Behavior of Pile Groups Subjected to Vertical Loading

(A Comparative Study)

Singh P. K.
Department of Civil Engineering, National Institute of Technology, Kurukshetra, Haryana, INDIA

Arora V. K.
Professor,
Department of Civil Engineering, National Institute of Technology, Kurukshetra, Haryana, INDIA

***Abstract*--Steel pipe piles are highly durable, provide reliable foundation, have shorter construction period and can be driven into such medium where other piles can't like boulder medium. Steel pipe piles are economical for long piles into deeps loose soil. Till now most of research has been directed towards the response of individual piles to vertical loads. The present investigation was performed to study the load settlement characteristics of rectangular, square and circular pile groups under axial load conditions. The load was applied at the top of pile cap with the help of jack. The behaviors of thirteen pile groups were studied. The spacing of piles at the bottom of pile cap was kept 2.5d in each case where d is diameter of the pile for testing. The piles were arranged in planes perpendicular to the direction of load and were symmetrical. Tests were conducted in laboratory under controlled density conditions using dry, clean, uniform sand. Only deflections at ground level were measured.**

Keywords- Pile Foundation, pile cap, bearing capacity, pile group

I. INTRODUCTION

Pile foundations have been used as load carrying and load transferring systems for many years. Pile foundations are the part of a structure used to carry and transfer the load of the structure to the bearing ground located at some depth below ground surface. The main components of the foundation are the pile cap and the piles. Piles are long and slender members which transfer the load to deeper soil or rock of high bearing capacity avoiding shallow soil of low bearing capacity. The main types of materials used for piles are Wood, steel and concrete Steel pipe piles are highly durable, provide reliable foundation, have shorter construction period and can be driven into such medium where other piles can't like boulder medium. Steel pipe piles are economical for long piles into deeps loose soil. Because of the relative strength of steel, steel piles withstand driving pressure well and are usually very reliable end bearing members, although they are found in frequent use as friction piles as well. The common types of steel piles have rolled H, rectangular and circular cross-section (pipe piles).

At the earlier time, the capacity of pile groups was taken as equal to the sum of the capacities of the individual piles. However, in practice, when piles are placed close to each other, the stresses transmitted to the soil through neighboring piles will overlaps, resulting in a considerable change of the group capacity [1, 2].

A method by which the load capacity of the individual piles in a group embedded in sand could be assigned. According to this method, the capacity of a pile is reduced by 1/16 by each adjacent diagonal or row pile. Based on this method, different loads will be assigned to different piles in the group [1].

Cumaraswamy Vipuianandan, Daniel Wong, and Michael W. O'Neill, , (1990) Methods available for estimating the bearing capacity of piles installed with vibratory drivers are inadequate and do not explicitly incorporate important variables, such as soil parameters and in situ stresses. The influence of relative density (65% and 90%), particle size (0.2 mm and 1.2 mm), and in situ horizontal stress (10 psi and 20 psi) on the load-movement relationship and bearing capacity of vibro-driven displacement piles in sand is investigated using a large scale laboratory testing system. The test results indicate that, among the variables investigated, the most important parameter influencing the rate of penetration and the bearing capacity of the vibro-driven piles is the initial relative density of sand deposit. Based on pile capacity tests and analytical study, several models are proposed to predict the nonlinear unit load-transfer curves, load-movement relationships and bearing capacity of vibration driven displacement piles. The model parameters are related to the important test variables investigated in this study. The model predictions are in agreement with the experimental results. Performance of vibration driven piles is compared with that of impact-driven piles.

The bearing capacity of open-ended piles is affected by the degree of soil plugging, which is quantified by the incremental filling ratio .There is not at present a design criterion for open-ended piles that explicitly considers the effect of IFR on pile load capacity. In order to investigate this effect, model pile load tests were conducted on instrumented open-ended piles using a calibration chamber. The results of these tests show that the IFR increases with increasing relative density and increasing horizontal stress. It can also be seen that the IFR increases linearly with the plug length ratio ~PLR and can be estimated from the PLR. The unit base and shaft resistances increase with decreasing IFR. Based on the results of the model pile tests, new empirical relations for plug load

capacity, annulus load capacity, and shaft load capacity of open-ended piles are proposed. The proposed relations are applied to a full-scale pile load test performed by the authors. In this load test, the pile was fully instrumented, and the IFR was continuously measured during pile driving. A comparison between predicted and measured load capacities shows that the recommended relations produce satisfactory predictions [3]. The results from an experimental investigation were designed to examine the effect of soil-core development and cyclic loading on the shaft resistance developed by open-ended piles in sand. An instrumented open-ended model pile was installed either by driving or jacking into an artificially-created loose sand deposit in Blessington, Ireland. The tests provided continuous measurements of the soil-core development and the radial effective stresses during installation and subsequent load tests. The equalized radial effective stresses developed at the pile-soil interface were seen to be dependent on the degree of soil displacement (plugging) experienced during installation, the distance from the pile toe, and the number of load cycles experienced by a soil element adjacent to the pile shaft. A new design method for estimating the shaft capacity of piles in sand is proposed and compared with measurements made on prototype field-scale piles [4].
Although many studies have been done to investigate the axial behaviors of open-ended piles in sands, few studies have been reported for weak clayey silts. To develop reliable models for the design of open-ended steel-pipe piles driven into 29-m-thick varied clayey silt deposits, a series of full-scale field load tests including large-strain dynamic tests and static cyclic axial-compression-load tests was conducted on two groups of instrumented piles. Through analysis of the test data, soil parameters were back-calculated for estimation of pile capacities using the static-bearing-capacity formulas and cone-resistance-based methods. The comparisons between the calculated results and the field load test data demonstrated that the following considerations can be adopted in the design of static compression capacities of an open-ended pipe pile penetrating through thick varied clayey silts to end-bearing in dense cohesion less soils: (1) a fully plugged condition can be assumed, (2) cone resistance with an upper limit of 4,788 kPa (100 ksf) can be used for unit base resistance on the soil plug, and (3) exterior unit shaft resistance can be estimated using two-thirds of the total unit shaft resistance [5] .

II. MATERIAL AND METHODOLOGY

The research work was divided into different headings as determination of index properties of loose sands, procurement of pile cap material, steel piles, sand etc, preparation of test model, testing of pile and pile groups and finally evaluation and comparison of test results.

Material

Sand: The sand used in the investigations has been collected from the banks of river Yamuna from a village near Radaur in Yamunanagar district of Haryana. The physical and engineering properties of Yamuna sand have been summarised in table below.

TABLE 1.0 PROPERTIES OF YAMUNA SAND

S. No.	Property	Value
1.	Effective size (D_{10})	0.139mm
2.	Uniformity coefficient (C_u)	2.06
3.	Coefficient of curvature (C_c)	0.982
4.	IS Classification	SP
5.	Passing 1.18 mm IS Sieve	100%
6.	Mean specific gravity (G)	2.64
7.	Minimum void ratio e_{min} /max. dry density	0.54/1.690 gm/cc
8.	Maximum void ratio e_{max}/min. dry density	0.77/1.472 gm/cc

Steel pile: It is usually desirable from economical and practical considerations that the smallest model should be used. At the same time the model pile should be slender and also wide enough so that the effect of individual soil grains is negligible. The piles were used having 2 cm diameter and the total length of pile was 50 cm with an embedded length of 40 cm.

Mild steel plates: Mild Steel Plates can come in various sizes and grades. Thicknesses available range from 3mm up to as thick as 150mm. The plate used in making of pile caps is of 12 mm thickness which is cut according to the required dimension of pile caps.

The spacing between the piles in each group was 2.5 times of pile diameter. The size of pile cap varied according to the number of piles in a group. A minimum cover of approximately half the pile spacing was provided around the outer piles. The length of embedment of piles was kept as 40 cm in all the tests. Total thirteen tests were performed in which five circular groups were also tested. The details of the pile groups are shown in the Appendix A.

Test Procedure

Single Pile Testing

The single pile was driven into the sand. The proving ring was attached to the lower end of screw jack. Two dial gauges were fixed at the top of pile cap. These Baty dial gauges (least count 0.01 mm and 25 mm travel) were supported on a cross angle sections through magnetic bases. The average of dial gauge readings was taken as the average settlement under a particular load.

The load was applied in small increments. Load was maintained constant after an increment, till the settlement was constant. When there was no movement of dial gauge readings were recorded. Next increment was applied and the process was repeated till failure i.e. when the pile started setting rapidly. Graph below shows the load-settlement curve obtained from the Appendix B.

Pile Group Testing [7]

The proving ring was attached to the screw jack. Two dial gauges were suitably mounted on the pile cap at opposite corners. It was checked by tapping that the dial gauges were securely fixed. The load is than applied at the centre of pile cap by turning the screw jack. After each increment, the load was maintained constant till the settlement was constant.

After the settlement was complete, the next increment of load was applied and the process continued till the pile group started sinking. From the recorded readings of proving ring and dial gauges, the values of load settlement were computed. The load settlement curves obtained for various pile groups are shown in Appendix B.

Theoretical Pile Group Efficiency

The efficiency of pile groups were calculated by using the pile group efficiency equation. There are many pile group equations. These equations are to be used very cautiously, and may in many cases be no better than a good guess. The Converse-Labarre Formula is one of the most widely used group-efficiency equations which is expressed as

$$\eta = 1 - \frac{\alpha(n-1)m+(m-1)n}{90mn}$$

Where:

η= Group efficiency
m= no. of rows.
n= no. of columns.
$\alpha = \tan^{-1}(d/s)$
s= centre-centre spacing of piles.
d=Pile diameter.

Experimental Pile Group Efficiency

The spacing of piles is usually predetermined by practical and economical considerations. The design of a pile foundation subjected to vertical loads consists of

1. The determination of the ultimate load bearing capacity of the group Q_{gu}.

2, Determination of the settlement of the group, S_g , under an allowable load Q_{ga}.

The ultimate load of the group is generally different from the sum of the ultimate loads of
individual piles *Qu.*

$E_g = Q_{gu} / \sum Q_u$

is called group efficiency which depends on parameters such as type of soil in which the piles are embedded, method of installation of piles. The efficiency of pile groups obtained by using this formula and experimental efficiency are shown in the table 4.3.

Test Result and Interpretation

The data obtained from tests on single pile and pile groups at various spacing's is presented and interpreted in the following sections.

Failure load

Graph.4.2 shows the load settlement plot obtained from the single pile test. A glance of this figure indicates that load settlement plot approaches almost vertical tangent when the pile start sinking rapidly. The load corresponding to rapid sinking of the pile is taken as the failure load of the pile. The failure loads for the pile groups have been obtained in the similar manner.

Comparison of Experimental Failure Load with Other Theories

The ultimate bearing capacity (Qu) of piles in granular soils is given by the following formula:

$$Q_u = A_p\left(\tfrac{1}{2} D_\gamma N_r + P_D N_q\right) + \sum_{i}^{n} K P_{Di} \tan\delta A_{si}$$

where

Ap = cross-sectional area of pile toe in cm^2;
D = stem diameter in cm;
γ = effective unit weight of soil at pile toe in kgf/cm3;
PD = effective overburden pressure at pile toe in kgf/cm2;
N_r and N_q = bearing capacity factors depending upon the angle of
internal friction ϕ at toe.

$\sum_{i=1}^{n}$ = summation for *n* layers in which pile is installed;
K = coefficient of earth pressure;
*P*Di = effective overburden pressure in kg/cm2 for the *ith*
layer where *i* varies from 1 to *n;*
δ = angle of wall friction between pile and soil, in degrees (may be taken equal to ϕ)
*A*si = surface area of pile stem in cm2 in the *i*th layer where *i*
varies from 1 to *n*.

NOTE 1 — N_r factor can be taken for general shear failure as per IS: 6403-1981*.

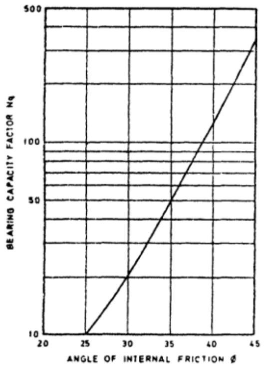
NOTE 2 — N_q factor will depend, apart from nature of soil on the type of pile and its method of construction, for bored piles, the value of N_q corresponding to angle of shearing resistance are given in Fig. 1. This is based on Berezantseu's curve for *D*/*B* of 20 up to = 35° and Vesic's curves beyond = 35°.

NOTE 3 — The earth pressure coefficient *K* depends on the nature of soil strata, type of pile and its method of construction. For bored piles in loose medium sands, *K* values between 1 and 3 should be used.

NOTE 4 — The angle of wall friction may be taken equal to angle of shear resistance of soil.

NOTE 5 — In working out pile capacities using static formula, for piles longer than 15 to 20 pile diameter, maximum effective overburden at the pile tip should correspond to pile length equal to 15 to 20 diameters.

TABLE 2.0 PLOT FOR BEARING CAPACITY FACTOR VS ANGLE OF INTERNAL FRICTION



CONCLUSION

From the model tests carried out on vertical pile groups of rectangular, square and circular in loose sand following conclusion have been drawn:-

1. The pile group load increases as number of piles in a group are increased.
2. The efficiency of driven pile groups in sand is maximum in square group and lesser in rectangular group.
3. The experimental efficiency of pile groups is more than the efficiency obtained by converse-Labarre formula.
4. The efficiency obtained by the experiment in loose sand is more than 1.
5. The ultimate bearing capacity of circular groups is more in comparison of other two groups.
6. The experimental failure loads for pile groups are higher than the failure loads obtained using I.S code method.

APPENDIX-A

S.No	Pile Group	Types	No. of Piles	Weight (kg)
1.	------	Square	1	0.331
2.	1X1	Rectangular	2	0.735
3.	------	Circular	3	1.08
4.	2X2	Square	4	1.07
5.	------	Circular	5	1.70
6.	2X3	Rectangular	6	1.57
7.	------	Circular	7	2.31
8.	3X3	Square	9	2.23
9.	------	Circular	11	2.71
10.	3X4	Rectangular	12	2.99
11.	3X5	Rectangular	15	3.77
12.	4X4	Square	16	3.92
13	4X5	Rectangular	20	4.90

The details of pile groups are shown below.

WEIGHT OF PILE CAPS

THEORETICAL AND EXPERIMENTAL EFFICIENCY OF VARIOUS PILE GROUPS

S.NO	Pile Group	No. of Piles	Theoretical Efficiency	Experimental Efficiency
1.	1X1	2	1	1.12
2.	2X2	4	0.87	1.09
3.	2X3	6	0.871	1.10
4.	3X3	9	0.83	1.11
5.	3X4	12	0.83	1.02
6.	3X5	15	0.829	1.08
7.	4X4	16	0.81	1.125
8.	4X5	20	0.80	1.05

ULTIMATE BEARING CAPACITY OF VARIOUS PILE GROUPS

S.NO	Pile Group	No. of Piles	Theoretical Ultimate Bearing Capacity (kgf)	Experimental Ultimate Bearing Capacity (kgf)
1.	---	1	34	40.33
2.	1X1	2	68	90.735
3.	2X2	4	118.32	176.088
4.	2X3	6	178	266.57
5.	3X3	9	254	402.23
6.	3X4	12	339	492.99
7.	3X5	15	423	653.77
8.	4X4	16	441	718.92
9.	4X5	20	544	844.90

APPENDIX –B

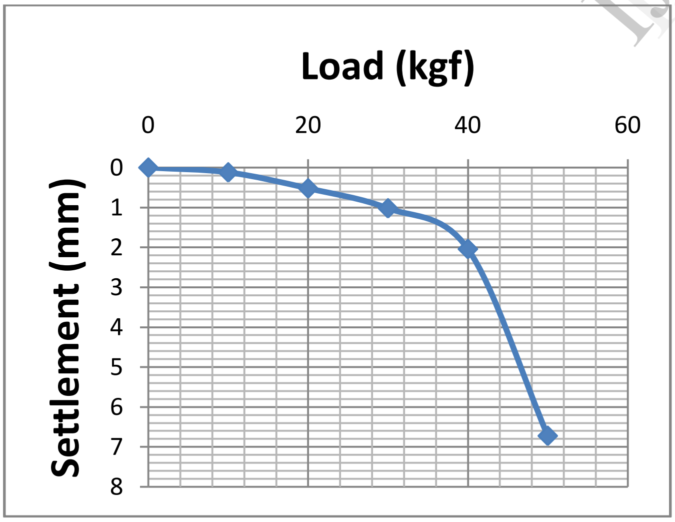


FIG 1.0 LOAD SETTLEMENT CURVE OF SINGLE PILE

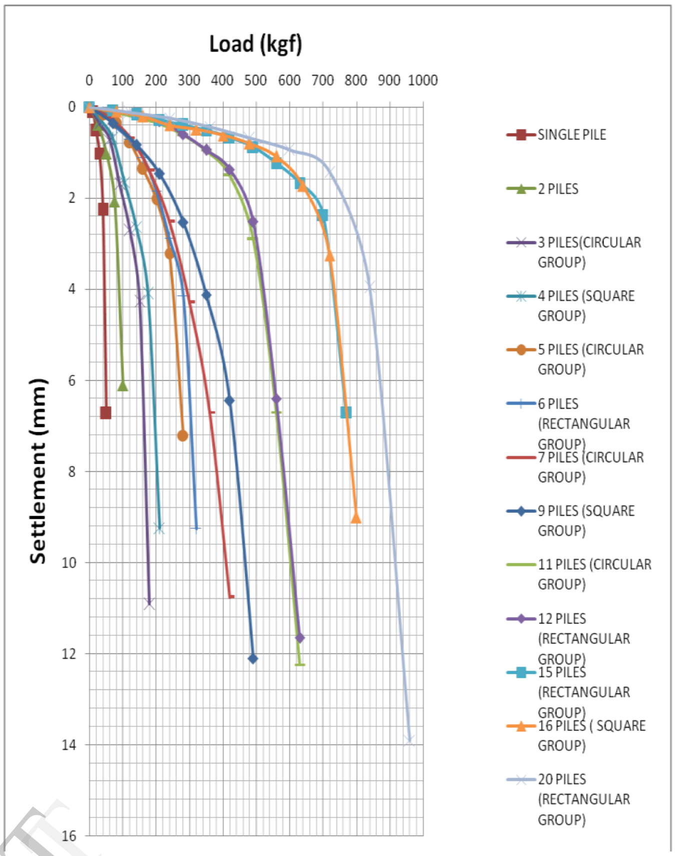


FIG 2.0 LOAD SETTLEMENT CURVES OF TOTAL 13 GROUPS

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