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Rigid Piles Under Inclined and Eccentric Loads

by

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Introduction

IN practice, piles are often subjected to combined action of horizontal and vertical loads coupled with moments. The ultimate resistance of a pile under horizontal load or pure moment was obtained based on an assumed lateral soil pressure distribution (Broms 19C4, Meyerhof and Ranjan 1972). Lateral soil pressures from laboratory and field pile load tests are reported for the cases of pure moment and horizontal load(Adam, and Radhakaishna 1973 Briaud *et al.* 1983 Kerisel and Adam, 1967). Though the bearing capacity of piles in homogeneous soils under inclined eccentric loads has been earlier presented (Meyerhof and Yalcin, 1984, Meyerhof *et al.* 1983), the lateral soil pressures and the base resistance were not studied. However the, distribution of lateral soil pressures along a pile buried in sand and subjected to central inclined load has been reported (Chari and Meyerhof, 1983).

In continuation of the above work, pile tests on fully instrumented model piles have been carried out to study the variation of lateral soil pressures along the pile shaft with the total load on the pile, the carrying capacity of the pile and its base resistance. Piles were jacked into homogeneous soils and were subjected to central inclined and vertical eccentric loads.

Model Tests

Soil Data

Dry silica sand of effective size $D_{10} = 0.38$ mm and uniformity coefficient $C_u = 2.8$ was used in the tests. The grains were angular with a texture and the initial porosity of sand was $\eta = 47$ percent. This corresponds to a loose state with a relative density $D_r = 0.2$. From detailed

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triaxial tests (Sastry, 1977) the angle of internal friction ϕ_t at this porosity was about 30° while in plane strain value of $\phi_p = 35°$ was considered appropriate.

The clay used in the tests had a liquid limit $W_L = 43$ percent, plastic limit $W_p = 21.3$ percent, and water content $W_n = 33$ percent having the degree of saturation, s_r, of about 98 percent. The average undrained shear strength of clay, c_u, was measured as 15 kPa from the unconfined tests carried out on samples obtained from the test beds.

Pile Data

The vertical steel hollow pile was 1100 mm long, 74 mm in outside diameter with a 7 mm wall thickness. The pile was split longitudinally to facilitate instrumentation and the two halves were held together by ring connectors. In each tests, 18 pressure transducers were placed in the pile along the shaft, 9 on each half at the spacing of 74 to 148 mm to measure the lateral soil pressure. The total load applied through a hydraulic jack was measured by a proving ring and the vertical component of the base resistance was recorded by a load cell of 36 kN capacity. Eccentric loads were applied to a loading arm consisting of a steel box section 100 × 150 mm which was firmly bolted to the pile cap. The displacements of the pile cap were measured by a series of dial gauges. The experimental setup is schematically shown in Fig. 1.

Test Details

Sand was rained through a funnel from constant height into a corrugated steel drum about 1000 mm in diameter and 1600 mm high. The average sand density was found to be 13.6 kN/m^3. The clay was hand backed into a durm 600 mm in diameter and 1400 mm high. The pile was jacked at a rate of about 13 mm/min. to a depth of 950 mm and an axial load tests was conducted to verify the uniformity of soil layer in all the tests. After unloading, the pile test was immediately carried out with the load at the required inclination or eccentricity. A roller bearing was placed between the jack and the raction frame to allow the movement of jack under load commensurate with displacement of the pile cap (Fig. 1). Two jacks, one vertical and the other horizontal, were used coupled with roller bearings to apply an inclined load. Pure moment was induced by two vertical jacks spaced 724 mm apart, one causing a downward force on the loading arm, while the other was causing an equal upward force through a yoke.

The failure of the pile was caused by loading in 10 to 12 equal increments and each load increment was maintained constant until the rate of displacement was less than 0.025 mm/min. in the case of piles in sand and less than 0.1 mm/min. in the case of piles in clay. Pile displacements, total load and base resistance were recorded for each load increment while

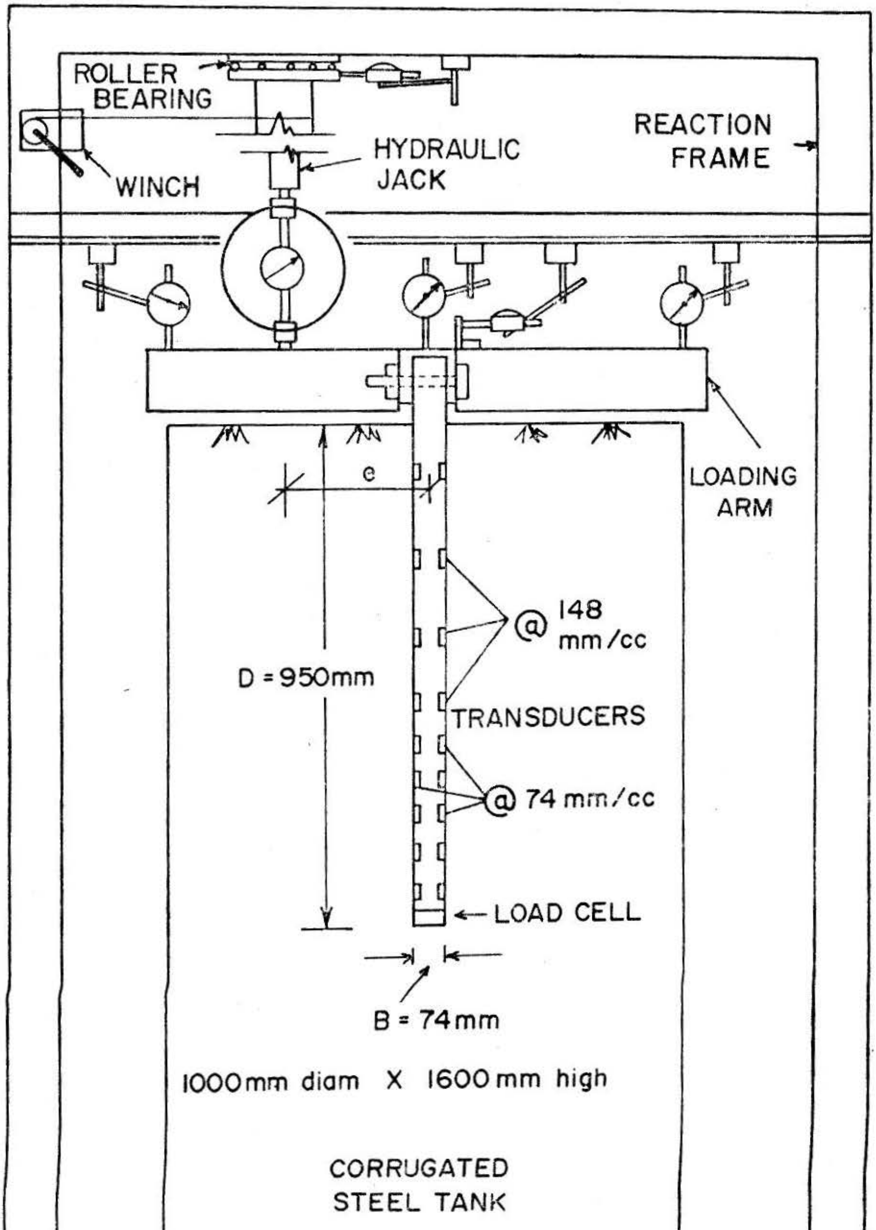

FIGURE 1. Schematic Diagram of Test Setup.

the lateral soil pressures were noted for alternate load increments. After the pile test in sand was completed, two static cone penetration tests were carried out, one on either side of the pile.

A total of six tests were carried out on the pile in sand, three under inclined loads and three under eccentric loads. The values of load inclination α with the vertical were chosen as 30, 60, and 90°; while the ratios of eccentricity of load e/depth of pile D, were $\frac{e}{D} = 0.16$, 0.38, and ∞ (pure moment). In the case of pile in clay, six tests were conducted, two under inclined loads and four under eccentric loads. The values of load inclination chosen were $\alpha = 45°$ and 90°, while the values of eccentricities used were $\frac{e}{D} = 0.16$, 0.38, 1.17 and ∞.

Test Results

Typical load-displacement and moment-rotation curves for piles in sand and clay are shown in Figs. 2 and 3 respectively. The falure load or moment at which the increase in the displacement rate first reached its miximum was at a resultant pile displacement of about 0.5 — 3.5 percent of the pile length and occurred at a rotation of 1 — 2°.

The lateral soil pressure σ_i at any depth Z, due to the installation of pile in sand was negligible compared to lateral soil pressure σ due to the subsequent loading of the pile (Fig. 4). For selected cases of loading, the distribution of σ due to increasing inclined and eccentric loads on the pile are presented in Figs. 5 and 6, respectively.

However, in case of a pile jacked into clay, considerable lateral soil pressure is induced, as if the soil was prestressed due to installation (Fig. 4). Upon loading the pile, the σ_i value at any depth Z on the passive side was found to be increased by a value of σ_p while on the active side it was reduced by a value of σ_a. This phenomenon was observed both above and below the point of rotation along the pile. The σ_p and σ_a values at any depth are numerically added to obtain the net lateral soil pressure σ due to loading of the pile. The distribution of this net pressure σ with depth at different loads, for typical inclinations and eccentricites of load are shown in Figs. 7 and 8, respectively.

For piles under inclined loads, the ultimate load Q_u and the corresponding base resistance Q_p are presented in Figs 9 (*a*) and (*b*) for sand and clay, respectively, using polar bearing capacity diagrams. In the case of piles under eccentric loads, the load eccentricity e is converted into an equivalent inclination θ where $\theta = \tan^{-1} e/D$. The failure load Q_u is plotted as ordinate, while M_u/D is plotted as abscissa for sand and clay in Figs 10 (*a*) and (*b*), respectively ($M_u = Q_u.e$).

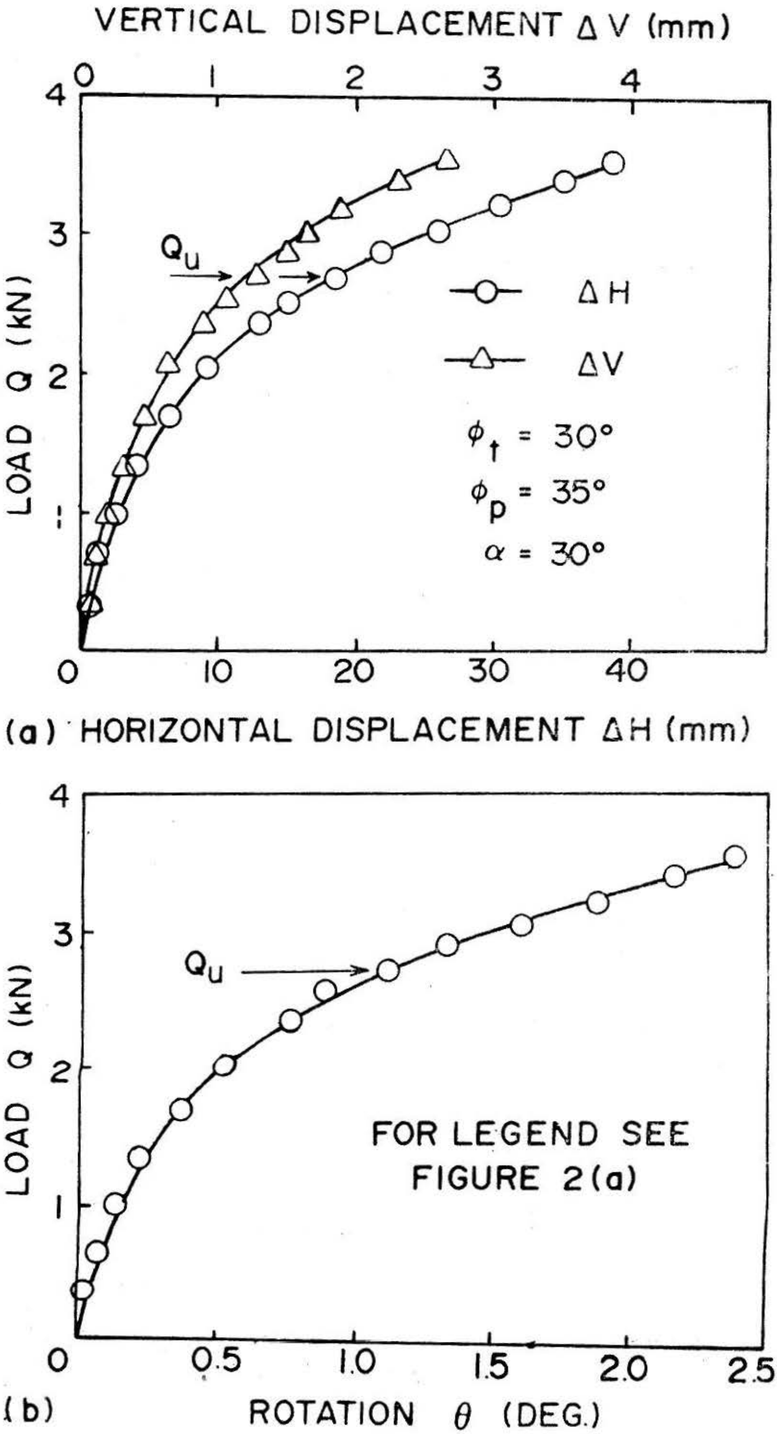

FIGURE 2. Typical Results of Model Pile in Sand under Inclined Load (*a*) Load-Displacements (*b*) Load-Rotation.

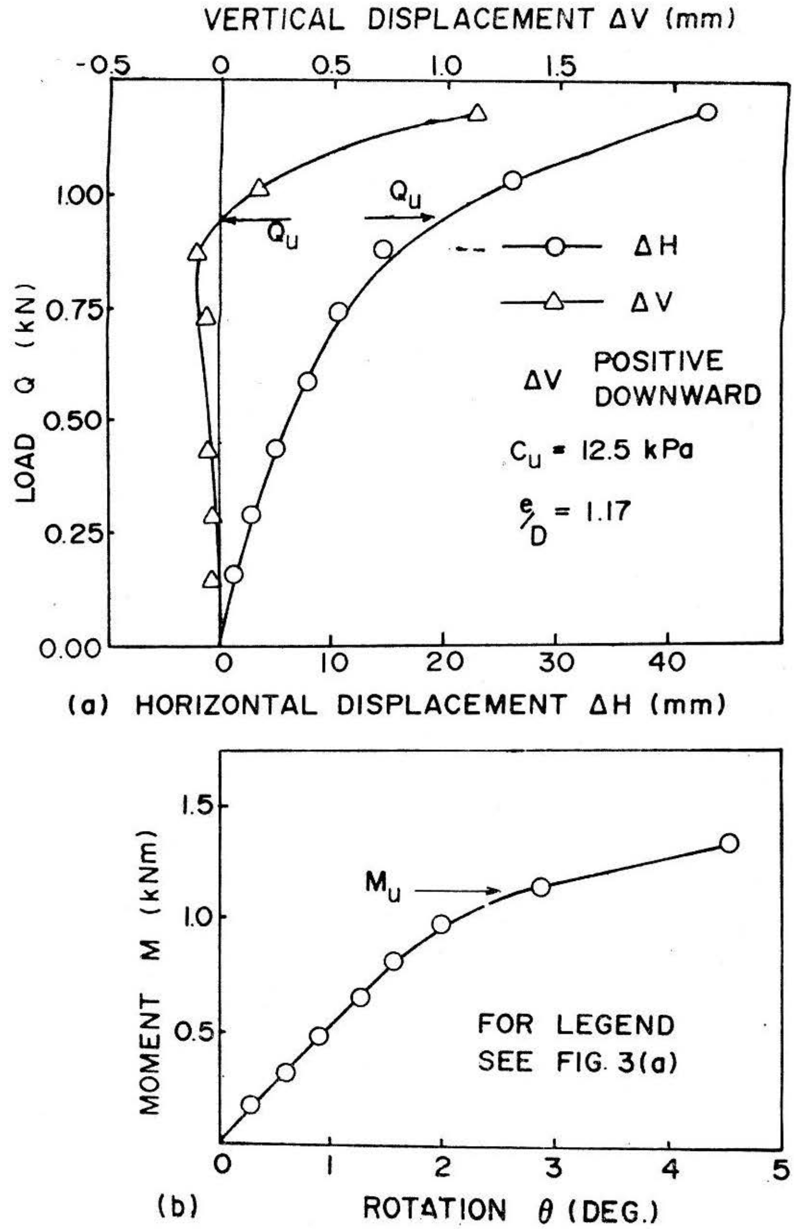

FIGURE 3 Typical Results of Model Pile in Clay under Eccentric Load (*a*) Load-Displacements (*b*) Moment-Rotation.

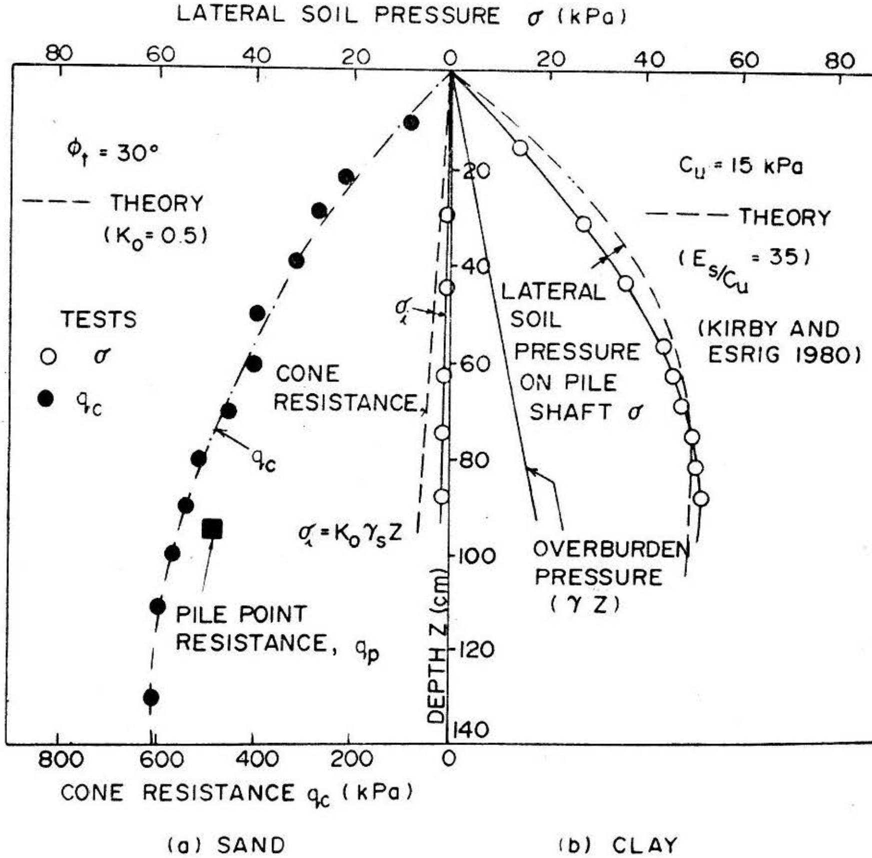

FIGURE 4 **Lateral Soil Pressures due to Pile Installation and Results of Cone Penetration Tests.**

Since the lateral soil pressures were measured for alternate load increments, the ultimate loads and moments stated in Figs 7 and 8 for some cases are slightly different from the failure loads and moments shown in Figs 9 and 10.

Analysis of Test Results

Pile in Sand

Lateral Pressures

When a fully embedded rigid pile is acted upon by a central inclined load or an eccentric vertical load Q_u, the lateral soil pressure σ at failure is approximately represented by triangular distribution (Fig. 11 (*a*)). The forces P_1 and P_2 are inclined at δ_1 and δ_2 while the point resistance Q_p is inclined at δ_3. The vertical component of Q_p is measured by the load cell while the horizontal component is obtained from the equilibrium considerations. The value of σ_i at any depth Z due to the pile installation may be

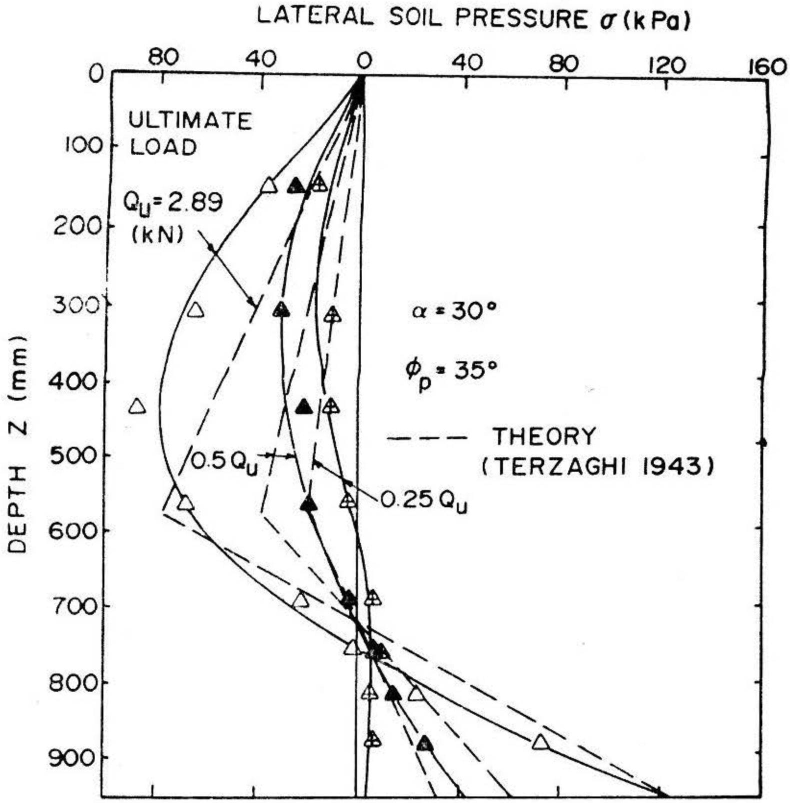

FIGURE 5(a) Varition of Lateral Soil Pressure with Applied Load in Case of Pile in Sand under Inclined Loads at $\alpha = 30°$

estimated from $\sigma_i = K_o . \gamma . Z.$ in which $K_o = 1 - \text{Sin } \phi_t$ and γ = density of the soil. The measured σ_i value due to the pile installation are found to be much less compared to the estimates (Fig. 4) and are seen to be insignificant when compared to the σ values induced due to subsequent loading of the pile. As a result, the effect of pile installation is neglected in the further analysis.

For the purpose of computing the pile capacity, the distributions of net lateral soil pressure at failure due to a horizontal load or pure moment can be roughly estimated by the triangular distribution suggested by Terzaghi (1943). The magnitude of the lateral soil pressure at shallow depth is obtained from earth pressure theory incorporating a shape factor while at the base of pile it is obtained by considering the bearing capacity of a vertical semicircular strip footing at depth. Reasonable agreement is noticed between the observed and estimated values of σ at failure (Figs. 5 and 6(b)). The effect of increasing total load Q on the magnitude of σ at any depth is analysed by examining the measured σ values under working loads (Figs. 5 and 6). The maximum lateral soil

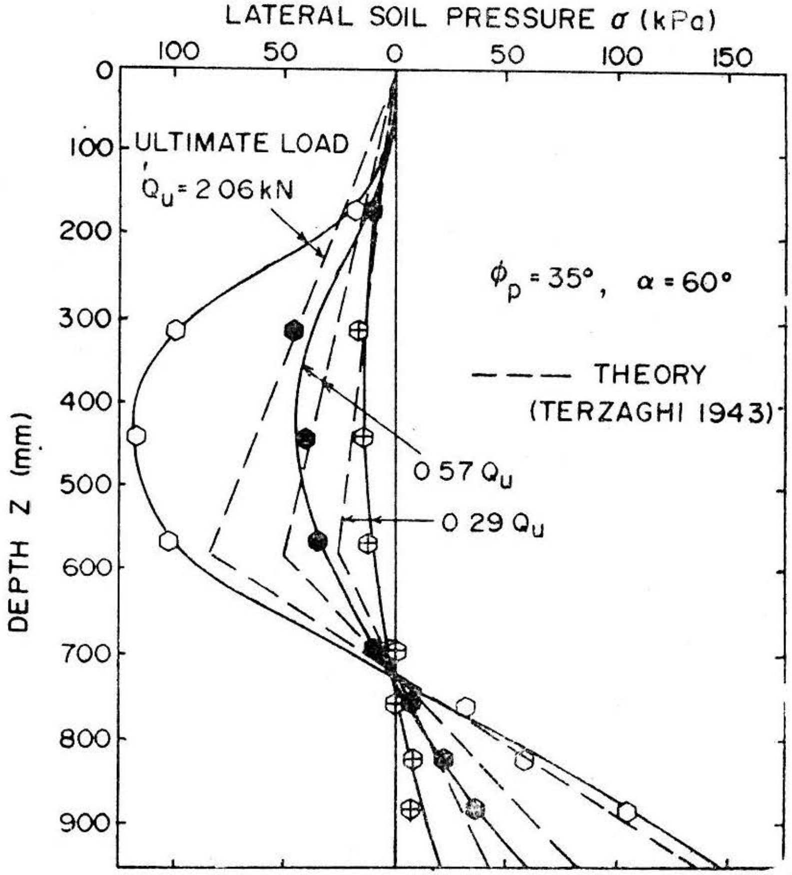

FIGURE 5(*b*) Variation of Lateral Soil Pressure with Applied Load in Case of Pile in Sand under Inclined Loads at $\alpha = 60°$.

pressures (σ_1 and σ_b), the horizontal components of earth thrusts (P_1 and P_2) on the pile above and below the point of rotation, under a given load Q (Fig. 11(*a*)) are seen to be approximately linearly varying with the load Q. Typical values of σ/σ_u at salient points along the shaft were plotted on the abscissa with corresponding values of Q/Q_u plotted as ordinates, σ_u and Q_u representing values at failure. Most of the points were seen to fall along the 45° line through origin indicating a linear relation between Q and σ (Sastry and Meyerhof, 1986). Thus, the theoretical lateral soil pressure distribution, due to an ultimate horizontal load or pure moment, can roughly be linearly stepped down in proportion to the load or moment, in order to arrive the distribution under corresponding working loads. However, this linearisation is seen to be valid only up to a factor of safety of about $F \leqslant 2$ where $F = Q_u/Q$. For intermediate inclinations and eccentricities, the ultimate σ_b value at the pile base can be estimated from Figs. 9(*a*) and 10(*a*), as will be explained in the next section. The σ distribution under working loads can be obtained proportionate to the

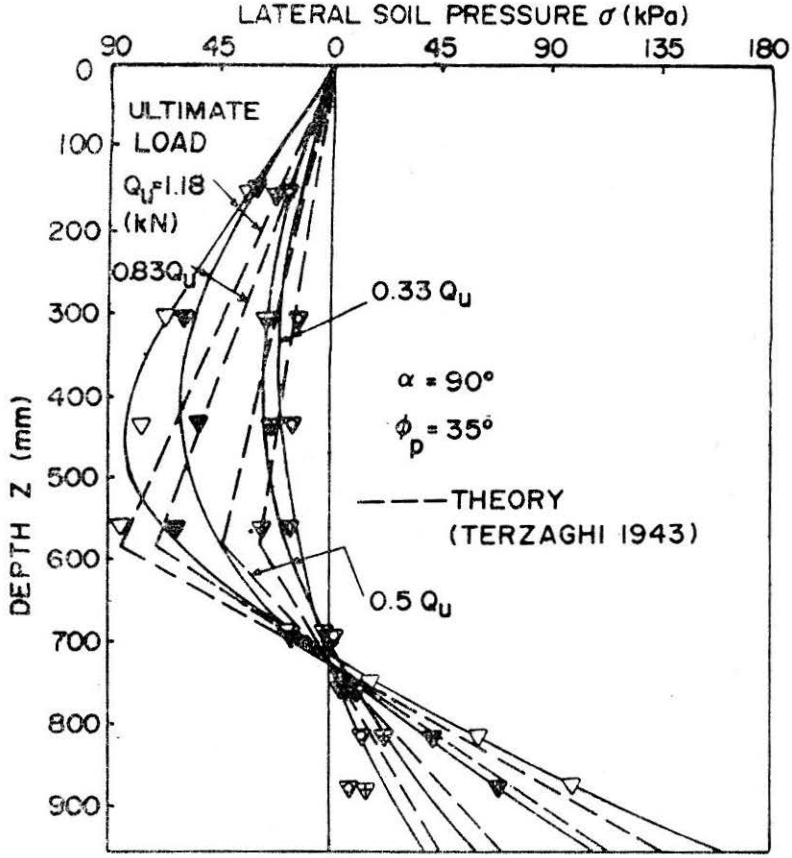

FIGURE 5 Variation of Lateral Soil Pressure with Applied Load in Case of Pile in Sand under Inclined Loads at $\alpha = 90°$.

load on the pile. Reasonable agreement is noticed between the estimated and observed values of σ for the two types of loading investigated (Figs. 5 and 6).

The depth of point of rotation, where the σ value is zero, is seen to agree closely with the estimated value for both inclined and eccentric loads. This depth is greater in the case of inclined loads when compared to eccentric loads as would be expected theoretically (Figs. 5 and 6). The actual pattern of lateral soil pressure distribution is quite complicated. It is seen to depend not only on the load level but also on the inclination or eccentricity of the load. The point of rotation seems to move downwards with increasing load for a given inclination or eccentricity of the load. Thus, the lateral pressure distribution suggested by Terzaghi (1943) seems to hold promise in the case of a pile under pure horizontal load or moment at failure.

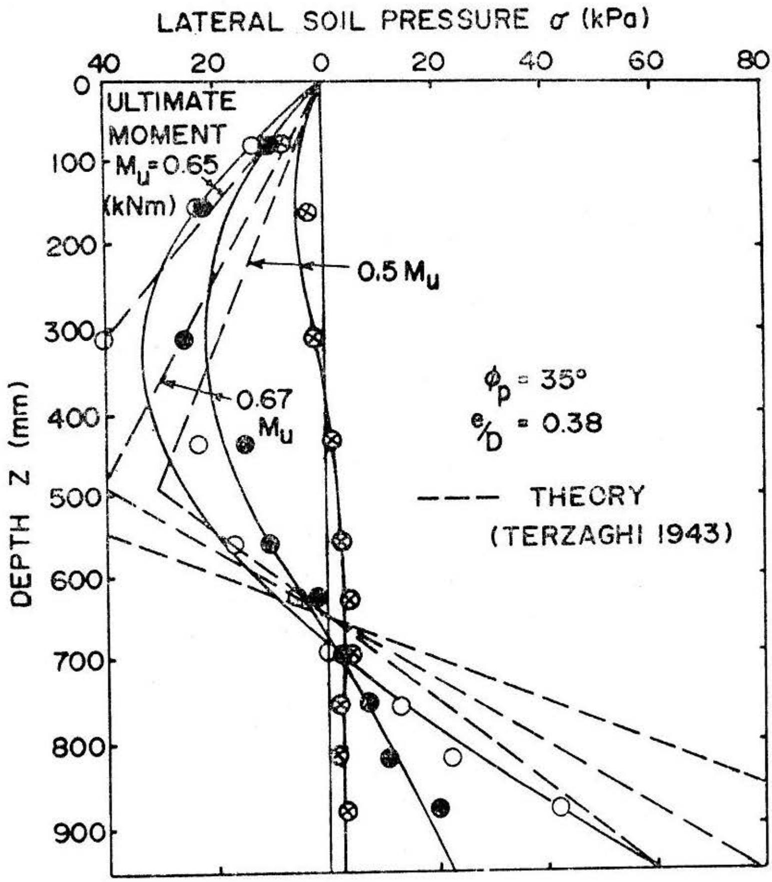

FIGURE 6(*a*) Variation of Lateral Soil Pressure with Applied Load in Case of Pile in Sand under Eccentric Loads for $e/D = 0.38$.

Bearing Capacity

The pile capacity under inclined load is bounded by two extremes of the axial and lateral capacities of the pile. The axial capacity Q_o is obtained from conventional bearing capacity theory (Meyerhof, 1976).

$$Q_o = \gamma \cdot D \cdot N_q \cdot A_p + \tfrac{1}{2} \cdot \gamma \cdot DK_s \tan \delta A_s \qquad \dots (1)$$

in which N_p = Bearing capacity factor, A_p and A_s are the areas of pile point and shaft respectively, δ = Friction angle between soil and pile, K_s = Average coefficient of earth pressure on pile shaft and the other symbols as before. The pile capacity Q_{90} under a horizontal load is obtained from the equilibrium considerations, assuming a smooth pile ($\delta_1 = \delta_2 = 0$) so that

$$Q_{90} = 0.12\, K_b\, \gamma\, BD^2 \qquad \dots (2)$$

$$= 0.12\, \sigma_b\, B.\, D \text{ where } \sigma_b = K_b \cdot \gamma\, D; \qquad \dots (3)$$

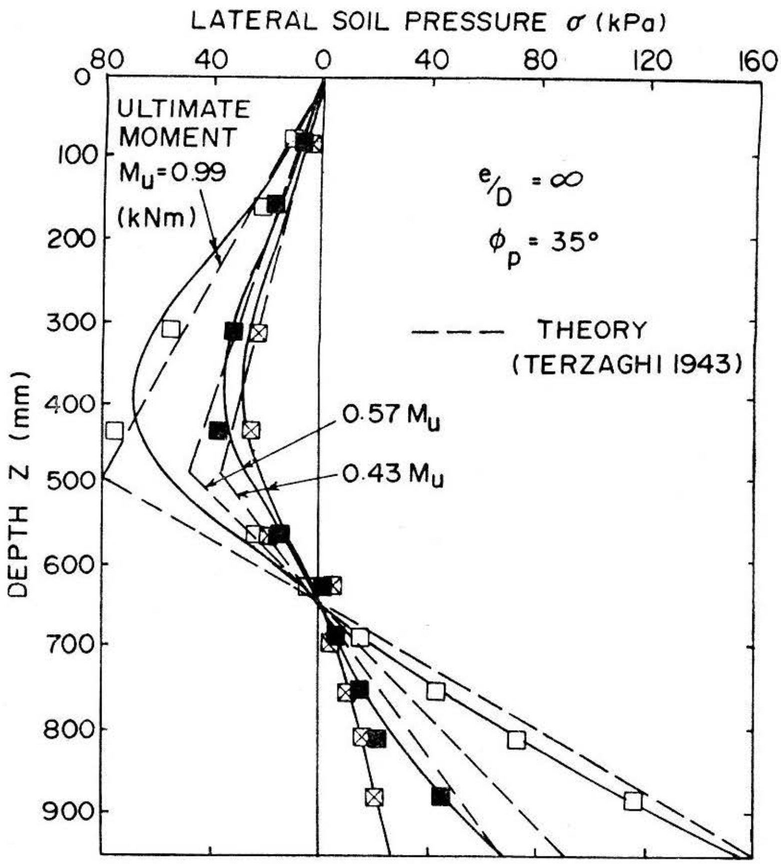

FIGURE 6 Variation of Lateral Soil Pressure with Applied Load in Case of Pile in Sand under Eccentric Loads for $e/d = \infty$.

B = Diameter of pile, K_b = Lateral earth pressure coefficient for pile (Meyerhof *et al.* 1981).

For any intermediate inclination α, Q_u may be obtained by using the inclination factor (Koumoto *et al.* 1986), so that

$$Q_u = Q_o \left[1 - \left(1 - \frac{Q_{60}}{Q_o} \right) \sin \alpha \right] \qquad \ldots (4)$$

The shaft friction δ was seen to be 0.6 ϕ_p in the case axial load while $\delta = 0$ was mobilized under horizontal load. For intermediate inclinations, a linear variation of δ may be adopted to estimate Q_u from Eq. (4) in which Q_{90} was computed with a K_b value appropriate to the δ mobilized for that inclination.

The observed values of Q_u, when compared with the estimates based on

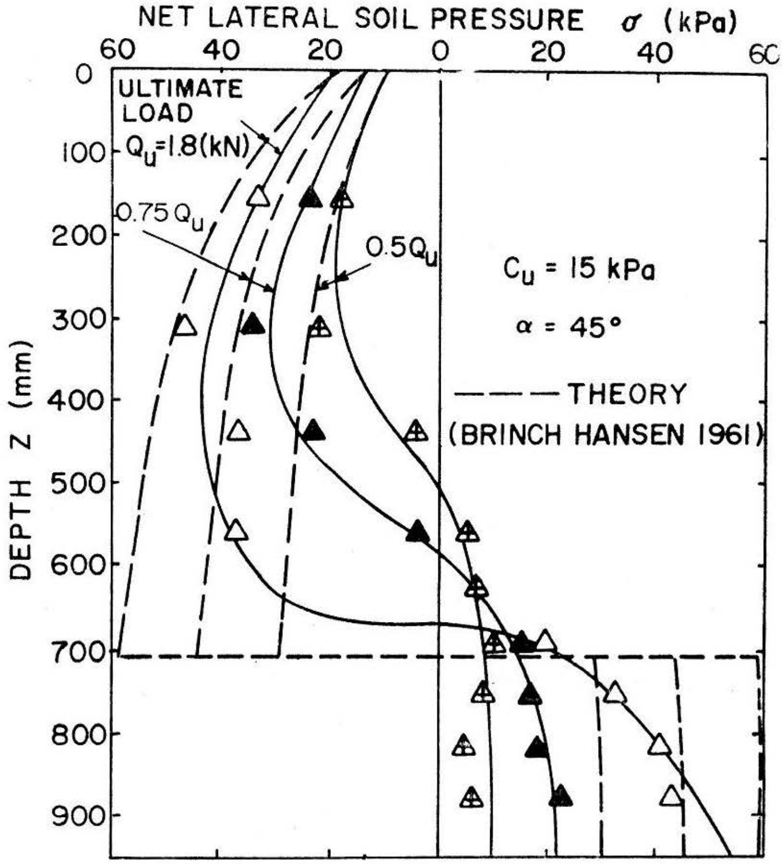

FIGURE 7(*a*) Variation of Net Lateral Soil Pressure with Applied Load in Case of Pile in Clay under Inclined Loads at $\alpha = 45°$.

a value of $\phi_p = 35°$ indicate that the theory is on the side of safety (Fig. 9(*a*)). Since Q_{90} is also expressed in terms of σ_b (Eq. 3) and σ_b value is linearly relatad to Q, the curve for Q_u in Fig. 9(*a*) can also be used to estimate ultimate σ_b for any intermediate inclination α of the load.

The pile capacity under an eccentric load is bounded by the axial and pure moment capacities of the pile. The maximum pure moment at failure M_o is obtained by assuming a smooth pile ($\delta_1 = \delta_2 = 0$) and setting $P_1 = P_2$ so that

$$M_o = 0.083\, K_b \,.\, \gamma \,.\, BD^3 \qquad \ldots (5)$$

$= 0.083\, \sigma_b\, BD^2$ with symbols as before; M_o can also be expressed in terms of Q_{90} as ... (6)

$$M_o = m\, Q_{90}\, D \text{ where } m = 0.7 \qquad \ldots (7)$$

The pile capacity under a given eccentricity e can be expressed in terms of the eccentricity factor (Koumoto *et al.* 1986).

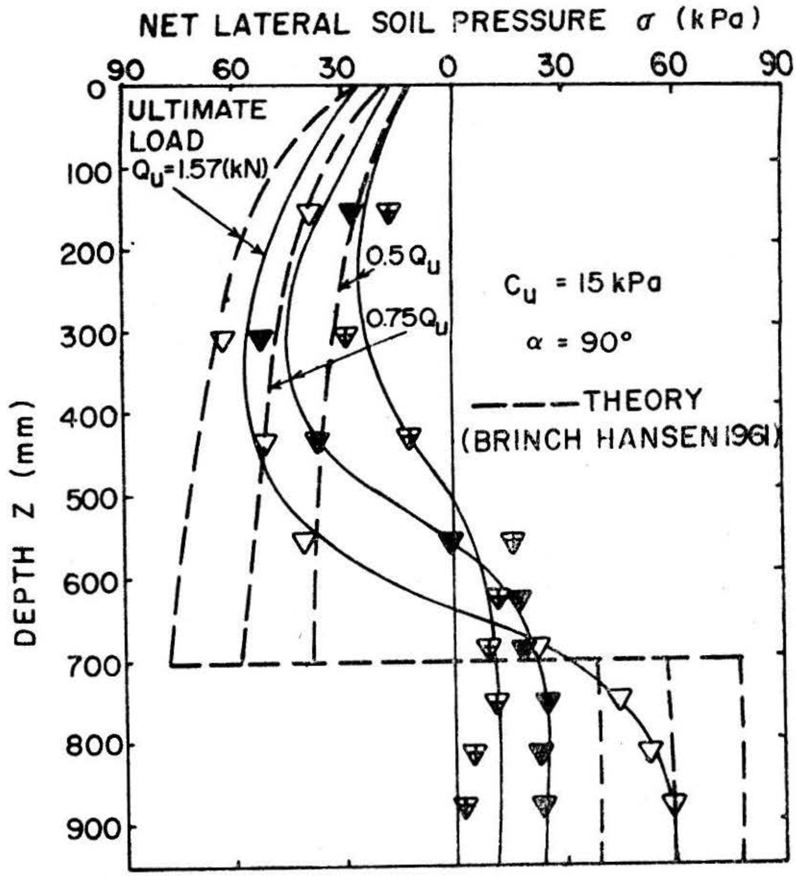

FIGURE 7 **Variation of Net Lateral Soil Pressure with Applied Load in Case of Pile in Clay under Inclined Loads at $\alpha = 90°$.**

$$Q_u = Q_o \cos\theta \left[1 - \left(1 - \frac{mQ_{90}}{Q_o} \right) \sin\theta \right] \qquad \ldots (8)$$

where $\theta = \tan^{-1}\left(\frac{e}{D}\right)$ ... (9)

In estimating Q_u values from Eq. (8), for intermediate eccentricities, a linear interpolation of δ value between 0.6 ϕ_p for the axial case and zero for the pure moment case is suggested when computing an appropriate Q_{90} for the mobilized δ. It is seen that the predicted values Q_u are in good agreement with the observed values (Fig. 10(*a*)). Once again, since M_o is expressed in terms of σ_b (Eq. 6), the curve of Q_u in Fig. 10(*a*) can also be used to estimate the σ_b values for intermediate eccentricities of the load.

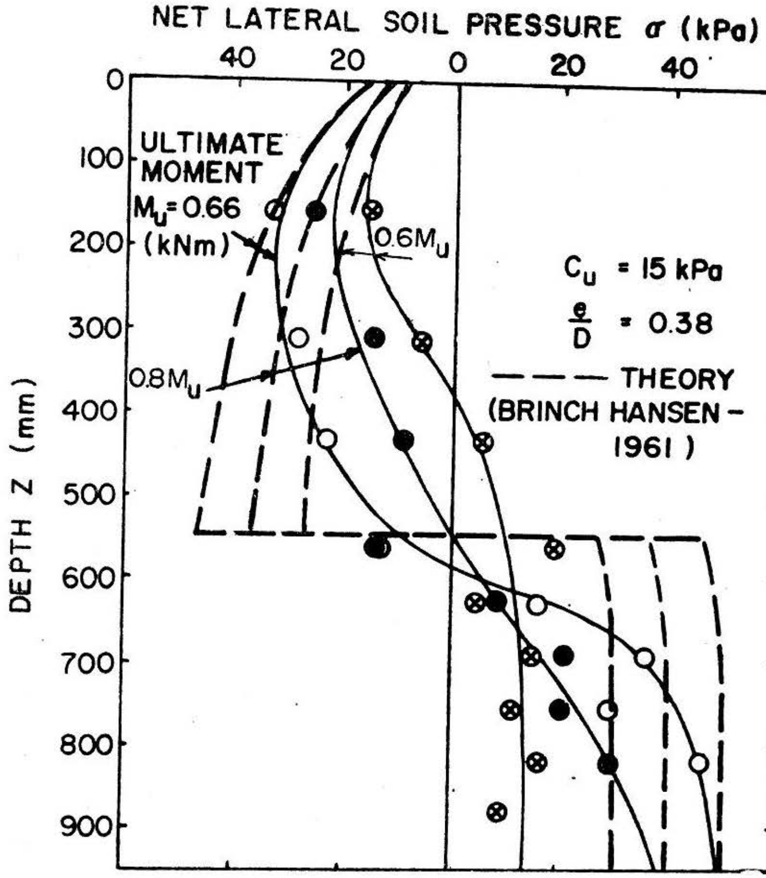

FIGURE 8(*a*) Variation of Net Lateral Soil Pressure with Applied Load in Case of Pile under Eccentric Loads for $e/D=0.38$.

The variation in the magnitude and inclination of the base resistance Q_p with increasing total load Q on the pile was analysed. Q_p was seen to vary approximately linearly with Q while its inclination δ_3 changed roughly linearly from about $-\phi_t$ to $+\phi_t$ as Q/Q_u changed from 0 to 1. The value of Q_p under axial load is given by the first part of Eq. (1). In case of inclined or eccentric loads, its value can be readily obtained from the inclination factor i_γ (Meyerhof and Hanna, 1978). The estimated values of Q_p are safe when compared to the observed values (Fig. 9(*a*)).

Pile in Clay

Later Pressures

When a rigid pile embedded in clay is subjected to an inclined or eccentric load Q_u, the lateral soil pressure at failure is approximately represented by the roughly rectangular distribution (Fig. 11(*b*)). Apart from the earth thrusts P_1 and P_2, soil adhesions C_1 and C_2 are mobilized

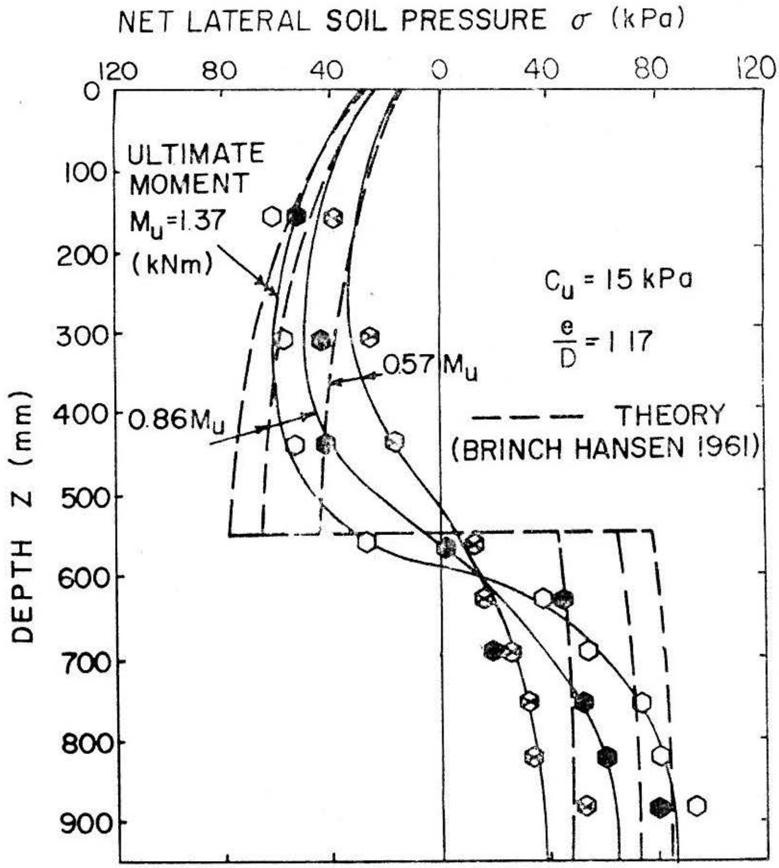

FIGURE 8(*b*) Variation of Net Lateral Soil Pressure with Applied Load in Case of Pile in Clay under Eccentric Loads for e/D=1.17.

along the shaft with the tip resistance Q_p inclined at δ_3. Due to the jacking of pile into the clay, considerable lateral soil pressures are developed on the pile shaft. The theoretical estimate of this soil pressure in the present tests is $\sigma_i = 3.3\ c_u$. This estimate is obtained for the value of $E_s/C_u = 35$ (Kirby and Esrig, 1980), where E_s = Secant modulus and C_u = Undrained strength of the soil. Assuming a parabolic vriation of σ_i from zero at the surface to its maximum value at a depth of 10 B, reasonable agreement is noticed between the observed and estimated values of σ_i (Fig. 4).

From the observed lateral soil pressures along the shaft of a pile subjected to a horizontal load or pure moment, the distribution of σ suggested by Brinch Hansen (1961) was seen to hold promise. The theoretical estimate of σ values at failure is based on the earth pressure coefficients (Meyerhof, 1972), allowing 80 percent of the values proposed for rough piles to account for the smoothness of the shaft in the present tests. Once

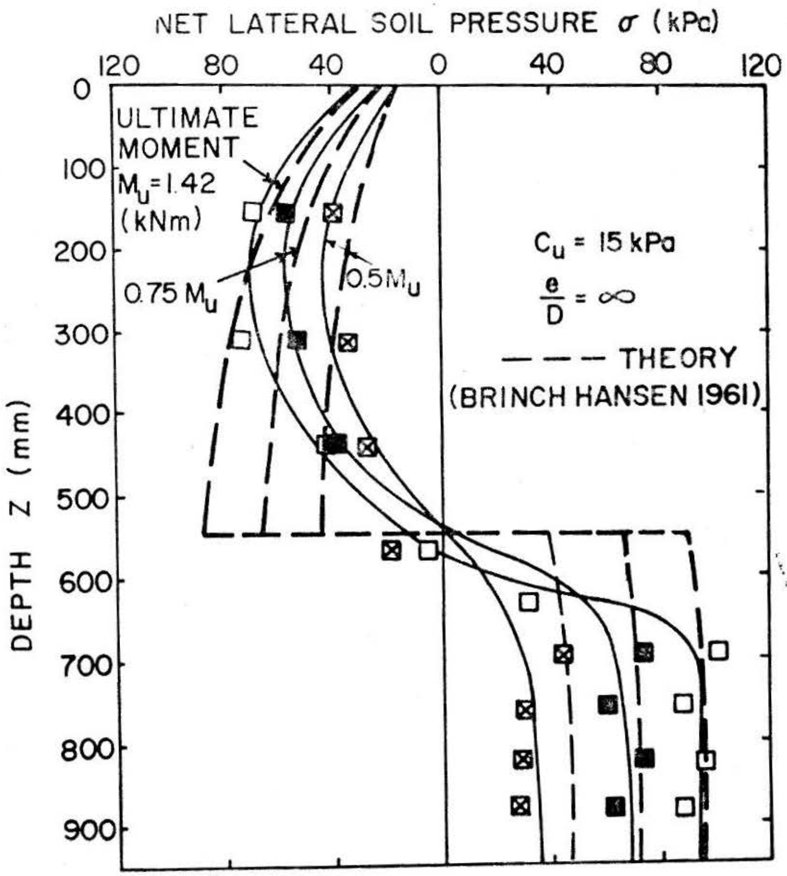

FIGURE 8(c) **Variation of Net Lateral Soil Pressure with Applied Load in Case of Pile in Clay under Eccentric Loads for $e/d = \infty$.**

again, the effect of increasing load on the σ value is analysed by examining the measured σ values under increasing load or moment at salient points along the shaft. Although the σ is not precisely linearly varying with the load Q, a linear relation may safely be assumed between Q and σ (Sastry and Meyerhof, 1986). Thus, the theoretical pressure distribution σ at ultimate horizontal load or pure moment may be linearly reduced proportionate to the load or moment in order to arrive at the σ distribution under corresponding working loads up to about $F \leqslant 2$. For intermediate inclinations and eccentricities of the load, the lateral soil pressure at pile base σ_b value can be estimated from Fig. 9(b) and 10(b) respectively.

When compared with the measured values, the estimated values of σ are seen to be on the safe side (Figs. 7 and 8).

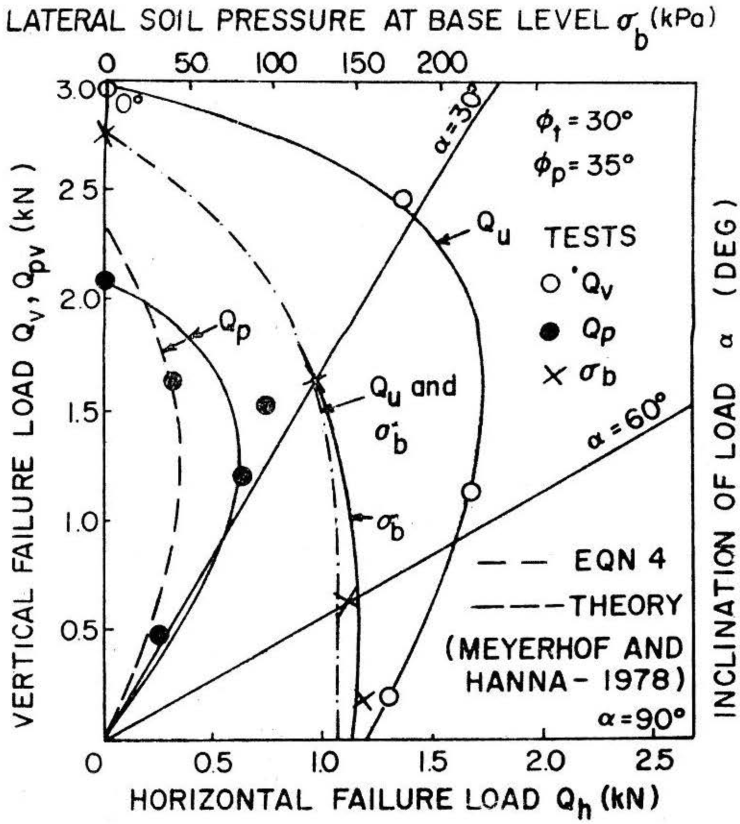

FIGURE 9(*a*) Polar Bearing Capacity Diagram for Pile under Inclined Loads in Sand.

Bearing Capacity

The pile capacity under axial load is obtained theoretically from:

$$Q = 9\ c_u\ A_p + r c_u\ A_s \qquad \ldots (10)$$

Where r = adhesion factor and other symbols as before. The Q_{90} under a horizontal load can be obtained by considering the equilibrium of the smooth pile (Fig. 11(*b*)) so that

$$Q_{90} = 0.4\ K_c \left(1 - 5\frac{B}{D}\right) c_u\ BD \qquad \ldots (11)$$

$$= 0.4\ \sigma_b \left(1 - 5\frac{B}{D}\right) B\ .\ D \qquad \ldots (12)$$

Since $\sigma_b = K_c\ .\ c_u$

Where K_c = Earth pressure coefficient for smooth pile (Meyerhof *et al.*, 1981). For intermediate inclinations the pile capacity Q_u and the net

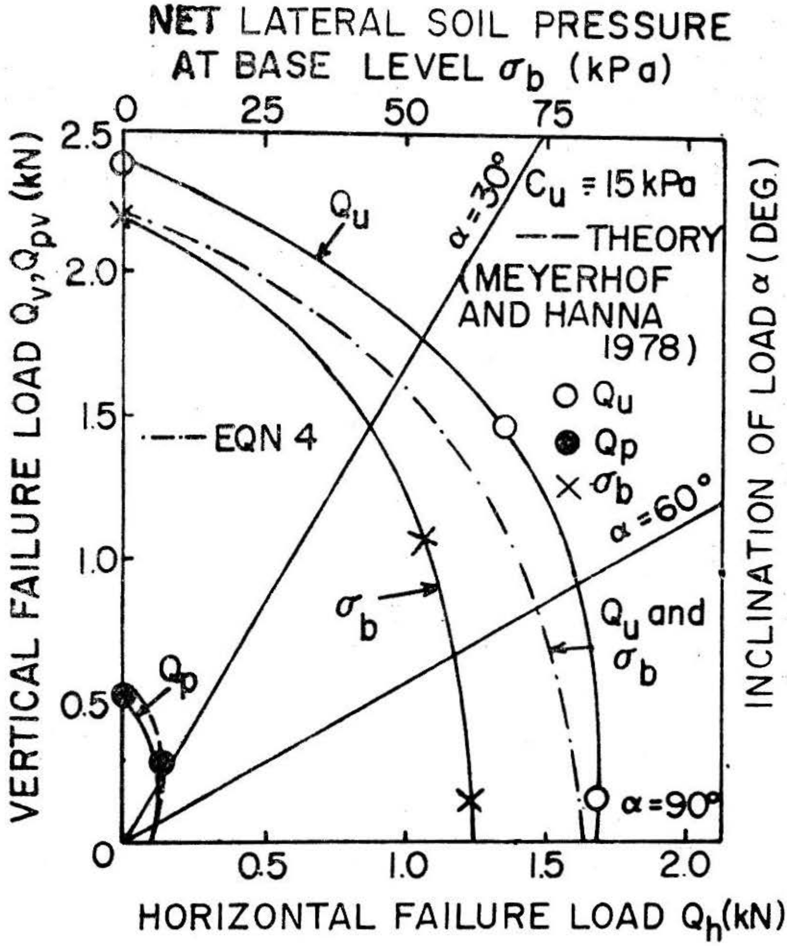

FIGURE 9(*b*) Polar Bearing Capacity Diagram for Pile under Inclined Loads for Clay.

lateral pressure at the pile base σ_b can be approximately estimated using Eq. (4). The estimated values of σ_b and Q_u are on the safe side compared to the measured values (Fig. 7 and 9(*b*)).

The maximum pure moment at failure M_o is once again obtained from the equilibrium considerations of the smooth pile.

Thus,

$$M_o = 0.25 \ K_c \ c_u \, B \, D^2 \left(1 - 4 \frac{B}{D} \right) \qquad \dots (13)$$

$$= 0.25 \, \sigma_b \, B \, D^2 \left(1 - 4 \frac{B}{D} \right) \text{ with symbols as before} \qquad \dots (14)$$

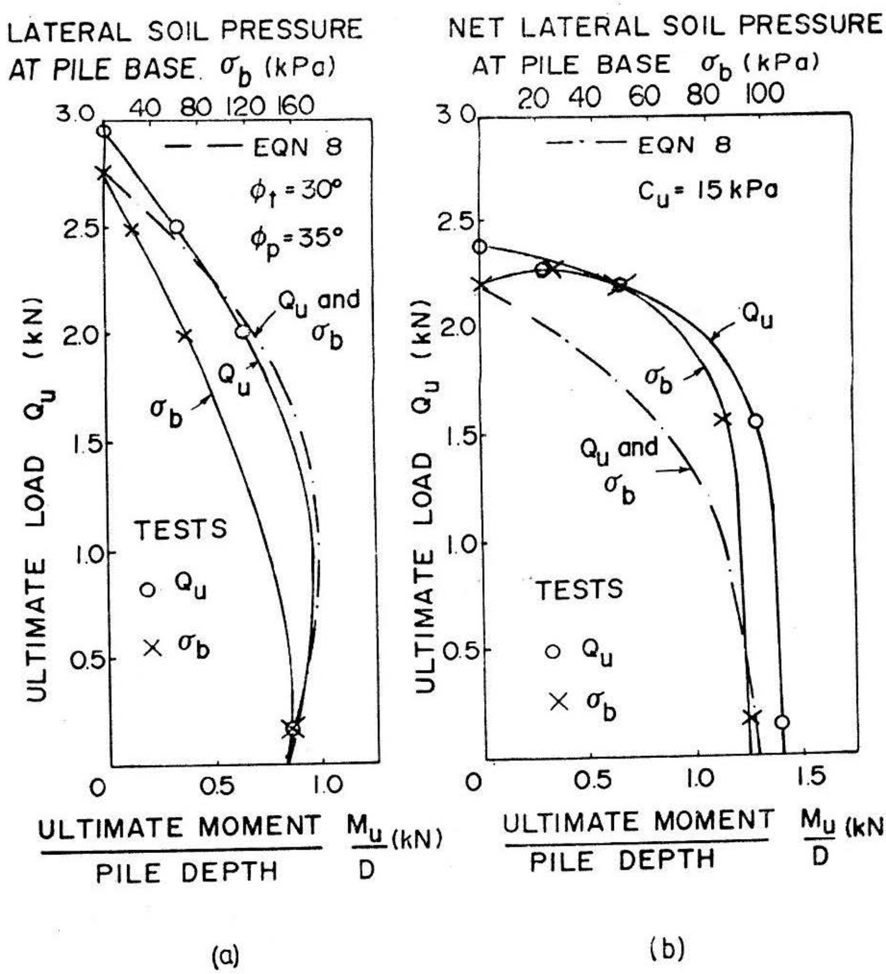

FIGURE 10 **Interaction Diagram between Ultimate Load and Moment for Pile under Eccentric Loads (*a*) Sand (*b*) Clay.**

M_0 can also be expressed in terms of Q_{90} using Eq. (7), where $m = 0.6\left(1+\frac{B}{D}\right)$. Thus, the pile capacity under intermediate eccentricity can be estimated from Eq. (8).

As explained earlier, the curve for Q_u in Fig. 10(*b*) also estimates the σ_b at failure for a given eccentricity of the load. The estimated values of Q_u and σ_b are on the side of safety compared to the observed values. The value of Q_p under axial load is given by the first part of Eq. (10) and in case of inclined or eccentric loads its value can be readily obtained from the inclination factor i_c (Meyerhof and Hanna, 1978). Close agreement is seen between estimated and observed values of Q_p (Fig. 9(*b*)).

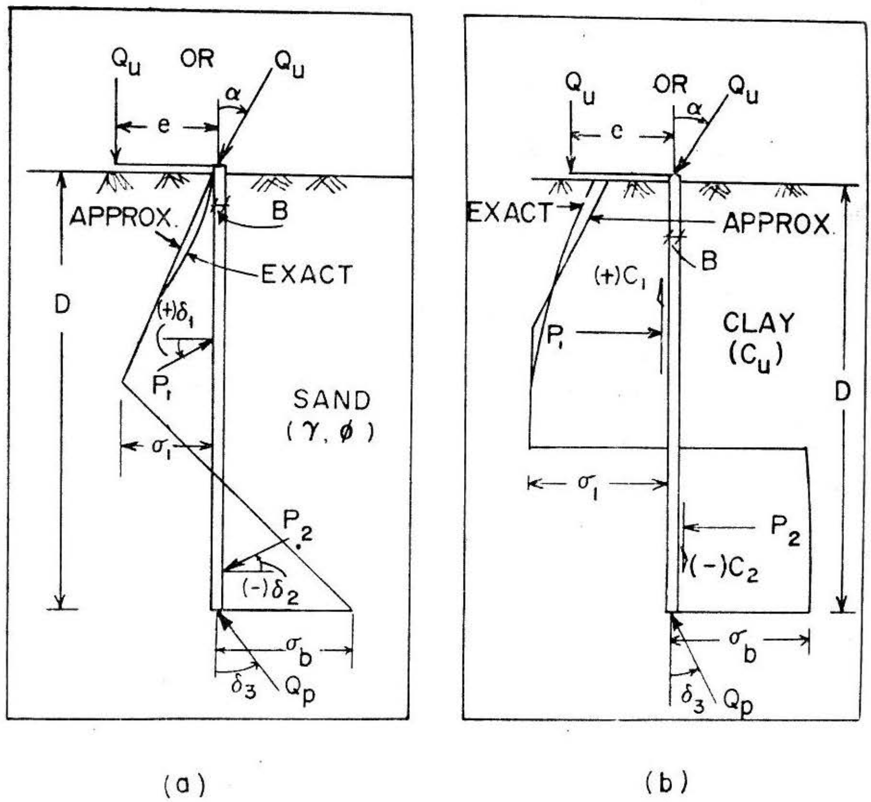

FIGURE 11 Forces at Failure of Pile under Inclined or Eccentric Load (*a*) Sand (*b*) Clay.

Conclusions

Analysis of observed lateral soil pressures along the pile shaft and the point resistance of piles jacked into homogeneous sand and clay, subjected to central inclined and vertical eccentric loads have lead to an understanding of their variation with the applied load. The ultimate pile capacity under horizontal load pure moment can be estimated based on the observed lateral soil pressure variation. For intermediate inclination or eccentricity of the load, the pile capacity and the lateral soil pressure at the base of pile can be roughly estimated from the inclination or eccentricity factors suggested earlier.

Triangular distribution of lateral pressures suggested by Terzaghi (1943) seems to be valid for piles in sand while the roughly rectangular distribution suggested by Brinch and Hansen (1961) seems to hold promise for piles in clay. The magnitude of the pressure can be computed from the suggested earth pressure coefficients (Meyerhof *et al.* 1981). The lateral soil pressures under working loads at intermediate inclinations and eccentricities can approximately be predicted. It is interesting to note that considerable lateral pressures are induced on displacement piles jacked into clay.

Even though the present tests have indicated the propriety of the proposed methods for estimating the lateral soil pressures, pile capacity and toe resistance, it is strongly felt that adequate field testing should be undertaken to verify the predictions.

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Notation

A_p = area of pile point

A_s = area of pile shaft

B = pile width

C_1, C_2 = soil adhesion on pile shaft

C_u = uniformity coefficient

c_u = undrained shear

D = depth

D_{10} = effective size

D_r = relative density

E_s = secant modulus of soil

e = eccentricity of load

F = factor of safety

i_c, i_γ = inclination factors for pile point resistance in clay and sand respectively

K_b = pile shape factor for lateral earth pressure in sand

K_c = pile shape factor for lateral earth pressure in clay

K_o = coefficient of earth pressure at rest

K_s = coefficient of earth pressure on shaft

M_o = pile capacity under pure moment

M_u = moment on the pile due to eccentric load at failure

m = coefficient

N_q = bearing capacity factor

P_1, P_2 = earth thrusts

Q = total load on pile

Q_o = pile capacity under axial load

Q_{90} = pile capacity under horizontal load

Q_p = point resistance of pile

Q_u = total load on pile at failure

r = adhesion factor

S = degree of saturation

W = liquid limit

W_p = plastic limit

W_n = water content

α = inclination of load with vertical

γ = unit weight of soil

$\delta, \delta_1, \delta_2$ = angles of skin friction

δ_3 = inclination of Qp with vertical

σ = net lateral soil pressure at any depth

σ_a = lateral soil pressure decrease due to load on pile in clay

σ_b = net lateral soil pressure at pile base at failure

σ_i = lateral soil pressure due to pile installation

σ_p = lateral soil pressure increase due to load on pile in clay

σ_u = net lateral soil pressure at any depth at failure

ϕ_p = angle of internal friction in plane strain

ϕ_t = angle of internal friction from triaxial test

η = porosity

θ = equivalent inclination of load