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BEHAVIOUR OF ISOLATED BORED ENLARGED BASE PILE UNDER SUSTAINED VERTICAL LOADS

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ABSTRACT

Many static methods and dynamic formulae are available in the published literature, for predicting the pile capacities. So also, the procedures for evaluating the ultimate capacities based on the load test data. All these methods and procedures, instead of providing unique answer, give different load values. Thus placing the designer in a dilemma with respect to the method that is to be adopted.

Most of experimental suggestions are based on the results of small size piles driven in sandy deposits. But none of these suggestions try to dwell upon their limitations.

In the present study an attempt has been made to analyse the behaviour of isolated piles, based on series of field tests, conducted on bored piles with enlarged base in a uniform deposit of silty sand.

The performance mechanism of isolated piles have been studied. Also suggestions for estimating their capacities based on load tests and static methods have been provided. The predicted load-settlement behaviour by the elastic theory approach does not agree with the observed one.

Key words: alluvial deposit, bearing capacity, cast-in-place pile, field test, foundation, load test, sandy soil, static, vertical load
IGC: E 4

INTRODUCTION

Piles are increasingly used to transfer effectively the loads of the super structure to the sub-soil, when the soil conditions at the site and certain other functional requirements, discard the use of shallow foundations. Choice of the piles depend upon loading conditions, sub-soil characteristics and technical requirements. The mechanism of load transfer by a pile to the surrounding soil depends upon the type of load, the sub-soil characteristics and type of pile itself as shown in Fig. 1. The bearing capacity of piles consists of two components e.g. skin friction and point bearing and contribution of each of these components depend upon the type of soil, stratification, if any, pile cross-sections, pile properties. One of the three general mode of failures e.g. general shear failure, local shear failure and punching shear failure, may be envisaged in case of cohesionless soils. But normally in case of piles in cohesionless soils, punching modes of failure can be anticipated, due to which, in general, a definite failure load can not be assigned to the piles. In order to design a safe and economical pile foundation, the designer should ensure adequate safety against failure load, the settlement under the specified loads should be within permissible limits.

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Bearing Capacity from Soil Properties

The ultimate bearing capacity Q_u, of isolated piles, embedded in soil is derived from two components, the skin friction (Q_s) and point bearing (Q_p) Fig. 1.

$$Q_u = Q_p + Q_s = q_p \cdot A_p + f_s A_s \tag{1}$$

where, q_p = the unit ultimate point bearing with area A_p, f_s = the average skin friction and A_s = surface area of pile shaft.

Point Bearing Capacity

The point bearing is normally estimated on the basis of approaches based on Prandtl (1920, 21) and Reissner (1924). These solutions are applied to the bearing capacity of footings and further extended to deep foundations by Terzaghi (1943), Caquot (1934) and Buisman (1935). These solutions in general have the following form:

$$q_p = cN_c\xi_c + qN_q\xi_q + \frac{1}{2}\gamma BN_\gamma\xi_\gamma \tag{2}$$

where c is the shear strength intercept, q and γ the overburden pressure and unit weight of soil respectively. N_c, N_q, N_γ and ξ_c, ξ_q, ξ_γ are the bearing capacity and shape factors respectively. Both these factors are the dimensionless function of φ, the angle of internal friction. In the case of cohesionless materials such as dry submerged sand

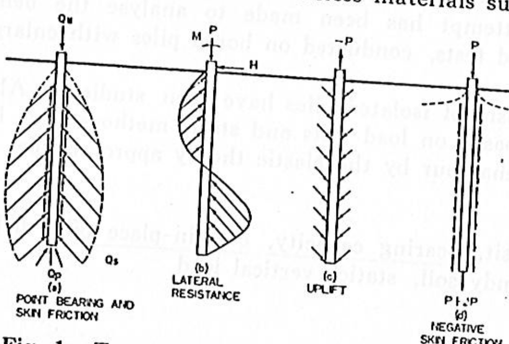


Fig. 1. Types of load transfer from pile to soil

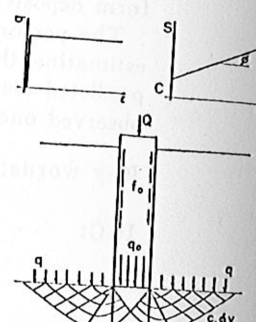


Fig. 2. (a) Prandtl reissner solution as applied by Caquot and Buisman

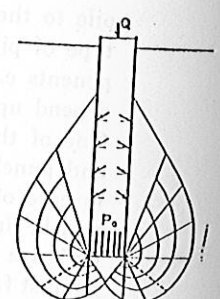


Fig. 2. (b) Shear pattern with rupture lines reverting to the shaft

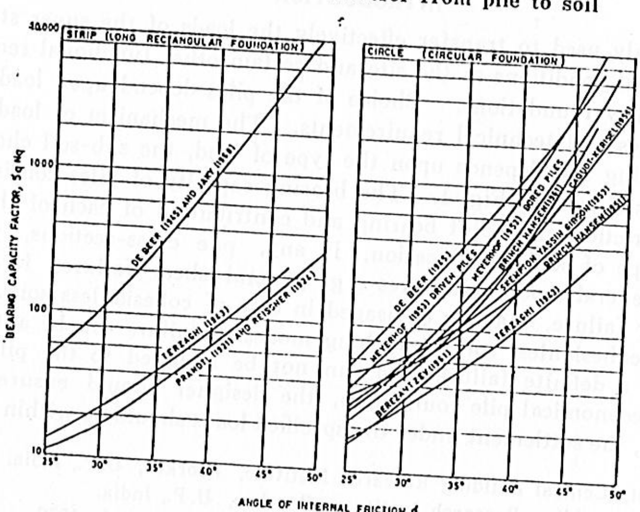


Fig. 3. Theoretical bearing capacity factors for deep foundations

$$q_p = qN_q\xi_q + \frac{1}{2}\gamma BN_\gamma\xi_\gamma \qquad (3)$$

In the case of cohesive materials

$$q_p = cN_c\xi_c + q \qquad (4)$$

The general shear patterns adopted in these procedures are given in Fig. 2(a). However, using the same approach but with different shear patterns: Fig. 2(b) having rupture lines reverting to the shafts, De Beer (1945, 1948), Jakey (1948) and Meyerhof (1951) have obtained similar solutions for the bearing capacity of piles and other deep foundations. The values of N_c and N_γ with φ have been provided by the above investigators in the similar pattern of Terzaghi and Peck (1967). Herein, for homogeneous sand the unit point resistance is represented by:

$$q_p = p_0N_q\xi_q + \frac{1}{2}\gamma BN_\gamma\xi_\gamma \qquad (5)$$

where p_0 is the effective overburden pressure. The values of $N_q\xi_q$ by different approaches are shown in Fig. 3. The values of N_q for different D_c/B values for the case of driven piles in sand is provided in Fig. 4 (Meyerhof, 1976). Average Skin Friction:-For piles driven in c, φ soils the average skin friction to the applied load is given by:

$$f_s = C_a + K_sp_0\tan\delta \qquad (6)$$

The first part is independant of normal pressure acting on the contact area, commonly termed as adhesion. The second part called as skin friction, is considered to be proportional to the normal pressure. In the case of cohesionless material such as dry and submerged sand the skin friction is given by:

$$f_s = K_sp_0\tan\delta \qquad (7)$$

where, K_s = the average coefficient of earth pressure on the pile shaft, p_0 — the average effective overburden pressure and δ — the angle of shearing resistance between pile and soil. Based on the works of Meyerhof (1953), Kerisel (1964), and Vesic (1967) Meyerhof (1976) provides values of skin friction f_s, for different pile types. The variation of f_s with φ has been illustrated in Fig. 5.

Bearing Capacity from Penetration Tests

The ultimate bearing capacity of piles in sands can also be closely estimated from the

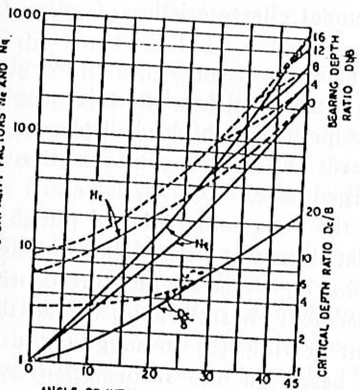


Fig. 4. Bearing capacity factors and critical depth ratio for driven piles

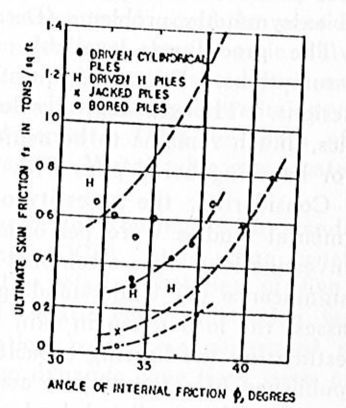


Fig. 5. Ultimate skin friction of piles in sand

data of static cone penetration test

$$Q_u = q_c \cdot A_p + f_s A_s \tag{8}$$

According to Meyerhof (1956) f_s can be evaluated from the following expression for driven piles

$$f_s = \frac{\bar{q}_c}{200} \tag{9}$$

and hence

$$Q_u = q_c A_p + \frac{\bar{q}_c}{200} A_s \tag{10}$$

where q_c and $\bar{q}_c$ are average static cone resistance around toe and for embedded length of pile respectively. Mohan et al. (1963) have shown $f_s = \bar{q}_c/50$ to $\bar{q}_c/100$. De Beer (1964) investigated theoretically that ultimate point bearing capacity q_p of bored piles in sand could be about 1/3 less than that of driven piles. Finally Meyerhof (1976) suggests that ultimate bearing capacity of bored piles is reduced by 1/2 to 1/3 than that of driven pile as calculated from static penetration test data. Using standard penetration test value N, Meyerhof (1976) also suggests the following empirical relationships for skin friction (tsf) and point bearing (tsf) in the case of piles embedded in cohesionless soil

$$q_p = 4N \cdots \text{for driven piles} \tag{11}$$

$$q_p = 1.2N \cdots \text{for bored piles} \tag{12}$$

$$f_s = \frac{N}{50} \cdots \text{for driven piles} \tag{13}$$

$$f_s = \frac{N}{100} \cdots \text{for bored piles} \tag{14}$$

Prediction of Load Settlement Characteristics

Poulos and Davis (1968) and Mattes and Poulos (1969) have presented, certain elastic solutions, to calculate the axial displacements of axially loaded piles using Mindlin's solutions. Poulos (1972) also presented a method to predict load-settlement behaviour of a pile or pier based on the above solutions. A linear relationship between skin friction and settlement and point bearing has been assumed to failure. These solutions require the evaluation of certain elastic constants of the soil, which are not easy to achieve. Further, the recent formulations and numerical techniques such as finite element methods also provide possibilities of predicting the load displacement characteristics of piles treating them as axisymmetry problems (Desai, 1972).

The procedures available for predicting load-displacement have varying unrealistic assumptions. Even if unquestioned these procedures may not be easily adopted for routine designs. Though, logical solutions require the prediction of load-displacement characteristics, much remains to be achieved in this regard. Further, no information seems to exist for handling bored piles with or without enlarged base.

Considering, the necessity of understanding the behaviour of bored piles, detailed experimental studies were performed on bored isolated enlarged base piles. The experimental investigations were attempted to study the following aspects which are considered to be of significance (a) Understanding the load displacement characteristics of isolated piles and to assess the load taken in skin friction and point bearing (b) to suggest the procedure for estimating the bearing capacity of bored piles based on the information available in the published literature (c) to assess the bearing capacity of pile from load test data (d) to compare the predicted load-settlement curve obtained by using method of Poulos (1972) with the observed one. The layout of the test facilities is illustrated in Fig. 6 and Photo. 1 and detailed in Table 1.

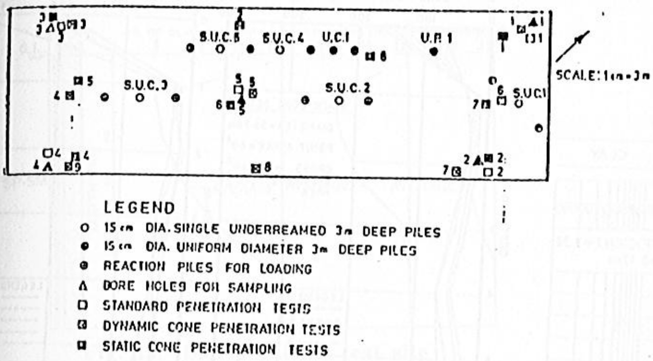


Fig. 6. Layout of test site

Photo. 1. A view of test site

Table 1. Details of test-facilities

Sl. No.	Pile Details: Type	Shaft Diameter cm	Under-reamed Diameter cm	Depth below Ground Level m	Designation	Type of Test	Remarks
1.	Single underreamed	15	37.5	3	SUC 1 to SUC 5	Compression	5 numbers
2.	Uniform diameter	15	—	3	UPI	Uplift	One number
3.	Uniform diameter	15	—	3	UCI	Compression	With air gap at bottom

SITE CONDITIONS

The experimental investigations were performed at Dhandhera, a place situated in the South-East direction to Roorkee at 5 km distance. The test site forms a part of the Cantonment area. Geologically, the site and the adjoining area fall within the Gangetic plains of the Northern India. Mainly the soil deposit encountered could be considered to be a part of the alluvium deposit. The surface accumulation of the Gangetic alluvium consists of sand and silt and in deeper depths clay medium. The sandy silt deposit encountered in the area is of considerable thickness. This site was chosen because preliminary explorations revealed that the site contains uniform silty sand deposit. Upto the depth of 8.0 metres the characteristics of the sub-soil remained the same. Water table was located below 8.0 m depth.

Detailed soil exploration of the site included exploratory borings with standard penetration tests (SPT), dynamic cone penetration tests (DCP), static cone penetration tests (SCP), and collection of undisturbed samples. The relative positions of five exploratory borings, nine deep dynamic cone penetration and 8 static cone penetration tests are also shown in Fig. 6 and Photo. 1. Standard penetration tests were performed according to the existing Indian Standard IS : 2131-1963. Deep dynamic cone tests were performed as per Indian Standard IS : 4968 (Pt-II) 1968.

Ranges of particle size distribution curves are shown in Fig. 7. These ranges cover the entire area and in as much reflect the gradation of soil collected from different boreholes situated at different positions. The results of the static cone penetration resistance are

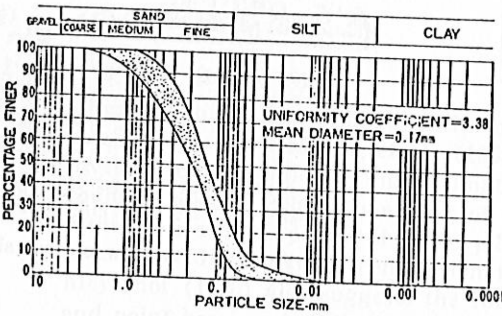


Fig. 7. Particle size distribution of soil at the test site

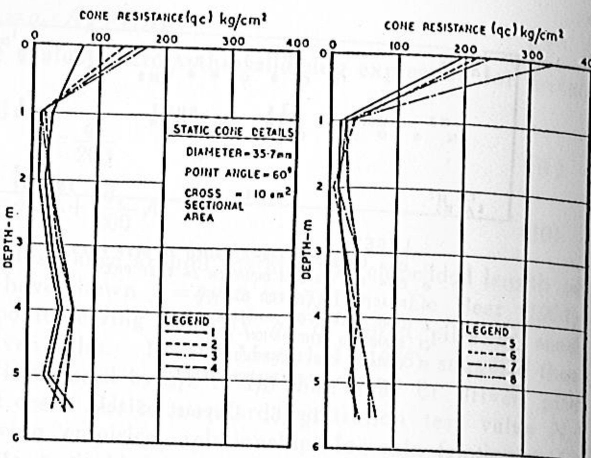


Fig. 8. Variation of static cone resistance with depth

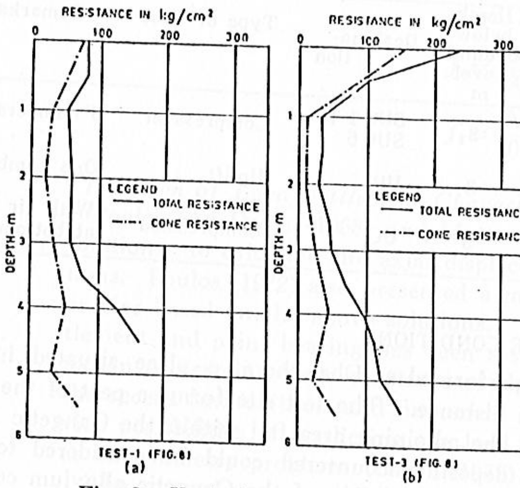


Fig. 9. Variation of total and cone resistance with depth

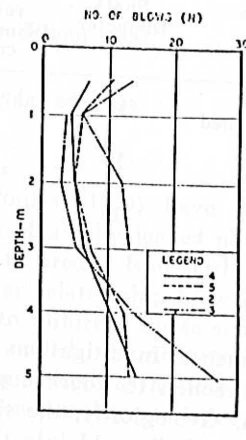


Fig. 10. Standard penetration test

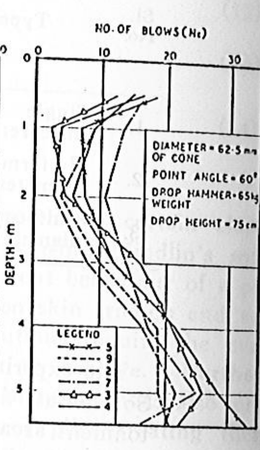


Fig. 11. Dynamic cone test

shown in Fig. 8. The skin resistance and cone resistance are separated and are given in Fig. 9. The results of standard penetration tests are presented in Fig. 10. The figure presents the observed spoon resistance from four different boreholes. It is interesting to note that the soil deposit is fairly uniform at the test site and N-value increases more or less linearly with depths. The variation of the dynamic cone resistance with depths is presented in Fig. 11.

A brief summary of exploratory boring is presented in Fig. 12. The variation of angle of internal friction with depth as well as the moisture content with depth have been presented. The soil in the area could be considered to fall under IS classification SP-SM. The values of angle of internal friction were determined from laboratory triaxial shear tests on 38 mm diameter and 75 mm high undisturbed soil specimens. The undisturbed samples were collected in the 100 mm diameter thin-walled tubes. From these tubes test specimens in laboratory were obtained by inserting 38 mm diameter tubes.

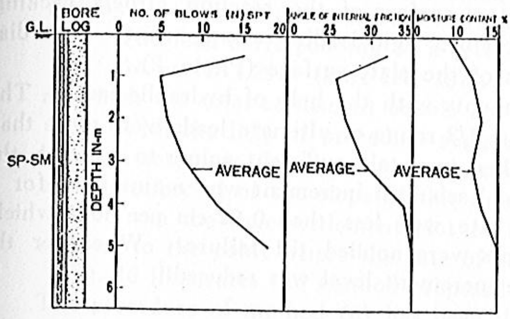


Fig. 12. Soil profile at test site

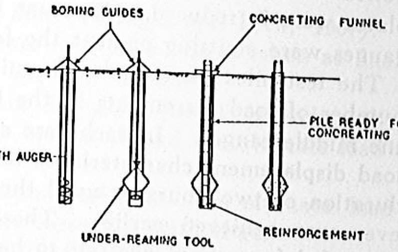


Fig. 13. Construction procedure for bored under ream piles

CONSTRUCTION OF TEST FACILITIES

The construction of underream piles were done as per I. S. 2911 (Part-III)-1973 and is illustrated in Fig. 13. In order to perform the load test effectively on these, suitable reaction facility was designed for each case. Precaution was taken to ensure that the total reaction capacity is at least 3 times the estimated single pile capacity so that the reaction piles do not show any movement till the ultimate load on the test pile. A typical reaction arrangement is illustrated in Photo. 2.

In order to understand the failure mechanism as well as estimate the skin friction along shaft of the main test pile a uniform dia. pile was constructed with air gap at the bottom. After boring to the desired elevation, a rod with a plate of 15 cm dia. was lowered and held in position with supports at ground surface. Sufficient air gap between plate bottom and bore hole bottom was ensured. The pile was concreted and the test results on such piles could give reasonable estimate of the skin friction. The arrangements for uplift test are depicted in Photo. 3 which was carried out on a separate uniform diameter pile to estimate the skin friction in uplift.

LOAD TEST PROCEDURE

The load tests on the piles were carried out after a period of about three months after their installation. On the top surface of the pile cap of isolated piles a mild steel plate

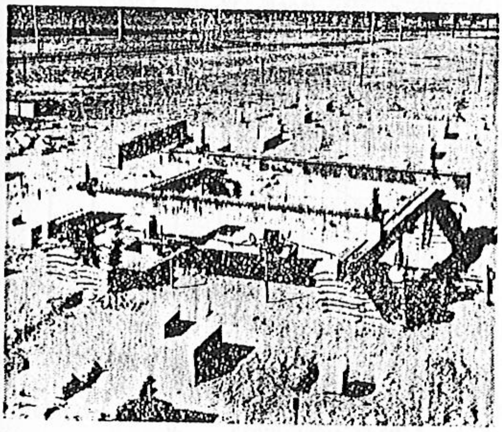


Photo. 2. Typical reaction arrangements two single piles under compression test

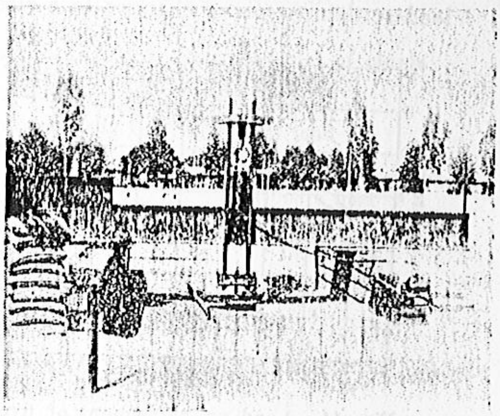


Photo. 3. Arrangements for uplift test

was placed centrally. On top of the plate hydraulic jack proving ring assembly was positioned. In between the proving ring and bottom surface of the reaction girders packing plates were introduced. Care was taken to obtain a rigid loading arrangement. Four dial gauges were abutting against the four corners of the plate surface (Photo. 2).

The test piles were loaded in suitable increments with the help of hydraulic jacks. The number of load increments in the first and last 1/3 range of ultimate loads were more than the middle ranges. In each case care was taken to obtain sufficient points to describe the load displacement characteristics of the piles. Each load increment was maintained for a duration of two hours or until the deflection rate was less than 0.02 cm per hour whichever was achieved earlier. These increments were applied till failure. Whenever the rate of deformation was felt to be faster the increment level was reduced.

PERFORMANCE OF PILES

The piles were placed at different arbitrarily chosen locations of the site. The load-displacement characteristics of these five piles have been shown in Fig. 14. It is seen from this figure that there is no appreciable scatter. Hence, it can be construed that the soil deposit in the area to be more or less uniform and of similar type. Such consistencies underline the reproducibility of test procedures and techniques, and as such ensure that the different tests can be compared. In most of the pile cases it was observed that the piles failed due to punching. Though the testing in the field could not positively show the mode of failure in all cases, punching was observed and separation between soil and shaft was noticed at top reaches. Around the piles no bulging was noticed, indicating that the failure lines of the bottom bulb did not necessarily travel to the ground surface.

In all the individual load-displacement characteristics of the isolated piles no definite failure load was indicated. However, the initial and final rate of deformation with load was different. This was ascertained that the load-displacement characteristics could be approximated by two straight lines of two different slopes.

Load-settlement curve for a typical pile SUC 4 is given in Fig. 15 which also represents almost the average of all five tests.

Ultimate capacity from Load Tests

For evaluating the ultimate load from the compression tests data the following methods were used:

1. The ultimate load has been determined as the point where the curved part of the load-settlement curve changes to a straight line (Schenck, 1955; Vesic, 1965).

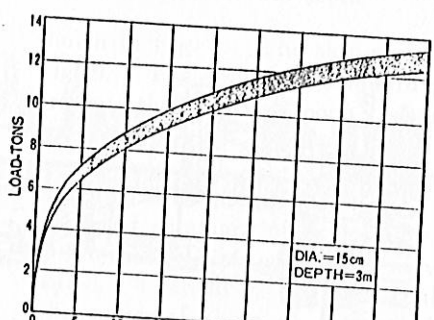


Fig. 14. Load vs. settlement of five pile tests (SUC 1-SUC 5)

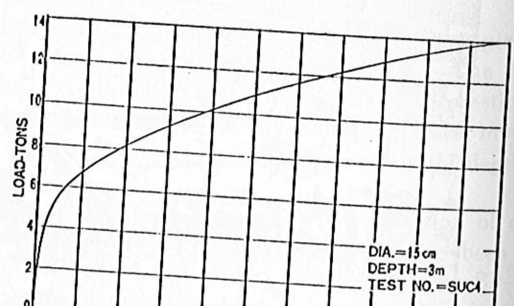


Fig. 15. Load vs. settlement of pile SUC 4

2. The ultimate load has been taken as the point at which initial and final tangents of the load settlement curve intersects (Mansur and Kaufmann, 1956).

3. The load corresponding to a settlement of 51 mm (Terzaghi and Peck, 1967).

4. As per IS : 2911 (Part-III)-1973. The ultimate load on a pile is taken as final load at which the total settlement equals 7.5 percent of the bulb diameter.

5. With the help of procedure suggested by Chin Fungkee (1970) wherein, the plot between settlement/load and settlement of piles have been used and the ultimate load taken as the reciprocal of the slope of this line.

6. From the time-settlement plot of the compression test data, the ultimate load correspond to the point where the rate of settlement increases fast with time.

Fig. 16 illustrates the method proposed by Kee (1970) for a typical case of pile SUC 4. The procedure of method (6) has been illustrated in Fig. 17. The ultimate loads for a typical pile SUC 4 as obtained by different methods have been compared in Table 2. Out of the six methods it is obvious that the methods which rely on tangents (1 and 2) requires judgement and as such may result in erroneous estimate of ultimate loads.

The choice of any particular settlement value for defining the ultimate load as in (3) and (4) is more or less arbitrary, though followed by different Codes of Practices through out the world. Method (5) would be able to give reasonable estimate of the ultimate loads in which sharp peak loads are not indicated in the load-settlement curves. That is after certain load increments the settlement increases in a definite way with each load increment. However this method is reported to estimate the values on higher side. By far the suitable procedure of estimating the ultimate loads of the piles which do not have sharp peaks is to follow the method (6). Herein, the time-settlement relationship for

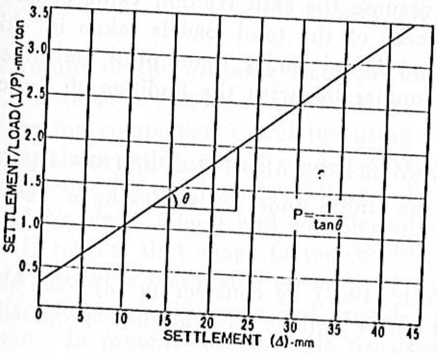


Fig. 16. Plot of Δ/p vs. Δ suggested by Kee (1970)

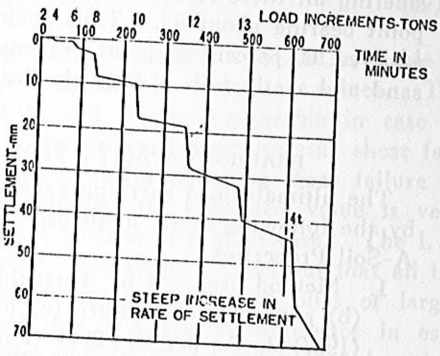


Fig. 17. Time displacement plot of single under ream pile SUC 4

Table 2. Ultimate load by different methods from a typical load displacement plot for pile SUC 4

Sl. No.	Method	Ultimate load tonnes
1.	Single Tangent Method	13.5
2.	Double Tangent Method	9.5
3.	Load Corresponding to 51 mm displacement (Terzaghi and Peck, 1967)	13.8
4.	As per IS 2911 (Part-III) 1973	11.3
5.	From Δ/p and Δ plots as proposed by Kee (1970)	13.8
6.	From Time-Settlement plot	14.0

each load increment is continuously plotted. Near the failure range the rate of increase in settlement would be very fast and would be adequately indicated in the plot, as shown in Fig. 18. The ultimate load would then correspond to the load after the application of which the rate of increase in settlement is quite rapid. For short-bored under-ream piles this method is suggested for evaluating the ultimate loads, when the piles are embedded in silty-sand or sandy soils. According to it the ultimate load is 14 tonnes.

Skin Friction and Point Bearing

One of the important aspects of the load-displacement characteristics of single-piles is the separation of loads taken by skin friction and point bearing. In majority of the cases it is reasonable to presume that the skin friction is mobilized at the point where the initial straight line portion of the load-displacement curve enters to a curved form (Leonards, 1972). For a typical pile SUC 4 it is also illustrated in Fig. 15.

In order to estimate the skin friction, pull-out test was performed on a uniform diameter pile with shaft diameter of 15 cm and of 3.0 m long, UPI. The result of the pull-out test on uniform diameter pile is given in Fig. 18. Further the result of the compression test (UCI) on uniform diameter pile with air gap at the bottom has been given in Fig. 19.

From these two figures the load taken in skin friction can be accurately ascertained. It is seen that the skin friction load from pull-out test works out to be 5 tons and based on air gap test works out as 7.0 tons. The movement required to mobilize skin friction is in the order of 4 mm or 2.7 percent of the shaft diameter.

The compression test data in the series of single piles also indicate the skin friction to be of the order of 4.0-7.0 tons and mobilization can be considered to occur with a movement of 2.5 to 6 mm or 2.0 to 4.0 percent of the shaft diameter (Leonards, 1972). Considering all these results it is reasonable to assume the skin friction value of about 6 t and point bearing about 8 t. Thus about 40 percent of the total load is taken in skin friction and rest 60 percent in point bearing for short bored under ream piles embedded in silty sand and sandy soils. This observation is comparable with the findings of Vesic (1967).

BEARING CAPACITY FROM SOIL PARAMETERS AND PENETRATION TESTS

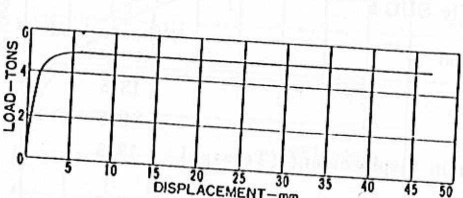
The ultimate load carrying capacities of the single underream piles have been evaluated by the following static methods.

A-Soil Properties

1. Method proposed by Terzaghi and Peck (1967) by considering (a) local shear failure (b) general shear failure (c) punching shear failure 2. Method proposed by Vesic (1967) 3. As per IS : 2911 (Part-III)-1973.

B-Penetration Tests

1. Based on Static Cone Resistance as proposed by Meyerhof (1956) considering reduc-



Figs 18. Load vs. uplift for uniform diameter plie

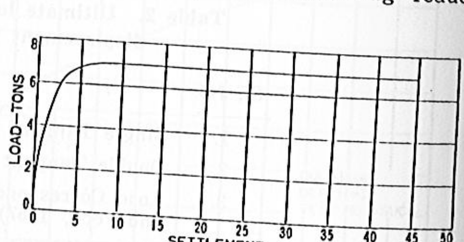


Fig. 19. Load vs. settlement for uniform diameter pile (with air gap)

tion for bored piles as proposed by (a) De Beer (1964) (b) Meyerhof (1976).

2. Based on SPT values as proposed by Meyerhof (1976).

Method A-1

$$Q_u=(0.6\gamma BN_r+\gamma D_f N_q)A_p+2\pi r f_s D_f \tag{15}$$

where γ=unit weight of soil, B=dia. of pile, D_f=Depth of embedment, r=radius of shaft, f_s=skin friction resistance coefficient, A_p=base area of pile, N_r, N_q=non-dimensional bearing capacity factors

Method A-2

$$Q_u=(0.6\gamma BN_r+\gamma D_f N_q)A_p+(K_s\tan\delta q)2\pi r D_f \tag{16}$$

where K_s=Co-efficient of earth pressure, δ=angle of shearing resistance between pile and soil, taken equal to angle of internal friction, q=Effective over burden pressure.

Method A-3

$$Q_u=\frac{\pi}{4}(D_u^2-B^2)\left(\frac{1}{2}\gamma D_u N_r+\gamma D_1 N_q\right)+\frac{\pi}{4}B^2\left(\frac{1}{2}\gamma BN_r+\gamma D_f N_q\right)$$
$$+\frac{1}{2}\pi K_s \gamma B\tan\delta D_f^2 \tag{17}$$

where D_u=dia. of bulb, D_1=depth of center of bulb

Method B-1

$$Q_u=q_c A_p+\frac{\bar{q}_c}{200}2\pi r D_f \tag{18}$$

Method B-2

$$Q_u=(1.2N)A_p+\frac{N}{100}2\pi r D_f \tag{19}$$

The results of the ultimate loads calculated by above methods are tabulated in Table 3. From this table it can be seen that there is a large variation in estimated values. The point bearing component calculated using values of N_r and N_q is comparable in case of punching shear failure (method A-1 (c)) while it is higher considering general shear failure and lower for local shear failure. Following N_q value for punching shear failure as proposed by Vesic (1967) and considering shape factor of 3, the estimated value is very high. It reflects that shape factor considered may not be true in present case. The I. S. formula also give reasonable estimate of point bearing. It may be pointed out that all the formulae are generalized one and provides reasonable estimate in case of piles of larger diameter. In present case the pile size is quite small due to which the difference in estimated and observed one is more. These estimates also indicate that there is no other than punching shear failure. The value of N_q, which is mainly responsible for point bearing component, may be taken, either the average of two values corresponding to general shear failure and local shear failure (Terzaghi and Peck, 1967) or values proposed by Vesic (1967). The estimates of skin friction is on lower side. In case of Terzaghi and Peck formula an average value of friction has been used which may not be true friction in present case. In method (A-2) and (A-3) the value of co-efficient of earth pressure K_s is mainly responsible for the departure of estimate values. The value of K_s is dependent on pile diameter which decreases with the increase in pile diameter (Vesic, 1967). The values given by different workers are generally based on piles of larger diameter than the piles in present case. Thus the value of K_s may be more than 2.2 and 1.75 as used in method (2) and (3) respectively. In fact the relation proposed by Meyerhof (1956) using static cone penetration resistance values is for driven piles. After reducing the point bearing to one third as proposed by De Beer (1964) and Meyerhof (1976), the estimates are com-

Table 3. Ultimate capacity from soil parameters and penetration tests

S. No.	Method	Point Bearing tonnes	Skin Friction tonnes	Total Capacity tonnes	Remarks
A-1	Terzaghi and Peck (1967)				
	(a) Local shear failure	3.91	3.85	7.76	Value of N_r and N_q for local shear failure.
	(b) General shear failure	10.21	3.85	14.06	Value of N_r and N_q for general shear failure.
	(c) Punching shear failure	7.32	3.85	11.17	Average value of N_r and N_q for local and general shear failure.
A-2	Vesic (1967)	15.78	4.3	19.81	N_q factor for Punching failure along with shape factor of 3. $K_s=2.2$ for buried piles.
A-3	IS : 2911(Part-III) 1973 formula	6.06	3.42	9.48	N_r and N_q same as in method 1 (c) and $K_s=1.75$
B-1	Static cone Penetration Resistance				
	(a) Meyerhof (1956)	27.6	1.41	29.01	
	(b) De Beer (1964)	9.2	1.41	10.61	Reducing point bearing in B-1 (a) by 1/3 for bored piles.
	(c) Meyerhof (1976)	13.8	0.70	14.50	Reducing capacity in B-1 (a) by 1/2 for bored piles.
		9.2	0.47	9.67	Reducing capacity in B-1 (a) by 1/3 for bored piles.
B-2	SPT Values Meyerhof(1976)	10.60	0.99	11.59	

parable but on higher side. It is because the above recommendations are for uniform diameter piles whereas in the present case the piles are of enlarged base and hence more chances of disturbances.

The same is true in case of relationship proposed using SPT values. Considering the point bearing equal to 8 t, the unit point bearing works out about $q_c/3.45$ and $N/1.1$. The skin friction component estimated from penetration resistance is rather very low. It suggests that unit friction $\bar{q}_c/200$ or $N/100$ as given in formulae should be different. Taking the skin friction value of 6 t, the unit skin friction works out about $\bar{q}_c/47$ and $N/16.5$. This is in confirmation with other workers (Sanglert, 1972), where unit skin friction is recommended $\bar{q}_c/50$. For silty sand following relationships are given:

$$q_c = 3N(\text{Sanglert, 1972}) \tag{20}$$

and $$N_c = (\text{upto 4 m depth}) \tag{21}$$

$$=1.75N(4\text{ m to }10\text{ m depth})\ (\text{Dinesh Mohan et al., 1970}) \tag{22}$$

The above relationships also hold good at present site. Thus any penetration test resistance may be used for estimating ultimate loads. It may be pointed out that dynamic cone is quicker and economical than SPT or static cone penetration tests.

LOAD SETTLEMENT CURVE BASED ON ELASTIC SOLUTION

The overall load-settlement curve could be drawn by superposition of the skin friction versus settlement and point bearing versus settlement assuming a linear relationship between the two upto failure (Poulos, 1972). Using the value of modulus of elasticity of soil as 100 kg/cm² based on the values proposed by different workers for silty sand and computed from static and standard penetration test results (Shrivastava, 1971), the overall settlement curve has been drawn. The predicted alongwith observed load-settlement plot for pile SUC 4 is shown in Fig. 20. Although there is sufficient difference between the two, it can be seen that initial part of the predicted curve is comparable with the observed one. There is no comparison in second part. It may be due to the fact that E_s is dependent on stress-strain behaviour which is nonlinear in case of a soil and a single value cannot be assigned to it. It decreases as the deformation increases. That is why predicted settlements in second part are less than observed one. Another difficulty is in assessing the value of E_s as there is large variation in the values obtained by different methods. Thus it is difficult to predict load-settlement curve using elastic solutions and there is need to develop more systematic approach for it.

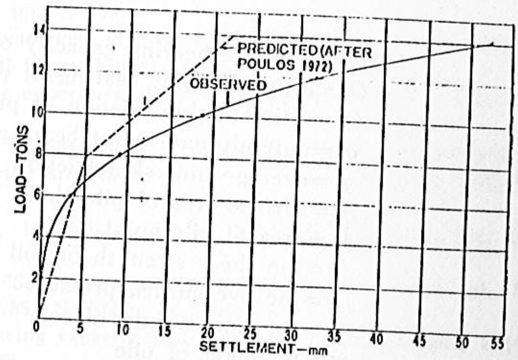


Fig. 20. Predicted and observed load-settlement curve for pile SUC 4

CONCLUSIONS

Based on the limited investigations carried out on isolated bored under-reamed piles in silty sand and the analysis thereof, the following observations are considered as significant.

1. Punching mode of failure seems to be true in case of isolated bored under-reamed piles.

2. From pile load test results, the ultimate load may be worked out using time-settlement plot. It could also be worked out from the relationship of Δ/p and Δ as proposed by Kee (1970). However, it has been reported that latter gives higher values.

3. The use of tangent methods for predicting ultimate loads may lead to erroneous predictions. Also no logic seems to apply for suggesting ultimate capacities as those corresponding to arbitrary settlement values, whether absolute or in terms of percent of pile diameter.

4. The skin friction is mobilized with a movement of 2.0 to 3.0 percent of pile diameter and the value of skin friction is about 40 percent.

5. The ultimate bearing capacity may be worked out using soil parameters considering bearing capacity factors for punching mode of failure in case of point bearing component. Estimate of skin friction may be made using coefficient of earth pressure about two.

6. The bearing capacity may also be estimated using static cone penetration resistance, q_c, after applying suitable reduction on point bearing component. In place of q_c, value of N and N_c may be used based on relationship among these values.

7. There is wide scatter between observed and predicted load-settlement curve based on elastic solutions as proposed by Poulos (1972). In addition to it there is a difficulty in assessing the correct value of E_s. Thus there is a need for developing a more systematic approach for predicting load-settlement curve.

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NOTATION

Q_u=ultimate bearing capacity of pile
Q_p=ultimate point bearing of pile
Q_s=ultimate skin friction of pile point
q_p=unit ultimate point bearing of pile
f_s=average unit skin friction of pile
A_s=surface area of pile shaft
A_p=area of pile point
c=the shear strength of soil
q=the overburden pressure
γ=the unit weight of soil
B=diameter of pile
N_c, N_q, and N_γ=bearing capacity factcors
ξ_c, ξ_q, ξ_γ=the shape factors
φ=angle of internal friction
C_a=adhesion
K_s=average coefficient of earth pressure on the pile shaft
p_0=average effective overburden pressure
δ=angle of shearing resistance between pile and soil
q_c=average static cone resistance around toe
$\bar{q}_c$=average static cone penetration resistance for embedded lengths of pile
N=number of blows in standard penetration test
N_c=number of blows in dynamic cone test
D_u=diameter of bulb
D_1=depth of centre of bulb
D_f=depth of embedment
tsf=tons per square foot
Δ=settlement of pile
D_c=critical depth of penetration of pile
D_b=depth in bearing stratum
E_s=modulus of elasticity of soil

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