Uplift Resistance of Underreamed Piles in Silty Sand

by

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Introduction

THE bored piles with enlarged base (single underreamed piles) or having two and more bulbs (multi-underreamed piles) are being effectively used to resist uplift loads for the foundation of transmission line, antenna and other elevated towers: dry docks and underground tanks etc. Very often the piles are used in groups to resist high uplift loads. For designing these pile foundations certain guidelines are available (IS : 2911 Part-III-1973, Sharma et. al, 1978). Empirical methods for determining the uplift capacity of uniform diameter and enlarged base bored piles have been proposed on the basis of model and the limited number of field tests (Meyerhof and Adamas (1968), Sowa (1970), Khadilkar et. al (1971), Meyerhof (1973), Das et. al (1975) and (1976), Tomlinson (1977) and Sharma et. al (1978)). However, there appear to be little published information about the behaviour of isolated and group of bored underreamed piles particularly based on field tests in sandy stratum.

Uplift Capacity of Straight Bored Piles

The uplift capacity of straight bored piles is calculated in exactly the same way as the skin friction in compression. But the capacity computed thus is generally higher as the skin friction in compression is more than the skin friction in uplift (Dinesh Mohan et.al. 1964, Chandrasekarn et.al, 1978). In view of this and bearing in mind that a small upward movement of pile is required to mobilize the peak resistance, a factor of safety three on the ultimate resistance is usually adequate (Tomlinson, 1977).

In computation of skin friction main debating parameter is the coefficient of earth pressure, K. Sowa (1970) from actual pulling tests on bored piles in the field has found large variation in K indicating more value for shorter piles than longer piles. The value of K has been found definitely more than one for shorter piles and it is about 1.5. Meyerhof (1973) has suggested the following values of K in terms of limiting uplift coefficient, K_u for bored piles

φ	20°	30°	40°
K_u	0.90	1.40	2.67

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This paper was received in October, 1978 and is open for discussion till the end of June, 1980.

Using these values of K_u Das et.nl (1975) has found a close agreement between the estimated and observed values of uplift capacity of uniform diameter piles of length equal to 10-12 pile diameter in model test in sand.

The published results on the group of bored piles are very scarce. Model tests on group of 30 cm long wooden piles with length to diameter ratio 24, carried out in sand, indicated that the maximum group efficiency could be achieved at a pile spacing of 4 to 6 pile diameters (Das et.al, 1976). Also for a given pile spacing of 4 to 6 pile diameter the efficiency decreased with the increased number of piles in the group.

Uplift Capacity of Enlarged Base And Underreamed Piles

The ultimate uplift capacity of footings with enlarged bases could be worked out either by empirical methods or by considering mobilization of shear resistance along slip surfaces in addition to the weight of foundation and soil block within the zone bounded by the rupture surfaces. In the most commonly used emperical method known as earth frustom method, the resistance is provided by the weight of foundation and soil within the frustum of a pyramid having a certain angle with the vertical which depends upon the type of soil. The field tests carried out in a sandy stratum on open and undercut footings alongwith underreamed pile, revealed that the pyramid angles were different for different foundations (Sharma et.al, 1968). Also the pattern of cracks, indicative of failure mechanism, observed at the surface was different in different cases. In case of underreamed piles, the cracks occurred were radical only which progressed gradually with increasing pull out. Thus the earth frustum method could not be used for estimating the capacity of underreamed piles. Under the uplift loads the actual slip surfaces may be rather complex as they depend on soil type, foundation geometry and the magnitude of uplift. For simplifying the computations they were idealised and, obviously not true representation of the actual mechanism of failure. In model tests on underreamed piles in granular soil, the actual rupture surfaces were found close to log spiral (Khadilkar et.at. 1971). To consider the mobilization of shear strength along this surface, sophisticated analysis is called for.

Based on the studies (Meyerhof and Adams, 1968) regarding the uplift resistance of a circular plate embeded in a partly cohesive ($c-\varphi$) soil, Tomlinson (1977) has suggested the use of following equation to estimate the uplift capacity of piles with base enlargements.

$$Q_u = \pi c\, D_u\, H + s \times \tfrac{1}{2}\pi \times \gamma \times D_u\,(2d_f - H)\, HK \tan\varphi + W \qquad \ldots(1)$$

where,

Q_u = ultimate uplift resistance of pile

C = cohesive strength of soil

D_u = Diameter of the enlarged base

H = Height of block of soil uplifted by the pile

s = shape factor

γ = unit weight of soil

d_f = Depth of pile below ground level

K = Coefficient, dependent on angle of internal friction, φ

and

W = weight of soil and pile within a cylinder of diameter, D_u

The value of H is dependent on enlarged base diameter (D_u) and φ. For φ equal to 30 degrees, H is equal to 4 D_u. If d_f is equal to H, equation 1 for non-cohesive soils ($c = 0$), neglecting the weight, W, reduces to equation 2 as follows:

$$Q_u = \tfrac{1}{2}\pi\gamma\ D_u\ d_f^2\ (s \times K)\ tan\ \varphi \qquad \ldots(2)$$

The ultimate uplift capacity given by equation 2 is thus the shear resistance mobilized along idealized cylinderical (diameter, D_u and height, d_f) failure surface. The shape factor s is also dependent on the ratio of H/D_u and φ. The combined values of $s \times K$ obtained from the values of of s and K (Tomilnson, 1977) for H/D_u equal to 4 are 0.92, 1.44 and 2.5 corresponding to φ equai to 20°, 30° and 40° respectively. These values are in close agreemens with the values of K_u reported by Meyerhof (1973) for bored piles. Thus the values of K_u can be used in place of $s \times K$ and can be taken after Meyerhof (1973).

It has also been pointed out by Tomlinson (1977) that the ultimate uplift capacity as obtained from the above approach (equations 1, 2) must not exceed the combined resistance of the enlarged base (considered as a buried deep foundation) and the skin friction. For working out the enlarged base, resistance, annular area *i.e.* $\pi/4\ (D_u^2 - D^2)$ where D is diameter of pile stem, is taken.

Sharma et. al (1978) have also suggested to work out the ultimate uplift capacity of underreamed pile by computing skin friction along the pile stem and bearing on annular area of underreamed bulb using the following expression :

$$Q_u = \tfrac{1}{2}\,\pi\ DK\,\gamma\ \mathrm{ran}\ \delta \left(d_1^2 + d_f^2 - d_n^2 \right) + \pi/4 \left(D_u^2 - D^2 \right)$$

$$[\tfrac{1}{2}\ n.\gamma.\ D_u\ N_\gamma + \gamma\ N_q \sum_{r=1}^{r=n} d_r] \qquad \ldots(3)$$

Where D, is the pile diameter, K, the coefficient of earth pressure, 1.75 ; δ, the angle of wall friction between pile and soil (may be taken equal to φ) ; d_1 and d_n, the depth of the centre of first and the last underreamed bulb respectively ; n, the number of underreamed bulbs ; dr, the depth of the centre of different underreamed bulbs in cm ; N_r and N_q, the bearing capacity factors depending upon the angle of internal friction and the values of these are tabulated in the said reference. All other terms are same as explained above. The factor N_q, which is after Vesic (1963) should be reduced by 50 per cent. It is based on the fact that in case of bored piles, the point bearing has been found half to one-third than in case of driven piles (Meyerhof, 1976). The above expression is said to estimate the uplift capacity on higher side particularly the bearing component and as such the factor of safety 3 in place of 2.5 (for compression capacity) has been suggested for working out safe load. For single underreamed pile, the expression 3 can be written as—

$$Q_u = \tfrac{1}{2}\pi DK\gamma \tan\delta\, d_f^2 + \pi/4\left(D_u^2 - D^2\right)\left(\tfrac{1}{2}\gamma D_u N_r + \gamma N_q d_1\right) \quad ...(4)$$

In the present study, the field tests were carried out on full scale isolated and group of single undesreamed piles in a silty sand deposit in order to investigate the behaviour of these piles in group and also to suggest the procedure for estimating uplift capacity of isolated pile from soil properties based on available information in the literature.

Sub Soil and Field Tests

The test site forms a part of the cantonment area, near Dhandhera, situated in the South-East direction to Roorkee at about 5 km distance. Geologically, the soil deposit encountered could be considered of the Gangetic alluvium deposit which is made of sand and silts at the top, underlain by clayey soils to greater depths.

The sub soil consisted of uniform silty sand deposit and has been classified SP-SM as per Indian standard. The water table was not encountered upto the depth explored. The sub-soil investigations at the site included exploratory borings with standard, dynamic and static cone penetration tests. In dynamic cone test, a mild steel cone of 62.5 mm diameter with apex angle of 60° was used and number of blows for a penetration of 30 cm under the impact of 65 kg weight falling freely from 75 cm height were recorded. The laboratory triaxial shear tests were run on undisturbed samples of 38 mm diameter and 75 mm height to determine the angle of internal friction, φ. The value of φ was also assessed from the penetration test records. The sub-soil profile alongwith average penetration tests values, moisture content and angle of internal friction is given in Figure 1. The ranges of particle size distribution over the entire

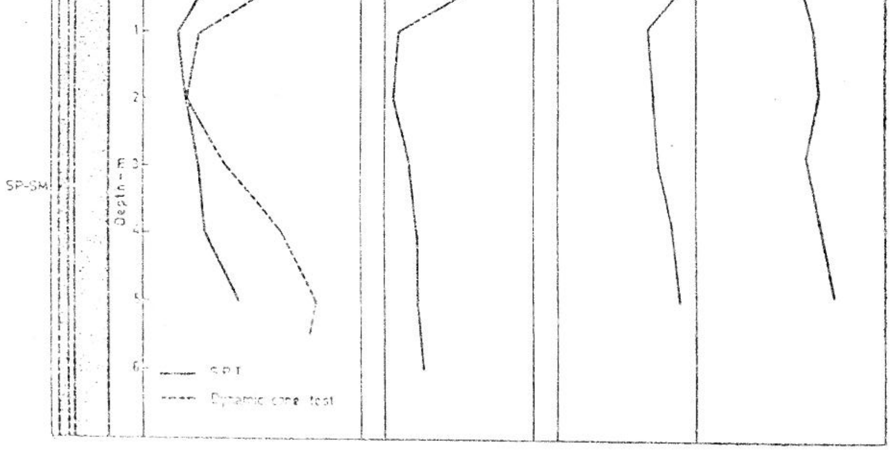

FIGURE 1 Sub-soil profile

area which reflects and gradation of soil collected from different bore-holes, is depicted in Figure 2. From the penetration resistance it could be inferred that in general the soil deposit at the test site was fairly uniform and penetration resistance increased more or less linearly with depth. Higher penetration resistance at top was due to the dessication of top 60 cm strata. Since the field tests continued for sufficient duration including rains also, its effect in taking an average value of soil parameters for computations was neglected. The average value of angle of internal friction, φ and unit weight of soil, γ could be considered 30° and 1.6 gm/cm^3 respectively over a depth of 3.5 m.

The field tests were carried out on isolated and group of 3.5 m long single underreamed piles of 30 cm diameter with underreamed diameter of 75 cm, Figure 3(*a*). Alongwith two isolated piles, the group of two and three piles at three different spacings of 3.75 D, 5.00 D and 6.25 D (D being pile diameter) were cast for the purpose of testing. Figure 3(*b*) and 3(*c*).

The piles were constructed by making a hole using spiral earth auger and then forming the bulb by an underreaming tool (Sharma et al., 1978). Finally the bores were concreted after placing the reinforcement, designed to resist uplift test loads, to full depth of the bore hole. A central bolt of 38 mm diameter (mild steel rod) was embeded in the centre of the pile. It was having threads towards the top for attaching the pull out frame, used for testing. In case of groups, the piles were connected by a rigid concrete cap which was resting on the ground. The requisite number of bolts of 32 mm diameter with the threads at the top for making the pull out set up were embeded in the caps. The bolt length embedded in concrete was adequate to provide bond strength to resist the uplift test loads.

The test set up for isolated pile was made by a hydraulic jack with a proving ring reacting against a frame attached to the pile top such that

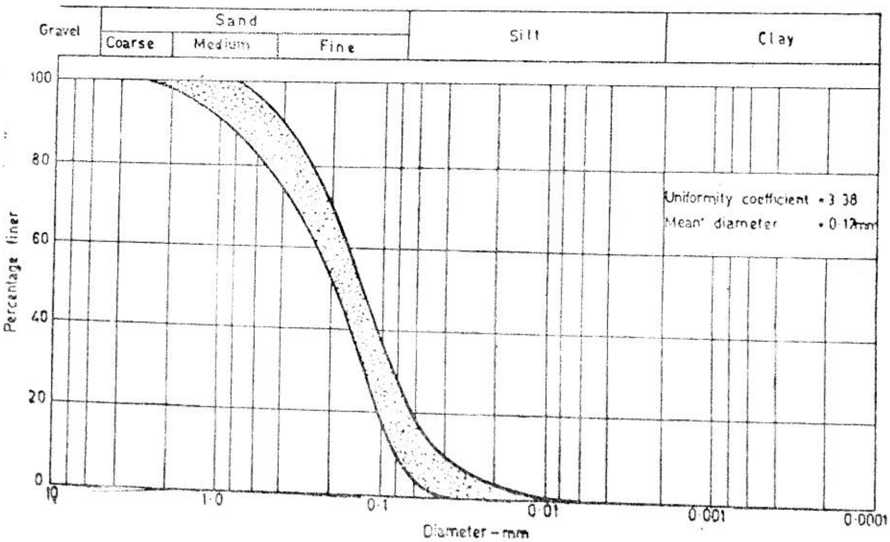

FIGURE 2 Grain size distribution at the test site

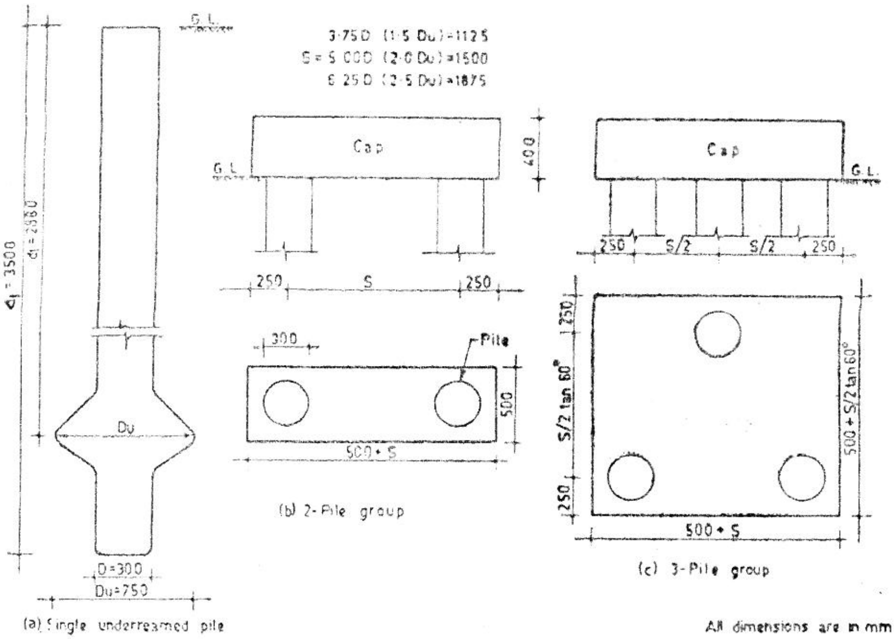

FIGURE 3 Details of pile and pile groups

when the jack was operated, the pile got pulled up. The jack was resting on rolled steel joints which in turn seated on two supports on the ground, Figure 4. There was sufficient gap between joist and bottom plate of frame for the movement of pile during the test. The displacement (pile movement) was measured with the help of dial gauges. The test set up for the pile groups was almost similar to the set up used in case of isolated piles with the difference that the reaction frame was made by clamping the rolled steel joists with the bolts embeded in the cap.

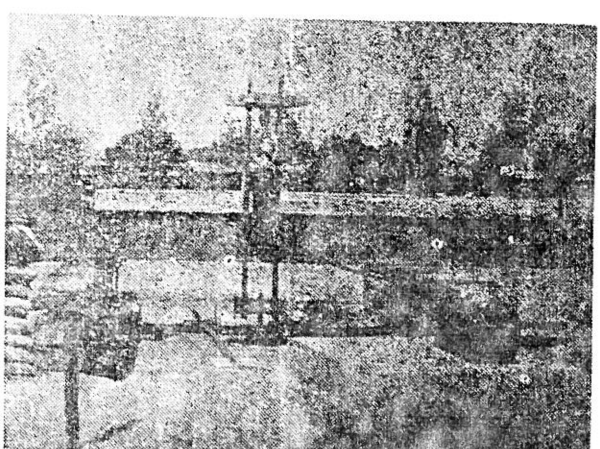

FIGURE 4 Test set-up on isolated pile

The tests were carried out by applying suitable load increments. The number of increments were decided in a way so that sufficient points could be obtained to describe the load displacement curve. Each load increment was maintained till the rate of displacement dropped to 0.02 cm per hour or for a maximum duration of two hours whichever was achieved earlier. The groups could be tested only upto limited displacements as the loading arrangement did not permit further application of uplift loads.

During the testing of isolated pile, radial cracks were observed at about 8 mm displacement. Subsequently, the cracks widened and finally extended to the maximum distance of 1 to 1.25 times the pile diameter from the periphery of pile, Figure 5. No upheavel of soil could be observed.

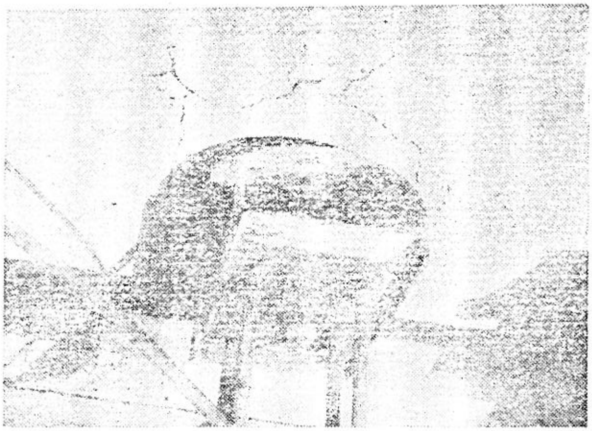

FIGURE 5 Cracks at the surface appeared in isolated pile test

Test Results and Discussion

Isolated Pile

The load-displacement curves for isolated piles are given in Figure 6. There is no appreciable scatter between the two results which ensured that the different tests could be compared. In both the cases no definite failure load, was indicated. From the trend of curves at higher loads, the ultimate uplift load for a single pile can be taken about 19 tons which also corresponded to 25 mm displacement. The ultimate uplift load, determined as the reciprocal of the slope of the line plotted between displacement/load and pile displacement (Chin, 1970), is about 19.9 t. Thus it will be appropriate to take ultimate uplift load corresponding to 25 mm displacement if no clear break is obtained in the load-displacement curve. The safe uplift load as worked out from the criteria suggested using uplift load test data (IS : 2911, Pt.-III, 1973 ; IS : 2911, Pt.-IV, 1979 and Sharma et. al, 1978) is 9.5 t. This corresponds to 1.6 mm displacement only and provides a factor of safety two on ultimate load of 19 t. The pattern of cracks observed during the load test (Figure 5) is similar to that reported in model studies by Khadilkar et. al.

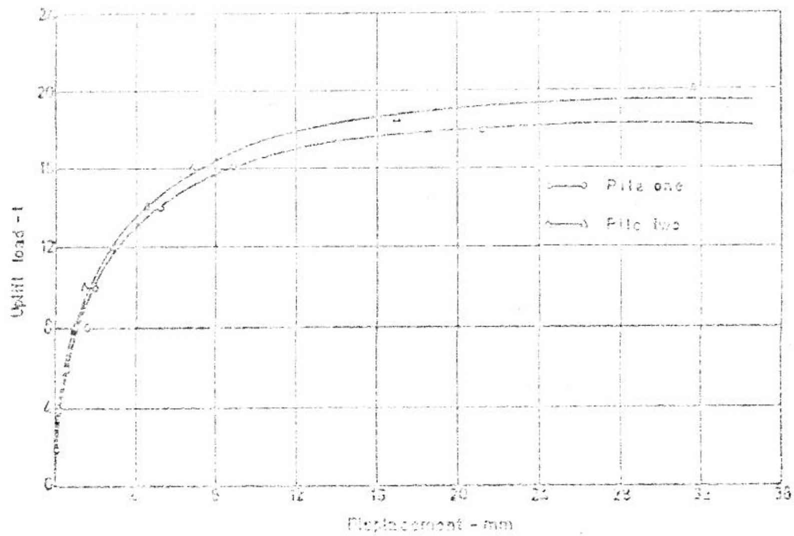

FIGURE 6 Uplift load-displacement curves for isolated piles

The uplift capacity from soil parameters has been worked out by the following methods and the calculated values alongwith those obtained from load tests are given in Table 1.

TABLE 1

Uplift Capacities of Isolated single underreamed pile

Method	Ultimate capacity			Safe capacity (t)	Factor of safety
	Friction (t)	Point bearing (t)	Total (t)		
Load tests	—	—	19	9.5	2
Soil parameters					
Method 1	—	—	18.7	9.3	2
Method 2	9.3	28.8	38.1	12.7	3
Method 3	7.5	16.5	24.0	9.6	2.5

Method 1, (Meyerhof and Adamas-1968 ; Tomlinson-1977)

In present case the centre of underreamed bulb is at 288 cm depth below ground level which is approximately equal to $4D_u$ (300 cm) for φ equal to 30 degree and thus H can be taken equal to d_f. Accordingly the ultimate uplift capacity has been computed considering shear resistance mobilized along cylinderical surface using equation 2. The value of d_f

has been taken equal to total depth of pile i.e. 350 cm and in place of $s \times K$, the limiting uplift coefficient, K_u as 1.4 corresponding to φ equal to 30 degrees (Meyerhof-1973) has been used. The computed values of ultimate and safe uplift capacities (factor of safety 2 on ultimate) are in good agreement with those obtained from the load tests (Table 1).

Method 2, (Sharma et. al 1978)

The ultimate uplift capacity has been computed using expression 4 taking D as 30 cm, d_1-288 cm ; N_r and N_q 18 and 14.5 respectively and all other terms as given above. The worked out ultimate and safe uplift capacities are found on higher side than those observed from load tests (Table 1). The estimate seems all the more higher in case of point bearing component which may be probably due to the incorrect assessment of the value of bearing capacity factors particularly N_q.

Method 3, (Modifying equation 4)

The equation 4 has been modified as given below to compute the ultimate uplift capacity.

$$Q_u = \tfrac{1}{2} \pi D K_u \gamma \tan \delta d_f^2 + \pi/4 (D_u^2 - D^2) \gamma d_1 N_q \qquad ...(5)$$

Here K_u is taken equal to 1.4 (limiting uplift coefficient) and the N_q is taken equal to 9.67 (reduced to one third of the values after Vesic). The estimated ultimate capacity is still higher about 1.26 times than that obtained from load tests. But the safe capacity with a factor of safely 2.5 (same as in case of compression capacity) is in close agreement with the value obtained from the load test (Table 1).

In Table 2, the observed ultimate and safe uplift loads, alongwith the corresponding displacements are compared with the values obtained from safe loads suggested by Sharma et. al in design table for use in the field. The ultimate load is twice the safe load given in above reference and corresponds to 25 mm displacement. The observed values are higher and also the observed displacements are lower. It is probably due to the fact that the values in design table are on generalised basis and on conservative side.

TABLE 2

Comparison of observed and Design Table Uplift Loads

Reference	Safe uplift load (t)	Displacement (mm)	Ultimate uplift load (corresponding to 25 mm) (t)
Sharma et. al, 1978	6	4	12
Load tests	9.5	1.6	19

Pile Groups

The load versus displacement plots for two and three pile groups are shown in Figure 7 and Figure 8 respectively. Although the tests could not be conducted upto larger displacements, the trend reflects the mobilisation of most of the uplift resistance corresponding to 25 mm displacement. The increase in the uplift capacity is found to be only marginal with the increase in pile spacing from 3.75 times to 6.25 times the pile diameter. The group efficiency, defined as the ratio of the load per pile in the group corresponding to a particular displacement to the load of isolated pile corresponding to that displacement has been plotted against spacing to pile diameter ratio (S/D) and against different displacements in Figure 9 and Figure 10 respectively. The efficiencies which are about unity at all

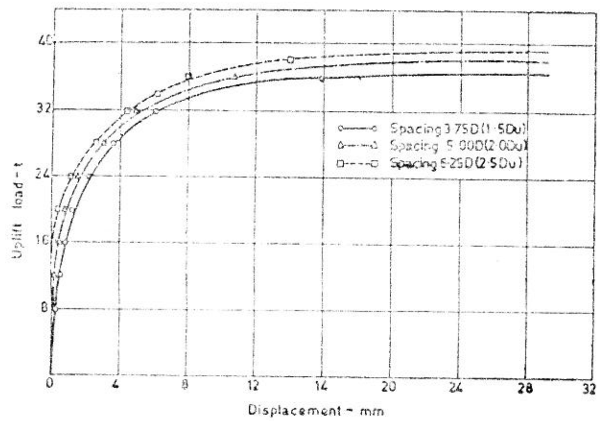

FIGURE 7 Uplift load-displacement curves for 2-pile groups

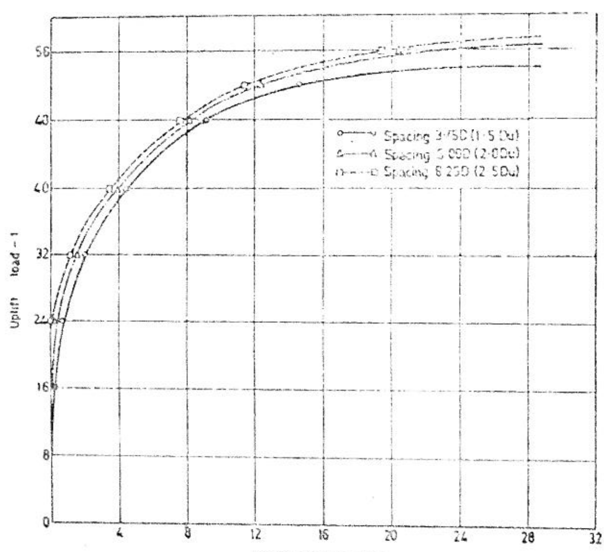

FIGURE 8 Uplift load-displacement curves for 3-pile groups

the three spacings increases linearly with the increase in spacing at a small rate, indicating the spacings studied are near optimum. It is probably because the maximum group efficiency in case of bored piles, could be achieved at a spacing of 4 and 6 pile diameter (Das et. al. 1976). Also the pattern of cracks observed at the surface in isolated pile load test suggests that in the group, the piles are likely to fail individually at a spacing of about 3.5 times the pile diameter. *Thus much difference in the capacity of groups having the piles at a spacing of about 4 to 6 times the* pile diameter could not be observed.

The efficiency can be represented by a single line beyond 8 mm displacement in case of three pile groups, indicating a constant value beyond this displacement. In case of two groups, it is not possible to represent *the same by a single line but could be taken almost constant beyond 8 mm* displacement. It is demonstrated more clearly by the efficiency versus displacement plot (Figure 10). At the small displacements, initially, the efficiency decreases with increase in displacement. It becomes constant beyond 8 mm displacement in case of three pile group and in case of two pile groups although it does not become constant, the decrease is very little beyond 12 mm. Thus one efficiency value could be used for working out group capacity from the isolated pile capacity for all values of dis-

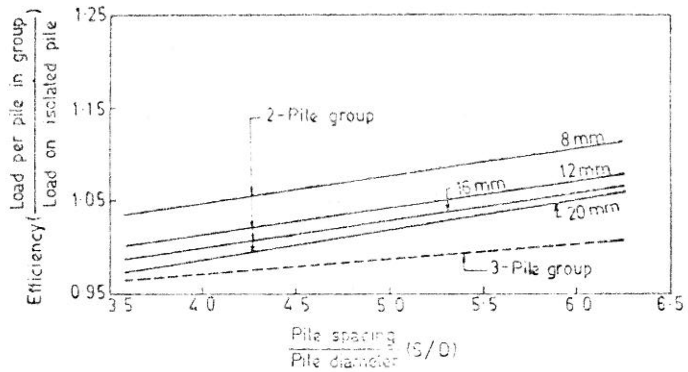

FIGURE 9 Plot of group efficiency versus spacing (in pile diameters)

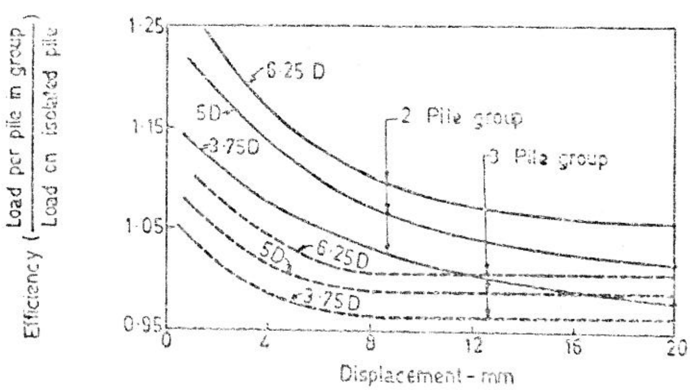

FIGURE 10 Plot of group efficiency versus of displacement

placements upto 25 mm. But this may give conservative values in the lower range of the displacements.

From the above results it is possible to construe that in case of groups with piles at the spacing of 3.75 to 6.25 times the pile diameter, the safe capacity can be taken equal to the safe capacity of isolated pile multiplied by the number of piles in the group. However, it may be mentioned here that provision of piles in a group at a spacing of 6.25 times the pile diameter will increase cap size considerable which in turn will effect the economy. Corresponding to the safe load of group, worked out as above, the displacement will also be the same to that of isolated pile. However, there can be marginal increase in displacement in case of larger pile groups since the trend of the plots (Figure 9 and Figure 10) is indicative of the decrease in efficiency with the increased number of piles in the group.

The variation in the settlement ratio, defined as the displacement of group corresponding to a specified average load per pile in the group to the displacement of isolated pile at that particular load, against the spacing to pile diamnter ratio (*S/D*) are shown in Figure 11. It reflects that precisely there can not be one value of displacement ratio and the same is different at different load levels. Also the displacement ratio decreases as settlement to diameter ratio (*S/D*) increases. It could be possible due to increase in stiffness resulted from increased dimensions of pile cap on account of larger spacing of piles. The displacement ratio also seems to be dependent on number of piles and their arrangement in the group. Thus the displacement ratio related with the group size only may be misleading.

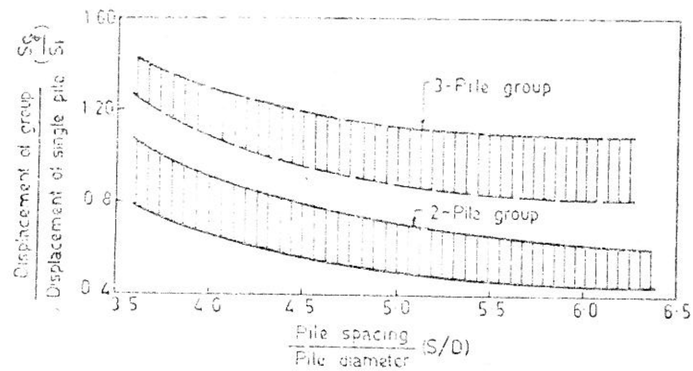

FIGURE 11 **Plot of displacement ratio versus pile spacing (in pile diameter)**

Conclusions

The following conclusions can be drawn from the present study on isolated and group of bored single underreamed piles of shallow embedments in silty sand:

1. The ultimate uplift capacity of isolated pile from a load-displacement curve can be taken corresponding to 25 mm displacement. The displacement corresponding to safe load with a factor of safety 2 on above ultimate is quite low.

2. The uplift capacity of an isolated pile estimated from soil properties by computing the shear resistance mobilised along idealised cylinderical failure surfaces using limiting uplift co-efficient (Method 1) is found in close agreement with the value obtained from load test. While the ultimate uplift capacity as a sum of skin friction using the limiting uplift co-efficient and point bearing using the N_q factor after Vesic, reduce to one third and neglecting N_r term (Method 3) is 1.26 times higher, the safe capacity with a factor of safety 2.5 on the said ultimate agrees well with that obtained from load tests.

3. The efficiency in case of groups having piles spaced at 3.75 times to 6.25 times the pile diameter, is found about one which increases marginally only with the increaae in spacing. The uplift capacity of the group from single pile test results, could be worked out using one value of efficiency i.e. unity for all values of displacements upto 25 mm. But, it will provide conservative values of group capacity in the lower range of displacements.

4. The displacement corresponding to the group capacity equal to the capacity of isolated pile multiplied by the number of piles in the group is likely to be the same to that of isolated pile. However, there could be marginal increase in displacements in case of larger pile groups.

5. The displacement ratio seems to be dependent at the load level at which it is calculated. In relating the displacement ratio with the dimensions of group, the number of piles and their arrangement in the group should also be considered.

Acknowledgement

The author would like to make special mention of Mr. K.G. Garg scientist, CBRI for his close association with the field work. The same is gratefully acknowledged. Also the work reported in this paper form a part of the regular research programme of CBRI, Roorkee and the paper is being published with the permission of the Director.

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