 1$ on the terms of D/B and H'/B in Eqs. 7 and 10, respectively. The factor R_q deduced from the limited number of tests on long driven piles in the field was roughly unity at both interfaces and had an average value of 0.9 near the middle of a thin cohesionless bearing layer (Fig. 10). Accordingly, the above equations hold essentially without modification, except for $H_2/B \leqslant 5$ when bending of the bearing layer becomes important.

The measured radial stress and corresponding

SYMBOL	LAYER 1	LAYER 2	LAYER 3	PILE DIA. B IN.
⊙	SOFT CLAY C_u = 3.5 psi	DENSE SAND ϕ = 48°	SOFT CLAY C_u = 3.5 psi	0.375
×	LOOSE SAND ϕ = 39° H = 8.33 B	DENSE SAND ϕ = 48°	LOOSE SAND ϕ = 39°	0.375
✳	LOOSE SAND ϕ = 39° H = 17.33 B	DENSE SAND ϕ = 48°	LOOSE SAND ϕ = 39°	0.375
•	LOOSE SAND ϕ = 36°	DENSE SAND ϕ = 43°	LOOSE SAND ϕ = 36°	1.4
▲	LOOSE SAND ϕ = 36°	DENSE SAND ϕ = 43°	LOOSE SAND ϕ = 36°	3.0
⊠	FIELD DATA	BISHOP, et. al., 1948, BOGDANOVIC, 1961; GEUZE, 1953; MEIGH, 1971		4—16

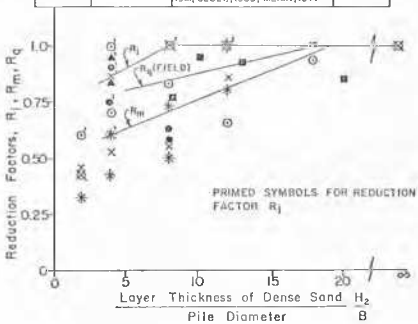

Fig. 10 Reduction Factors of Ultimate Point Resistance in Thin Layered Soils

local skin friction of the piles in a thin dense sand layer between loose sand layers were also smaller than the values for a very thick bearing layer.

PILE GROUPS

Two series of tests were made to study the behaviour of square groups of four 0.5 in. diameter piles at 1.5 in. centres in loose sand or soft clay underlain by compact sand. The results of the ultimate load per pile of the groups and for corresponding single piles show that the group efficiency in the bearing layer is roughly unity, similar to homogeneous sand, as found previously (Vesic, 1967). However, for pile groups in a firm stratum overlying a weak deposit the group efficiency is usually less than unity due to the reduced punching resistance of the group (Meyerhof, 1974a).

CONCLUSIONS

For short piles in layered soils conventional semi-empirical bearing capacity theory and punching theory can be modified and conveniently applied by using the design charts presented. For long piles, approximate bearing capacity relationships proposed by Meyerhof (1976) are found to be reasonably valid. In layered soils with a thin sand bearing stratum the maximum bearing capacity is reached near the middle of this layer, and semi-empirical reduction factors can be used to estimate the ultimate point resistance from that of a very thick layer. Available data are analysed to determine the scale effects on the magnitude of critical depth of bearing and punching resistance of two-layered soils for intrepretation of the static cone penetration resistance.

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