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Behaviour of Rigid Piles in Clay under Lateral Loading

by

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Introduction

In certain cases, foundations are subjected to large lateral forces caused by wind, wave action and lateral earth pressures. Structures such as transmission line towers, bridges, tall buildings and offshore platforms are usually founded on piles and safety of these structures depends on the ability of the supporting piles to resist large lateral forces. Batter piles are considered to be more effective in resisting horizontal loads. However, if there is large variation in the horizontal load with time, the batter piles are likely to be severely stressed in bending and for this type of loading conditions, it is found to be convenient to adopt vertical piles to resist lateral loads.

Extensive theoretical and experimental studies were carried out by several investigators on laterally loaded piles in clays to determine their ultimate resistance and their deflection under working loads. Matlock and Reese (1960) developed design curves for predicting the deflection in laterally loaded piles using the subgrade reaction approach. This method is applicable only if the deflection of piles is within the range of elastic compression of the soil. Hansen (1961) proposed a method to predict ultimate lateral load capacity of rigid piles based on earth pressure theory. Broms (1965) assuming the soil pressure distribution at failure presented a method

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for estimating ultimate lateral capacity for both rigid and flexible piles. Poulos (1971) used the elastic theory to analyse the displacement, rotation and moment in a pile subjected to horizontal load and moment. Significant differences exist between these solutions and those obtained from subgrade reaction theory. The effects of local yielding of the soil on the behaviour of pile are also examined. Meyerhof and Ranjan (1972, 1973) described methods for finding out the bearing capacity of rigid piles under inclined loads in sand for vertical and batter piles and for pile groups based on the tests conducted on model piles in the laboratory. Meyerhof (1981) proposed a semi-empirical interaction relationships between the axial and lateral components of oblique ultimate load for single rigid vertical and batter piles in clays. Meyerhof et al. (1981) extended the analysis for laterally loaded walls in layered soils to rigid vertical piles by determining the shape factors for the ultimate lateral resistance of piles in sand and clay. The lateral deflections of rigid piles in layered soils have also been related to those of corresponding by using shape factors. The lateral soil pressures at failure and the ultimate bearing capacity of single rigid piles subjected to vertical eccentric and central inclined loads were presented by Chari and Meyerhof (1983) for piles in sand and Meyerhof and Sastry (1985) for piles in clay. The ultimate lateral capacities, the lateral soil pressure and displacements at failure of the rigid piles subjected to vertical eccentric and central inclined loads were presented by Sastry and Meyerhof (1986). In most of these investigations, model tests were conducted to develop approximate design procedure for laterally loaded piles and the lateral load was applied either at ground surface or at some height above the ground level. In these tests, the application of lateral load at some height above the ground level was considered to be an eccentric loading. In many practical cases such situations of eccentric loading can arise. In the present investigation, the effect of load eccentricity on the lateral load deflection behaviour of vertical piles has been studied by conducting load tests at different eccentricities. A method has been proposed to estimate the load-deflection behaviour at any eccentricity based on the results obtained from tests with a known eccentricity. Tests have been carried out on piles embedded in clayey soil bed prepared at different moisture contents. A relationship has been developed to estimate the ultimate lateral capacity of the piles in terms of load eccentricity. A limited rectangular hyperbolic formulations are attempted to predict the load-deformation behaviour.

The proposed relationships have also been verified by using laboratory and field model test results reported by some investigators.

Experimental work

Model piles, soil used and tests conducted : The tests were conducted on model piles made of mild steel and aluminium pipes. All the piles were of a total length (L) 500 mm with a depth of embedment (L_e) of 300 mm. The diameter of the piles was varied in order to get a variation in the embedment ratio, L_e/D from 9 to 25. In the first series of tests, pipe piles of mild steel were used and a number of tests were conducted with piles embedded in clayey soil bed prepared with variations in the moisture content. Lateral load tests were conducted at consistency index (I_c) of 0.34, 0.42, 0.62 and 0.72. Consistency Index (I_c) is the ratio of difference between liquid limit and moisture content of the soil to the plasticity index. This range of consistency index was chosen to represent the in-situ conditions suitable to marine clay deposits of east coast of India. The height of load application was varied from 50 mm to 150 mm (values of e/L ranging from 0.17 to 0.50). At an $I_c = 0.34$, the tests were conducted on piles of diameters, 13.5 mm, 18.0 mm, 21.5 mm, 27.0 mm and 33.3 mm. For the values of $I_c = 0.42$, 0.62 and 0.72, the tests were conducted on piles of diameters, 13.5 mm, 18 mm and 21.5 mm. In order to have some tests at different values of rigidity, a second series of tests were conducted on two numbers of aluminium piles. The details of all these piles tested are given in Table 1. The soil used was soft marine clay obtained from a coastal deposit along the east coast of India. Its liquid limit is 82% and plastic limit is 82% and plastic limit is 32%. The tests were conducted in a circular steel test tank of 350 mm diameter and 500 mm height. The schematic diagram of the test set up used in this investigation is shown in Fig. 1.

Testing procedure

Holding the pile in position the soil was carefully hand packed into the test tank in layers of 50 mm thickness, in position. Each layer was pressed with a wooden template so as to remove entrapped air. In field situations, basically there are two types of piles used and these are driven piles and bored cast in-situ piles. In this testing programme, the test bed was prepared with the pile kept in position and the behaviour of this pile is considered to be fairly close to bored piles. However, there could be some minor differences in the surface characteristics and it can be assumed that this model pile is smoother than the field bored pile. The moisture contents used were sufficiently high and as such there was no difficulty in ensuring a homogeneous and uniform fill. A number of samples were taken out from the test beds and pore water pressure measurements were carried out. The Skempton's pore pressure parameter, B, was found to be around 0.99 indicating that the soil bed prepared was nearly fully saturated. The strength

TABLE 1
Details of Model Piles Tested

Sl. No.	Pile Material	Pile Diameter D (mm)		Flexural Rigidity E_pI_p ($N.mm^2$)	Relative Stiffness Factor, K_r			
		Outer Dia	Inner Dia		$I_c = 0.34$	$I_c = 0.42$	$I_c = 0.62$	$I_c = 0.72$
1	Mild Steel	13.5	9.2	2.622×10^8	2.02×10^{-1}	1.62×10^{-1}	9.52×10^{-2}	6.35×10^{-2}
2	Mild Steel	18.0	13.0	7.690×10^8	5.93×10^{-1}	4.75×10^{-1}	2.79×10^{-1}	1.86×10^{-1}
3	Mild Steel	21.5	17.8	1.140×10^9	8.80×10^{-1}	7.04×10^{-1}	4.14×10^{-1}	2.76×10^{-1}
4	Mild Steel	27.0	22.0	2.991×10^9	2.31×10^0	1.85×10^0	1.09×10^0	1.96×10^0
5	Mild Steel	33.3	24.7	8.628×10^9	6.66×10^0	5.33×10^0	3.13×10^0	2.09×10^0
6	Aluminium	12.2	10.0	4.324×10^7	3.34×10^{-2}	2.67×10^{-2}	1.57×10^{-2}	1.05×10^{-2}
7	Aluminium	18.4	16.3	1.567×10^8	1.21×10^{-1}	9.67×10^{-2}	5.69×10^{-2}	3.79×10^{-2}

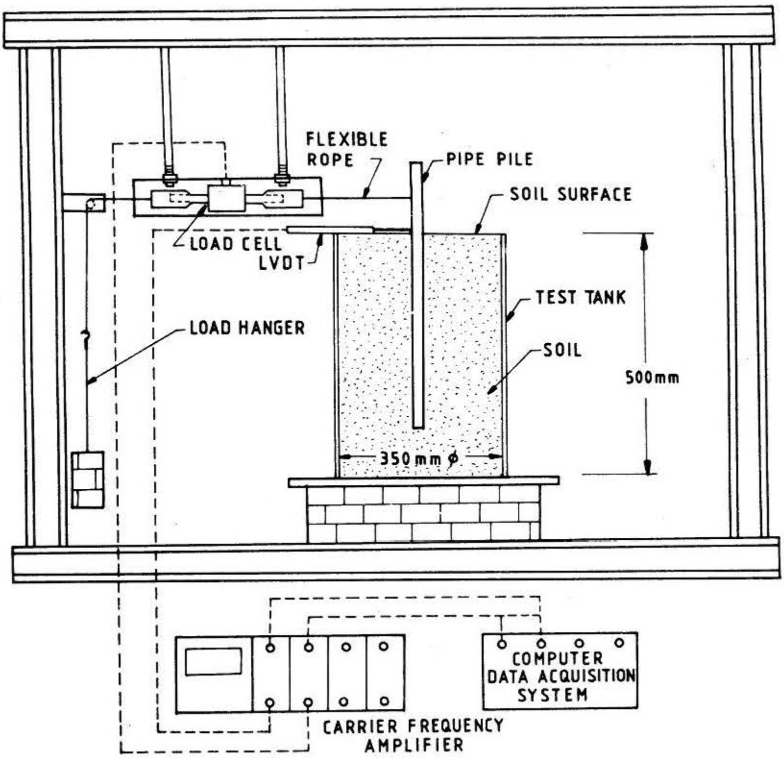


FIGURE 1 : Experimental Setup

of the soil was measured by conducting laboratory in-situ vane shear tests. The shear strengths measured are 3.4, 4.0, 7.2 and 10.0 kPa for I_c values of 0.34, 0.42, 0.62 and 0.72 respectively. The load tests were conducted and in these the static load was applied by placing weights in increments and at each load increment, it was waited till a rate of deflection approached a value of 0.02 mm/hour. The load applied was measured accurately with a load cell of 250 N capacity and the lateral displacement of pile at ground level was measured with LVDT of 20 mm travel. The outputs from the load cell and LVDT were measured using a carrier frequency amplifier and recorded using an apple IIe computer data acquisition system.

Results and Discussion

Typical load-deflection curves obtained from the model tests conducted are shown in Fig. 2. From the literature, it could be seen that there are many guidelines suggested to arrive at the ultimate lateral capacity based on

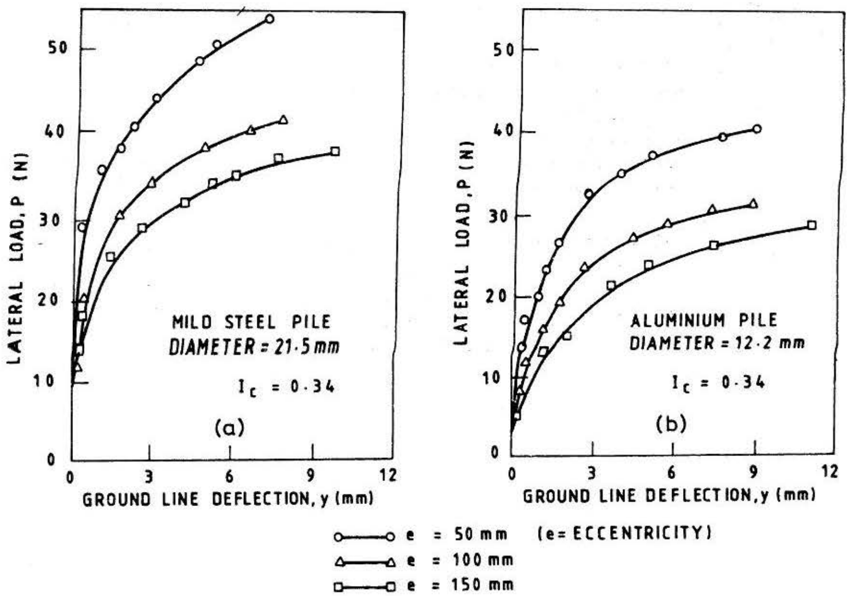


FIGURE 2 : Typical Load-Deflection

the load-deflection curves obtained from load tests conducted. From the general criterion used, the ultimate lateral capacity can be taken as (i) the load that causes certain deflection generally expressed in terms of percentage of pile diameter, (ii) the load at which the rate of deflection continues undiminished without further increase in the load and (iii) the load corresponding to the point where the portion of the load deflection curve becomes straight or substantially straight (Whitaker 1957). There is a practical difficulty in using the second criterion as the pile experience large deflections before satisfying such high rate of deflection and the load tests are usually not carried out to that stage. In the third method also, there are certain difficulties in the location of the point at which the load deflection curve becomes straight line (Whitaker, 1970). The load deflection curve obtained from present tests (Fig. 2) also confirm this aspect. In most of the cases, the criterion for ultimate load is based on the allowable deflection. According to Briaud (1983), permissible deflections are generally in order of 10% of the pile diameter. According to Broms (1964), the ultimate lateral resistance is generally reached when the deflection at the ground surface reaches a value approximately equal to 20% of diameter or width of pile. The lateral load capacity for piles in this study was defined based on deflection and the load corresponding to a deflection equal to 20% of pile diameter (as measured at ground level) is taken as the lateral capacity. From the load deflection point of view, it appears that the pile can be loaded beyond deflection of 20% pile diameter. However, from the existing methods it can be seen that the deflection of 20% is on the higher side. In view of this, due to practical considerations pile cannot be loaded beyond this deflection. From the load-deflection curves it can be observed that at this ultimate load level, any further increase in the load can result in an enormous increase in the deflections.

The details of the piles tested along with their relative stiffness factors (K_r) and flexural rigidity values are presented in Table 1. The relatives stiffness factor (K_r) is defined as (Banerjee and Davies, 1978; Poulos and Davis 1980)

$$K_r = \frac{E_p\, I_p}{E_s\, L^4} \tag{1}$$

Where E_s = average horizontal soil modulus

E_p = modulus of elasticity of pile material

I_p = moment of inertia of pile cross section

L = length of the pile

There are several experimental methods available for determining the soil modulus to be used in the calculation of K_r. The method suggested by Poulos and Davis (1980) is used in this investigation to estimate the E_s values and this is based on the results of static load tests. The advantages of this method are detailed by Mallikarjuna Rao (1992). In the case of rigid piles, it is known that the lateral load-deflection behaviour is independent of relative pile soil stiffness as these piles fail by rotation (Broms, 1964; Tomilson, 1986). In that case, the load deflection behaviour should be the same, irrespective of the type of pile material, for the same diameter and length of the pile. In Figs. 3 and 4, the load deflection curves obtained from aluminium piles (18.4 mm dia) and mild steel piles (18 mm dia) are presented and these confirm that all the points lie on the same curve indicating that the pile material has little influence on the load deflection behaviour. Further, according to Meyerhof (1981), the ultimate lateral resistance (P_u) of a free headed laterally loaded rigid pile in clay is approximately given by

$$P_u = 3\, C_u\, D\, L \tag{2}$$

where D and L are the pile diameter and length respectively and C_u is the average undrained shear strength of the soil. For piles subjected to a combination of load and moment. Budhu and Davies (1988) proposed the following equation for lateral capacity.

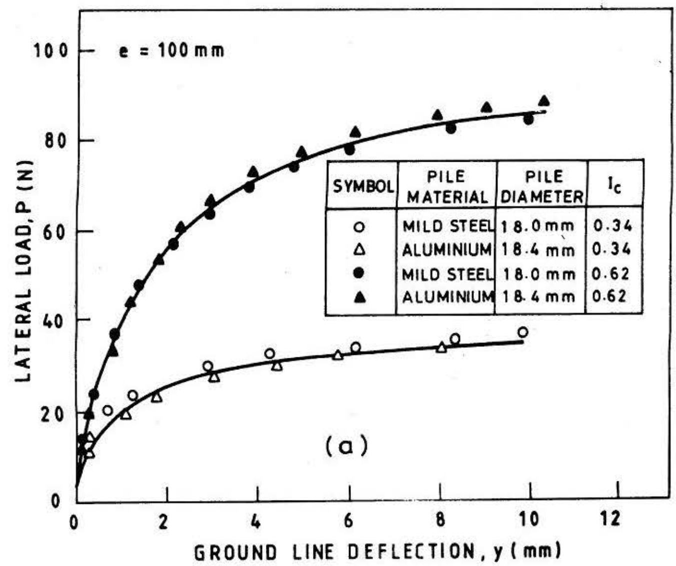


FIGURE 3 : Comparision of Load-Deflection Curves of Mild Steel and Aluminium Piles (e = 100 mm)

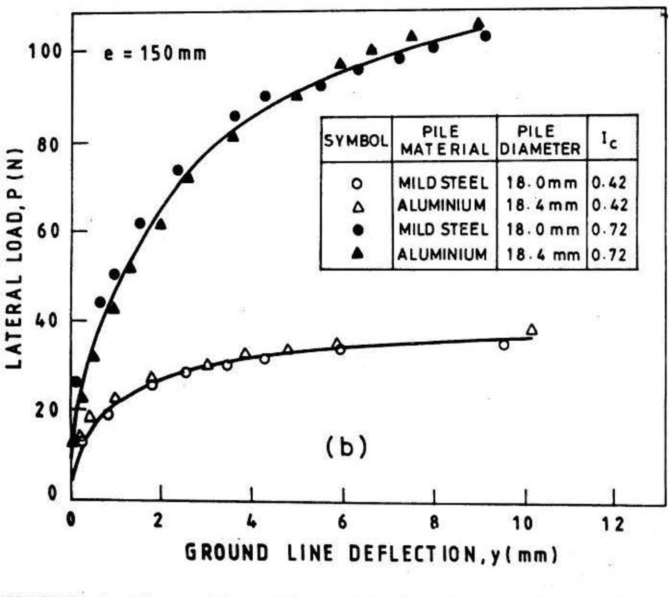


FIGURE 4 : Comparision of Load-Deflection Curves of Mild Steel and Aluminium Piles (e = 150 mm)

$$\frac{P_u}{C_u\, D\, L^2} = \frac{1.2}{g + 0.88} \qquad \text{when } \propto > g > 2/3 \tag{3}$$

where g = eccentricity ratio = e/L

Based on Eqns. (2) and (3), it can be concluded that the lateral capacity of rigid free headed piles is linearly proportional to the diameter of the pile for a given pile length. In Fig. 5. the variation in the lateral capacity with the pile diameter is presented for the tests conducted on the mild steel piles embedded in clay bed prepared at different consistencies. From this figure, it can be observed that at any consistency and at any given eccentricity, the relationship between lateral capacity and pile diameter is a straight line passing through the origin. Consistency index significantly influences the capacity. The lateral capacities of all mild steel and aluminium piles are plotted against pile diameter in Fig. 6. In all these cases, the capacities of both aluminium and mild steel piles are found to fall on the same straight line although there is a considerable variation in the modulii of pile material.

Estimation of ultimate lateral capacity

In case of rigid piles, it is known that the lateral load deflection behaviour is controlled by Diameter (D) and length (L) of the pile, load eccentricity (e) and undrained shear strength (Cu) of the soil. As such the lateral capacity (Pu) can be considered as a function given below.

$$P_u = f(C_u, D, L, e) \tag{4}$$

It is common practice to analyze the results in terms of non *dimensional factors.* For the Eqn. 4, two non dimensional factors can be suggested and these are (i) load capacity factor given by $P_u / C_u DL$ and (ii) eccentricity factor given by e/L. The results of the present lateral load tests are plotted in the form of $P_u / C_u DL$ versus e/L curves and are shown in Fig. 7. From this plot it can be observed that the values of K is fairly a constant at any given value of e/L. Such type of plots are found to be useful as a single curve can take care of wide variations in the strength of the soil and the sizes of the piles. This nondimensional capacity factor $P_u / C_u DL$ is found to decrease with e/L as expected. From this curve, an attempt has been made to suggest a general expression for lateral capacity (P_u) in the following form

$$P_u = K\, C_u\, D\, L \tag{5}$$

where K can be considered as a coefficient whose value depends on

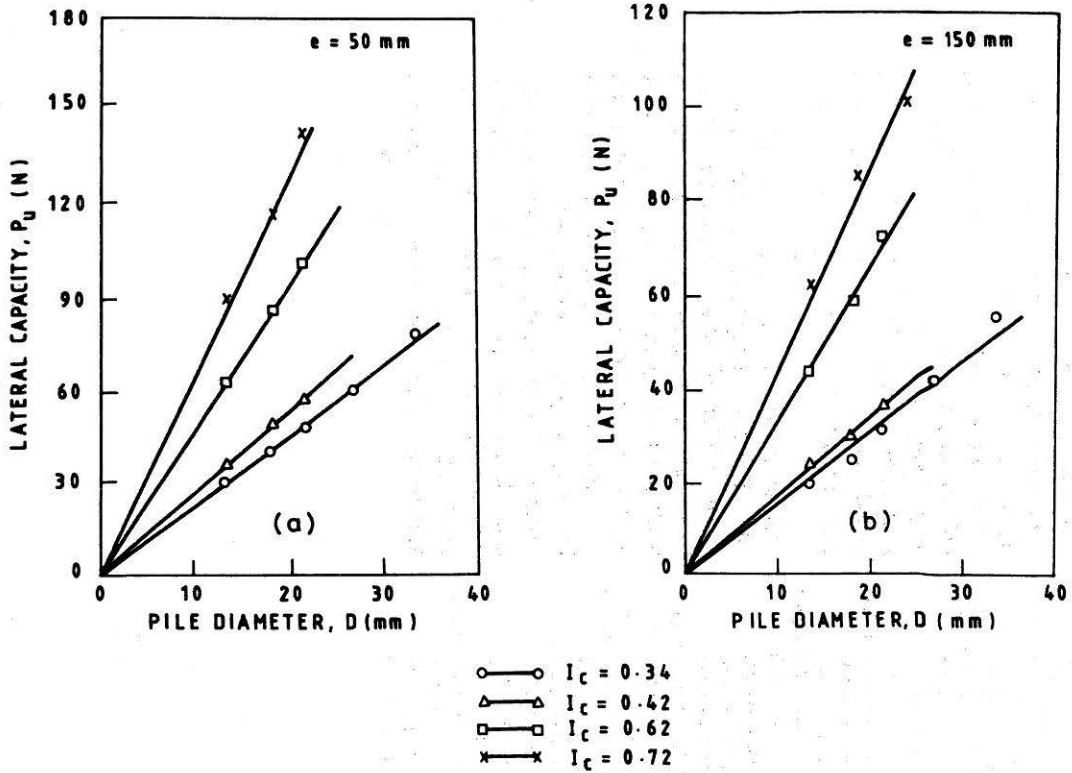


FIGURE 5 : Variation of Lateral Capacity with Pile Diameter (Mild Steel Piles)

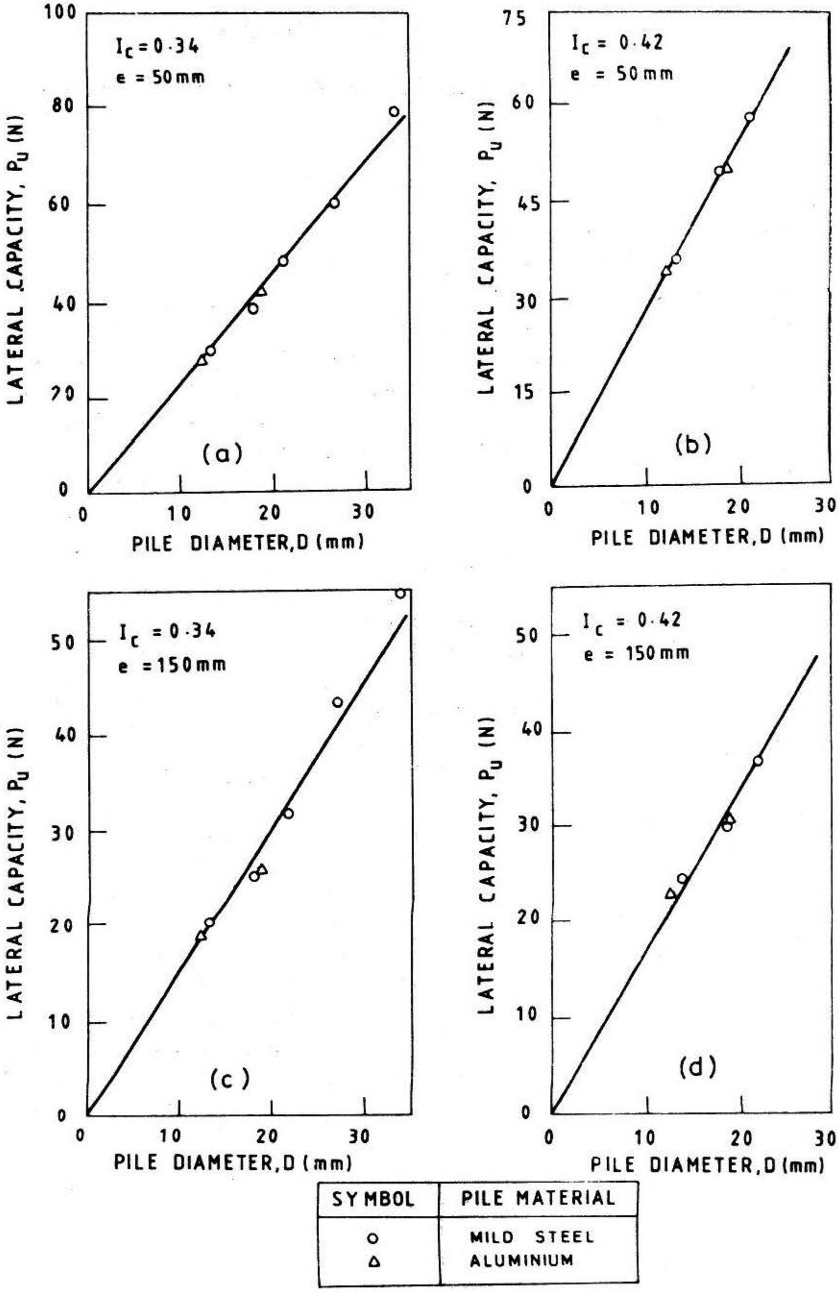


FIGURE 6 : Variation of Lateral Capacity with Pile Diameter

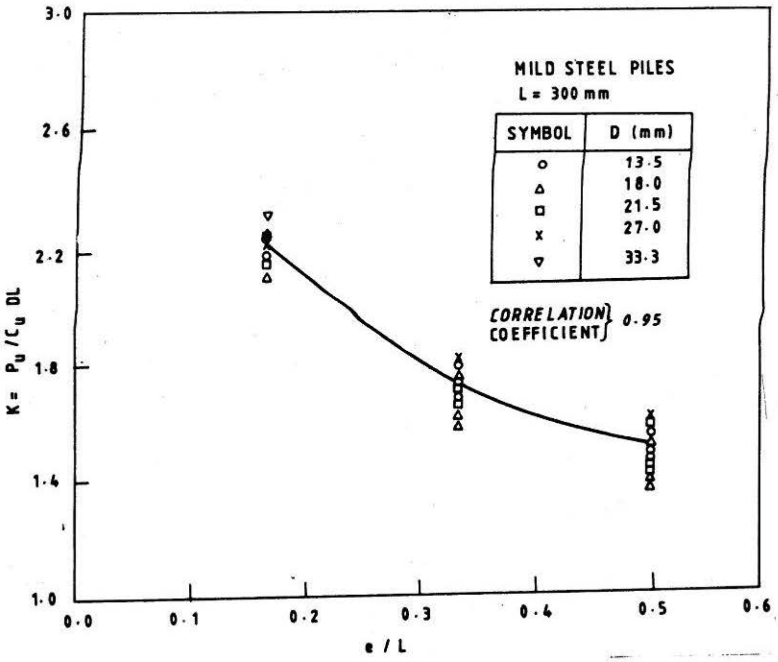


FIGURE 7 : $\frac{P_u}{C_u\,D\,L}$ Versus $\frac{e}{L}$ (Mild Steel Piles)

eccentricity ratio, e/L. From Fig. 7 it is quite obvious that there exists a relationship between the values of K (i.e. $P_u / C_u DL$) and e/L which is independent of consistency index, I_c. Non linear regression analysis yielded the following expression for K.

$$K = 2.44\,(0.32)^{e/L} \tag{6}$$

The correlation coefficient works out to be 0.95. The above equation can be used to arrive at K values for piles confirming to rigid conditions. Hence, the value of ultimate lateral capacity can be given by

$$P_u = 2.44\,(0.32)^{e/L}\,C_u\,D\,L \tag{7}$$

Verification with reported results

The reliability of any empirical equation or formulation can be established by comparing the estimated values with those of observed one for the data or results which were not used in developing the empirical equation. The available published data obtained from the lateral tests conducted on laboratory model piles and piles installed in the field are used to verify the formulations suggested. Meyerhof (1981) carried out load tests on laboratory model steel piles of 13 mm diameter and for L/D ratios of 10 and 20. The tests were conducted in a soil bed made of soft soil of medium plasticity (LL = 43%, PI = 21%) and the average undrained shear strength was reported to be about 24 kPa. The observed lateral capacities were 106 N and 225 N for L/D ratios of 10 and 20 respectively. The predicted capacities using Eqn. 7 are 99.0 N and 197.9 N respectively. The results of Meyerhof and Yalcin (1984) were also obtained from the model tests using steel piles of 12.5 mm diameter and 190 mm length embedded in saturated clay of an average undrained shear strength of 24 kPa. The horizontal failure load was reported to be 128.0 N as against the predicted value of 139.1 N

Baguelin et al. (1972), reported the results obtained from the tests carried out on prototype piles installed in field. A steel pile of 950 x 950 mm and of 4.5 mm depth was jacked into cohesive soil of an average undrained shear strength of 26 kPa. From the tests conducted with horizontal load applied at a height of 2000 mm above ground level, the observed failure load was 145 kN and the value predicted is 158.0 kN. Another set of results reported by Kerisel and Adam (1967) are also used in these comparisons. These results were obtained from field tests on open steel piles of 950 x 950 mm size jacked in to the clay to a depth of 3.5 m on a site underlain by stiff clay with an average undrained shear strength of 75 kPa. From the test conducted with the horizontal load applied at a height of 400 mm above the ground level, the observed failure load was 450 kN as against the predicted value of 534.2 kN. All these comparisons are summarized in Table 2. The predictions attempted for the reported laboratory test results and for the field test results appear to be reasonably good and this brings out that the k values obtained from the laboratory tests can reasonably be extended to observe the behaviour of the field piles.

Influence of height of load application

The load tests have been conducted on all the piles with load applied at eccentricities of 50 mm, 100 mm and 150 mm (e/L = 0.17, 0.34 and 0.50). From the load-deflection curves presented in Fig. 2, it could be seen that with increase in eccentricity, there is a significant reduction in the capacity of the pile under lateral load and this is quite obvious. Typical curves showing variations in the lateral capacity with eccentricity are presented in Fig. 8. From these cures it can be observed that there is a significant reduction in the lateral load carrying capacity of the pile with increase in the eccentricity. With an increase in the load eccentricity, there is an increase in values of deflection. In order to arrive at plots independent of eccentricity,

TABLE 2

Comparison of Observed and Estimated Lateral Capacities : Reported Results

Sl. No.	Pile Diameter (mm)	Pile Length (mm)	Load Eccentricity (mm)	Undrained Shear Strength C_u (kPa)	Lateral Capacity, P_u		Ratio P_{uo}/P_{up}	Results Used From
					Observed, P_{uo}	Estimated as per Eqn. 7, P_{up}		
1	13.0	260	0	24.0	225.0N	197.9N	1.14	Meyerhof (1981)
2	13.0	130	0	24.0	106.0N	99.0N	1.07	Meyerhof (1981)
3	12.5	190	0	24.0	128.0N	139.1N	0.92	Meyerhof and Yalcin (1984)
4	950.0	4400	2000	26.0	145.0kN	158.9kN	0.92	Baguelin et al. (1972)
5	950.0	3500	400	75.0	450.0kN	534.2kN	0.84	Kerisel and Adam (1967)

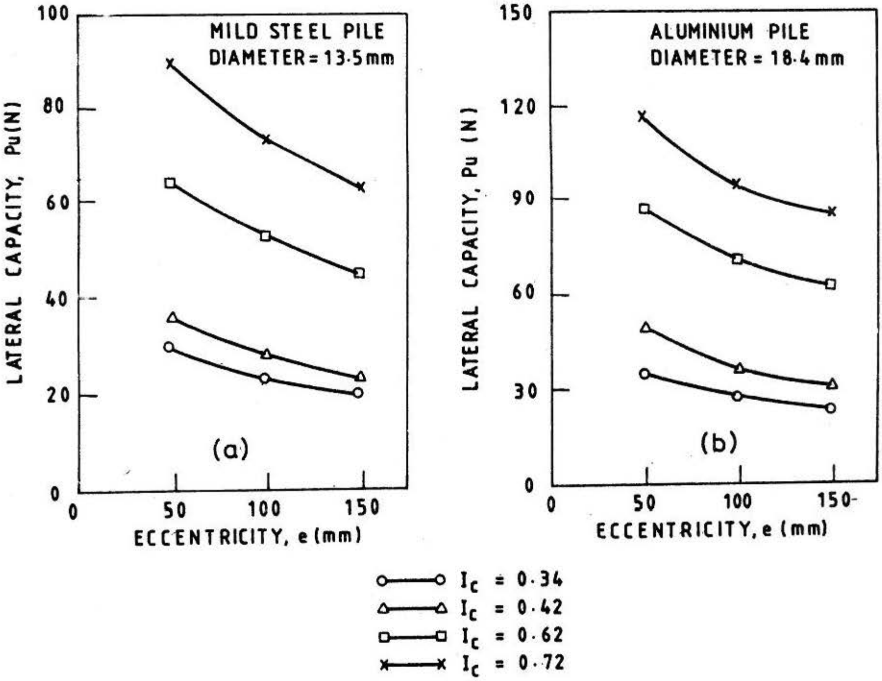
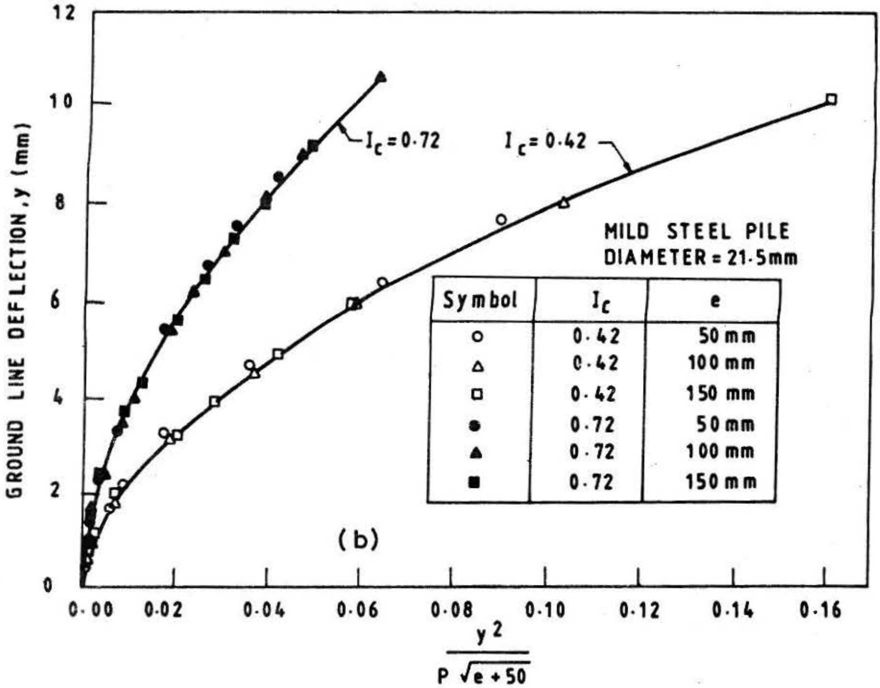


FIGURE 8 : Variation of Lateral Capacity with Eccentricity

it is suggested that the ratio $Y^2/P\sqrt{e+50}$ can be plotted against deflection, Y, as shown in Fig. 9. In this ratio, $Y^2/P\sqrt{e+50}$, Y = deflection in mm, P = lateral load corresponding to deflection Y is in N, and e = eccentricity in mm. From these plots it is quite clear that at any given consistency, the value of $Y^2/P\sqrt{e+50}$ is a constant even for different values of eccentricity, e. All these results suggest that such relationships between Y and $Y^2/P\sqrt{e+50}$ can be conveniently made and used to predict the lateral load – deflection behaviour at any eccentricity from the observed behaviour at any one particular eccentricity. This can also be verified using published data.

A few investigators carried out load tests with load eccentricity on rigid piles. Druery and Ferguson (1969) carried out a series of lateral load tests on free headed model brass piles of 6.35 mm diameter in Kaolin to obtain load deflection curves to failure. Load was applied at eccentricities of 19.05 mm, 25.4 mm and 33.78 mm. The observed load deflection curves are shown in Figs. 10 and 11. These results also confirm such relationships between Y and e. From this data also, there is no scatter to be observed.



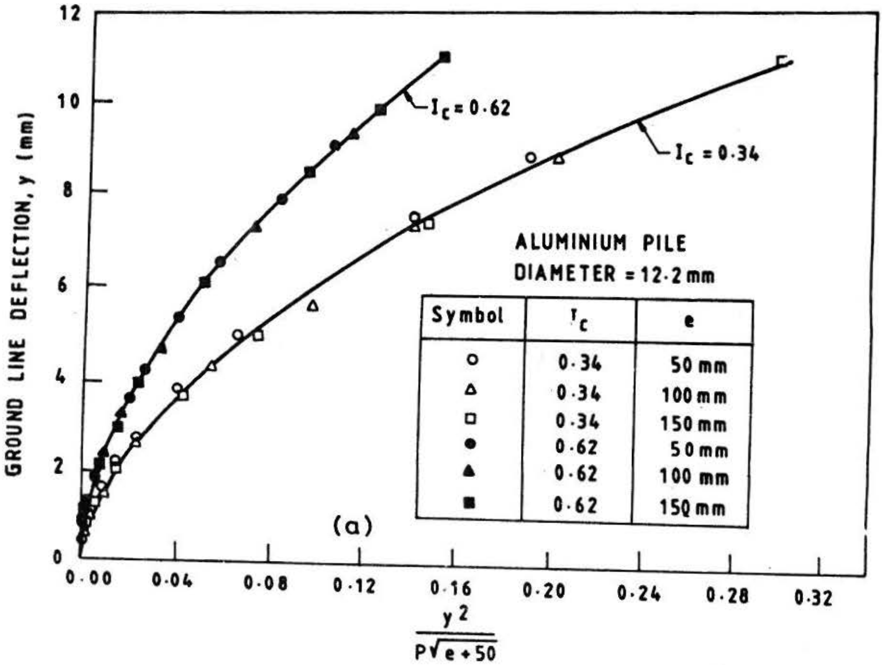


FIGURE 9 : Plots of Y Versus $Y^2/P\sqrt{e+50}$

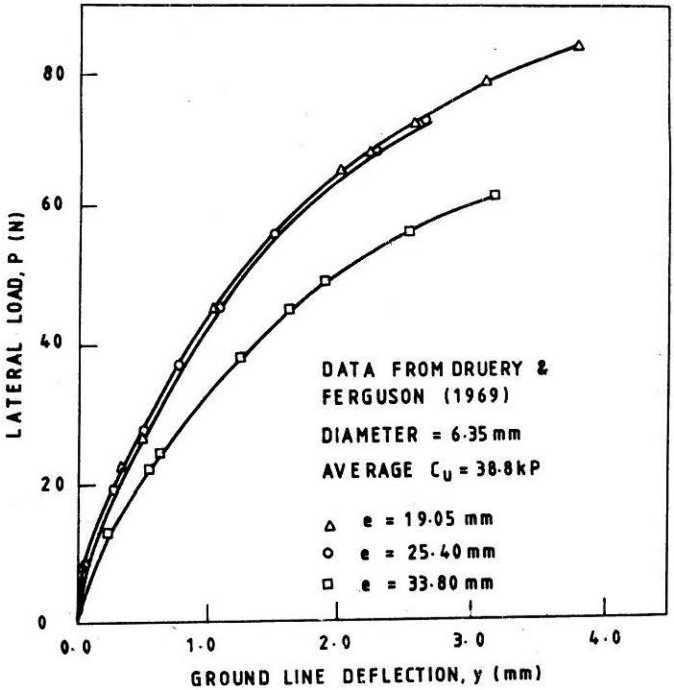


FIGURE 10 : Load-Deflection Curves (Data from Druery and Ferguson, 1969)

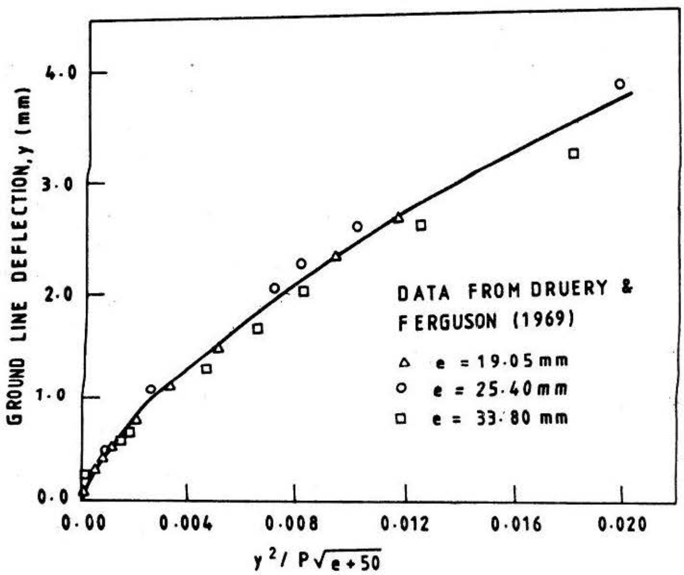


FIGURE 11 : Y Versus $Y^2/P\sqrt{e+50}$ (Data from Druery and Ferguson, 1969)

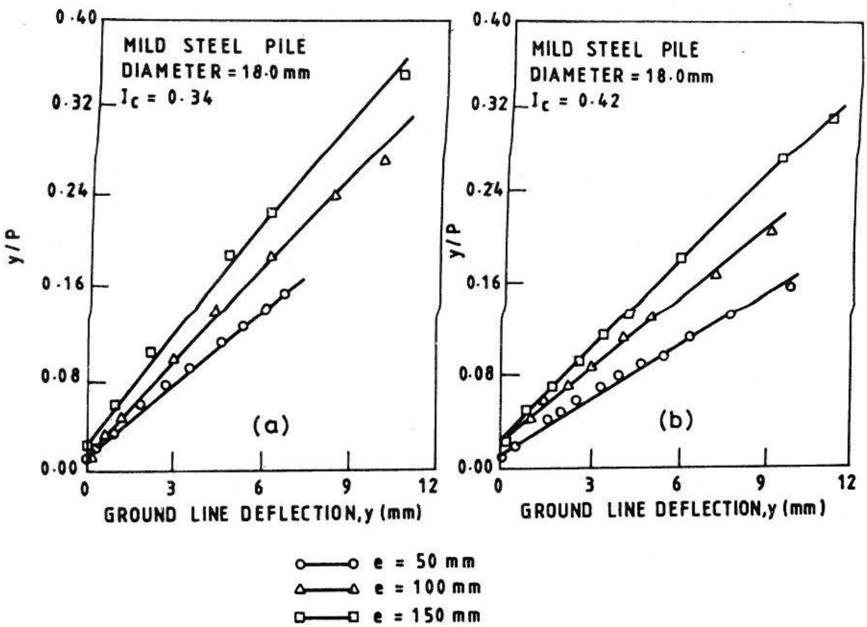


FIGURE 12 : Transformed Plots of Y/P Versus Y (Present Test Results)

Hyperbolic relationships

In the field of Geotechnical Engineering, the hyperbolic relationship are very well used to characterize stress strain relationship (Kondner, 1963; Duncan and Chang, 1970; Sridharan and Narasimha Rao, 1972). The typical load-deflection curves presented in Fig. 2 indicate that they can be represented by rectangular hyperbolic formulation between deflection and load as given by

$$P = \frac{Y}{a + bY} \quad (8)$$

where P = lateral load

Y = deflection at ground level corresponding to load P

'a' and 'b' are constants.

From the property of rectangular hyperbola, a transformed plot drawn between Y/P and Y should be a straight line. Typical transformed plots of Y/P Vs. Y for the present model tests are shown in Fig. 12 and these plots are straight lines. The linear regression analysis carried out yielded correlation coefficients (r) of 0.95 to 0.99 for the straight line plots. This brings out that the lateral load-deflection behaviour can be very well represented by a rectangular hyperbola. For the load tests with large deflections assuming $Y - \infty$, ultimate load can be expressed by

$$P_u = \frac{1}{b} \tag{9}$$

From the present piles tested, the computed values of ultimate lateral capacities from this equation are presented in Table 3. The observed capacities are the capacities obtained from load tests and are also presented in the same table for the purpose of comparison. The estimated lateral capacities are observed to be considerably greater than the test values. It may be noted that the observed ultimate load carrying capacity is taken as the load corresponding to a deflection equal to 20% of the diameter of the pile. However, for the load to reach an asymptotic value with respect to deflection axis, the deflection that is necessary is quite high. Because of other considerations, load tests can not be carried out to such high values of deflection. From the functional point of view, different values of permissible deflections are usually suggested depending on the type of structure and in view of this it is difficult to suggest generalized values of acceptable deflection. However, from the shape of the load-deflection curves, there is enough data to confirm that these curves can be very well represented by a rectangular hyperbola and using this concept, the capacities of piles can be fixed up based on the acceptable limits of deflections.

Verification with reported results

The reported results of Druery and Ferguson (1969), Briaud et al. (1983) and Meyerhof et al. (1988) were used to verify hyperbolic relationships. The load-deflection curves of model piles tests reported by Druery and Ferguson (1969) are already presented in Fig. 10. The transformed plots of Y/P versus Y for these results are shown in Fig. 13.

Briaud et al. (1983) reported the results of three field model tests conducted on concrete drilled shafts at a site in Texas A & M University. The soil at this site is a stiff clay with an average undrained shear strength of 95.8 kPa. The load-deflection curves are shown in Fig. 14, and the transformed plots of Y/P versus Y are shown in Fig. 15. These tests were not carried out up to failure. The failure loads corresponding to a deflection of 20% of the pile diameter were extrapolated using the hyperbolic relationships.

Meyerhof et al. (1988) presented the results of model piles of steel, timber and nylon of 12.5 mm diameter and 305 mm embedded depth. The soil used was a medium plastic clay with an undrained shear strength of

TABLE 3
Estimated Lateral Capacities of Piles Using Hyperbolic Method

Sl. No.	Pile Particulars	Consistancy Index I_c	Load Eccentricity e (mm)	Regression Coefficient b	Lateral Capacity (N)	
					Estimated (1/b)	Observed
1	13.5 mm diameter mild steel pile	0.72	50 100 150	0.0073 0.0090 0.0103	137.0 111.1 97.1	89.0 72.5 62.0
2	18.0 mm diameter mild steel pile	0.42	50 100 150	0.0149 0.0203 0.0261	67.1 49.3 38.3	49.0 35.0 30.0
3	21.5 mm diameter mild steel pile	0.62	50 100 150	0.0076 0.0091 0.0100	131.6 109.9 100.0	100.5 80.0 72.5
4	12.2 mm diameter aluminium pile	0.34	50 100 150	0.0201 0.0286 0.0291	49.8 35.0 34.4	42.5 30.0 26.0
5	18.4 mm diameter aluminium pile	0.42	50 100 150	0.0168 0.0239 0.0242	59.5 41.8 41.3	51.0 36.0 31.0

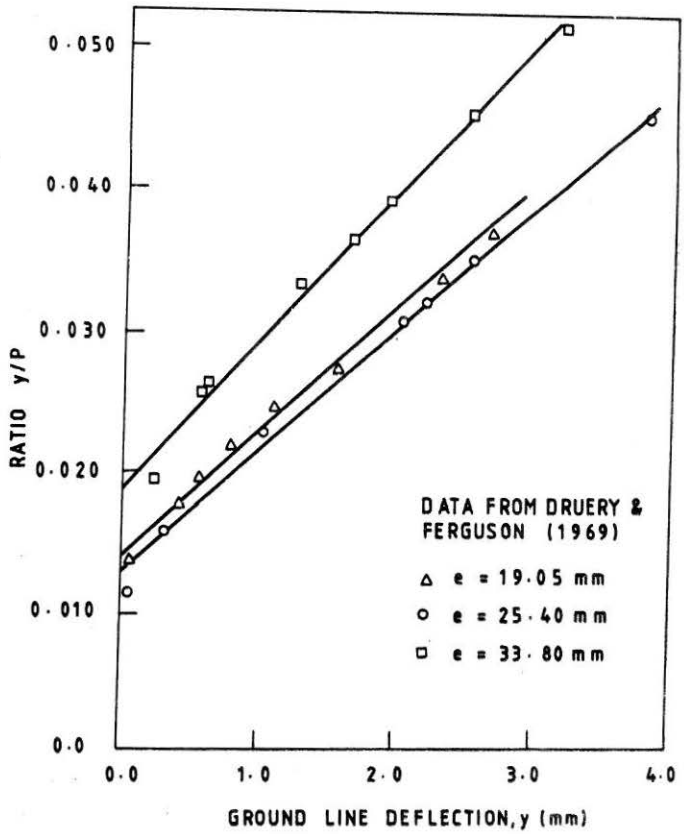


FIGURE 13 : Transformed Plots of Y/P Versus Y (Data from Druery and Ferguslon, 1969)

24 kPa. The load-deflection curves are shown in Fig. 16 and the transformed plots of Y/P Vs. Y are observed to be straight lines with correlation coefficients greater than 0.9 as shown in Fig. 17. Hence the lateral load-deflection curves can be very well represented by rectangular hyperbolae. The reported and estimated lateral capacities of all these results are summarized in Table 4. The lateral capacities estimated by hyperbolic methods are considerably high in all the cases.

Conclusions

The results of the model tests confirm that the ultimate lateral capacity of rigid pile is linearly proportional to the diameter. There is a significant reduction in the lateral capacity with load eccentricity and a relationship has

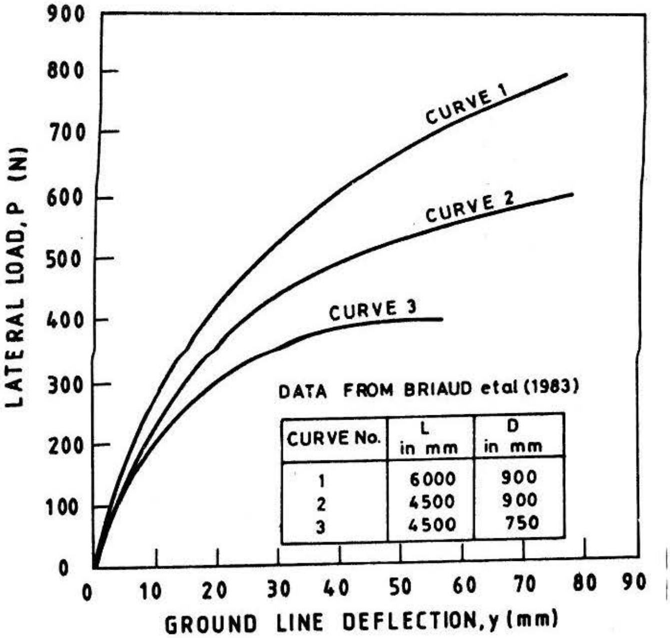


FIGURE 14 : Load Deflection Curves (Data from Briaud et al., 1983)

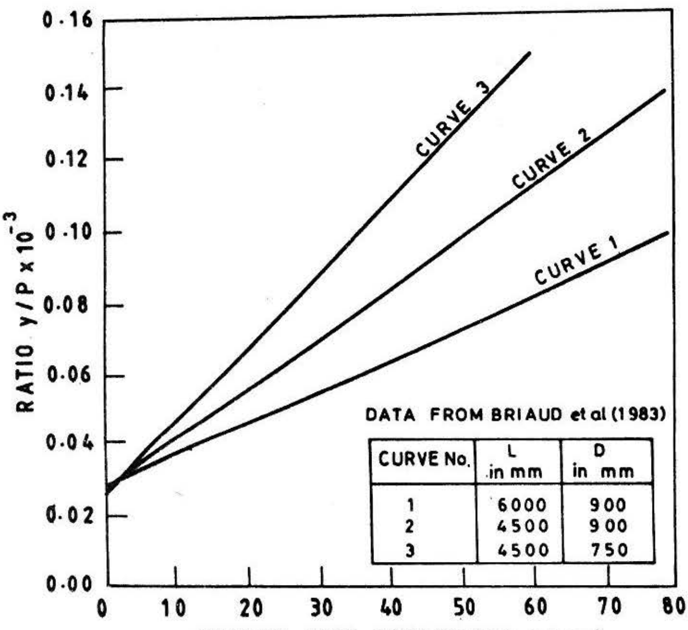


FIGURE 15. Transformed Plots of Y/P Versus Y (Data from Briaud et al., 1983)

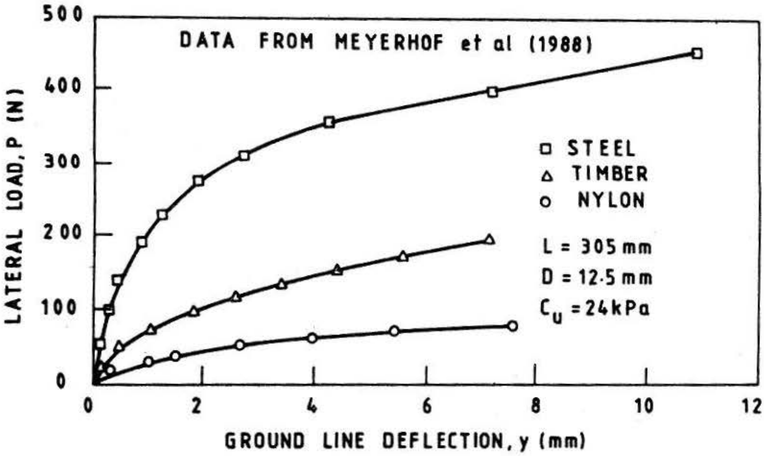


FIGURE 16. Load Deflection Curves
(Data from Meyerhof et al., 1988)

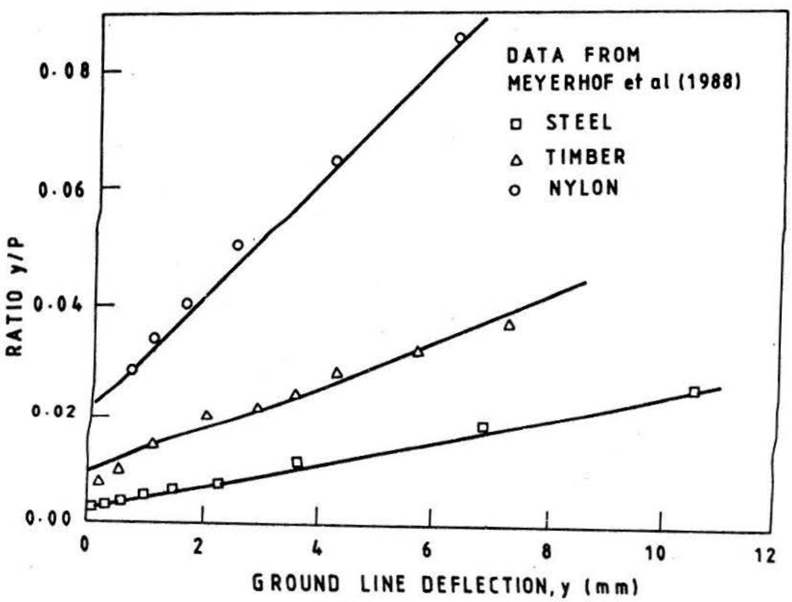


FIGURE 17. Transformed Plots of Y/P Versus Y
(Data from Meyerhof et al., 1988)

TABLE 4
Estimated Lateral Capacities of Piles Using Hyperbolic Method (from reported results)

Sl. No.	Pile Diameter, D (mm)	Pile Length, L (mm)	Load Eccentricity, e (mm)	Regression Coefficient, b	Lateral Capacity P_u		Results Used From
					Estimated (l/b)	Observed	
1	6.35	146.1	19.05	8.70×10^{-3}	114.9N	69.5N	Druery and Ferguson (1969)
2	6.35	139.7	25.40	8.70×10^{-3}	114.9N	69.5N	Druery and Ferguson (1969)
3	6.35	132.1	33.78	1.06×10^{-2}	94.3N	53.0N	Druery and Ferguson (1969)
4	900.00	6000.0	0.00	8.92×10^{-7}	1121.1kN	954.4kN	Briaud et al. (1983a)
5	900.00	4500.0	0.00	1.32×10^{-6}	757.8kN	678.3kN	Briaud et al. (1983a)
6	750.00	4500.0	0.00	1.68×10^{-6}	595.2kN	454.3kN	Briaud et al. (1983a)
7	12.50	305.0	0.00	2.00×10^{-3}	500.0N	360.0N	Meyerhof et al. (1988)
8	12.50	305.0	0.00	4.00×10^{-3}	250.0N	158.0N	Meyerhof et al. (1988)
9	12.50	305.0	0.00	1.10×10^{-2}	90.9N	55.5N	Meyerhof et al. (1988)

been developed between load-deflection characteristics and the load eccentricities. These relationships can be conveniently used to estimate the lateral load-deformation behaviour at different load eccentricities using the results of load tests conducted at a particular eccentricity. Further, the nondimensional plots drawn between $P_u / C_u DL$ and e/L suggest that the value $P_u / C_u DL$ decrease with e/L and is fairly constant at any given value of e/L even for different values of C_u and D. A relationship has been developed between $P_u / C_u DL$ and e/L based on nonlinear regression analysis which can be used to estimate the lateral capacity of a pile. It is shown that the lateral load-deflection curves can be very well represented by rectangular hyperbola. These relationships can be conveniently used to extrapolate the lateral load carrying capacity of a pile corresponding to any given deflection. All the proposed correlations compare well with the results of the present investigation and the published data.

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