Shaft Friction of Bored Piles in Sand

La Friction de Puits de Tas Ennuyés dans le Sable

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ABSTRACT

The precise prediction of maximum load carrying capacity of bored piles is a complex problem because it is a function of a number of factors. These factors include method of boring, method of concreting, quality of concrete, expertise of the construction staff, the ground conditions etc. besides the pile geometry. The performance of pile load tests is, therefore, of paramount importance to establish the most economical design of piles especially where bored cast-in-situ piles are to be provided to support a structure.

This paper describes the experience gained from five pile load tests at Greater Thal Canal Project in the Punjab Province of Pakistan. Geotechnical investigations at the site were carried out through a number of boreholes drilled to depths varying between 60 ft and 70 ft below the existing ground level. These investigations revealed that the subsoils consist of silty sand in very loose to medium dense state up to depth of 10 ft followed by medium dense to very dense fine sand up to the maximum explored depth of 70 ft. The historic data on depth to water table expose that the ground water levels are not evenly distributed over the whole project area. The ground water table was encountered at a depth varying from 9 ft to 30 ft. Topography is also characterized by elevated sand dunes and interdunal depressional areas. Five test piles of diameter 2.5 ft and length 55 ft, but in different GWT levels were casted in-situ by reverse rotary method and were then loaded by axial compression load to failure.

The load test data were analyzed using various state of the art techniques (intercept of two tangents, point of change of slope, 6 mm net settlement (AASHTO), 90 percent (Hansen 1963), 80 percent (Hansen 1963), limit value (Davisson 1972), Chin (1970). Based on a comparison of pile capacities from these methods with the theoretical values, recommendations are made on the method most applicable to estimate the pile capacity in the local conditions.

RÉSUMÉ

La prédiction précise de capacité de transport de charge maximum de tas ennuyés est un problème complexe parce que c'est une fonction d'un certain nombre de facteurs. Ces facteurs incluent la méthode pour ennuyeux, la méthode pour bétonner, la qualité de béton, l'expertise du personnel de construction, les conditions de terre etc. en plus de la géométrie de tas. La performance d'épreuves de charge de tas est, donc, de l'importance suprême pour établir le design le plus économique de tas surtout où a ennuyé des tas de cast-in-situ doivent être fourni soutenir une structure.

Ce papier décrit l'expérience a tiré profit de cinq épreuves de charge de tas au Plus grand Projet de Canal Thal dans la Province Punjab du Pakistan. Les enquêtes de Geotechnical au site se sont faites par un certain nombre de trous de sonde forés aux profondeurs variant entre 60 ft et 70 ft au-dessous du niveau du sol existant. Ces enquêtes ont révélé que les sous-sols se composent du sable limoneux dans très desserré à l'état dense moyen jusqu'à la profondeur de 10 ft suivis par moyen dense au sable parfait très dense jusqu'à la profondeur explorée maximum de 70 ft. Les données historiques sur la profondeur au rejéteau l'exposent les niveaux de nappe aquifère ne sont pas uniformément distribués sur la région entière de projet. On a rencontré la table de nappe aquifère à une profondeur variant de 9 ft à 30 ft. La topographie est aussi caractérisée par les dunes de sable élevées et interdunal depressional les régions. Cinq tas d'essai de diamètre 2.5 ft et longueur 55 ft, mais dans de différents niveaux GWT étaient casted dans - situ par la méthode rotative contraire et ont été alors chargés par la charge de compression axiale à l'échec.

Les données d'épreuve de charge ont été analysées en utilisant l'état différent des techniques d'art (l'interception de deux tangentes, le point de changement de pente, le règlement net de 6 millimètres (AASHTO), 90 pour cent (Hansen 1963), 80 pour cent (Hansen 1963), la valeur de limite (Davisson 1972), le Menton (1970). Basé sur une comparaison de capacités de tas de ces méthodes avec les valeurs théoriques, les recommandations sont rendues sur la méthode le plus applicable pour estimer la capacité de tas dans les conditions locales.

Key Words : pile design, load test, skin friction, end bearing, failure load of piles

1 INTRODUCTION

Pile foundations are the part of a structure used to carry and transfer load of a structure to the bearing ground located at some depth below ground surface. Depending upon various factors like nature of substrata, depth of ground water table, depth of stronger stratum, type and quantum of load to be supported etc., piles are designed. Pile testing is a fundamental and important part of pile design. It is one of the most effective means of dealing with uncertainties that inevitably arise during the design and construction of piles.

In Pakistan improvement in foundation practice has led to an increased reliance on bored cast-in-situ RC piles for supporting tall buildings and cross drainage structures. This paper deals with the analysis of data of five pile load tests performed at Greater Thal Canal Project. Results of these pile load tests have been compared with the load carrying capacity of the piles computed by empirical relations proposed by different researchers. In addition, seven different methods to interpret ultimate load from load/settlement data have been used with the objective to establish the method most suitable for the local conditions. Similarly tip bearing and shaft

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resistance have been interpreted from load/settlement data. The percentage of load taken by piles in skin friction and tip bearing along with slip needed to develop full mobilization of shaft friction has been computed.

The findings of this research are expected to help the designers to reach the most economical design of piles in sands under the local construction practices.

2 PILE DESIGN PARAMETERS

Theoretical pile capacities have been estimated using static equation and the following pile design parameters:

K = 0.5 (Unified Facilities Criteria UFC)
D_c = 20×Pile Diameter (EM 1110-2-2906, NAVFAC DM 7.2)
$\phi = 30^o$ (from laboratory test results)
$\delta = 3/4\ \phi$ (NAVFAC DM 7.2)
N_q = 10 (NAVFAC DM 7.2)

All the five piles tested were of diameter 2.5 ft and length 55 ft, however, GWT varies at each pile location. Theoretical pile capacity along with GWT level at each test pile location is summarized in Table 1.

Table 1 Summary of theoretical pile capacities

Test No.	Location (RD)	Depth of GWT (ft)	Ultimate Load (Tons)
1	13+662	10	153
2	44+850	13.5	163
3	126+388	30	205
4	171+900	22	186
5	210+870	22	186

3 PILE LOAD TESTS

The ASTM D1143 test procedure was followed, in general, for pile load testing. All the piles were subjected to axial compression load to reach failure. Table 2 presents the results of these pile load tests. The load against settlement plot for each load test is shown in Figure 1.

Using the various methods ultimate capacity of each test pile has been estimated from the load-settlement curve as given in Table 3. The Consultants for this project decided 12 mm as the settlement for the determination of failure load. The pile capacity against 12 mm settlement for each test pile is also included in the table.

Table 2 Pile load test results

Test No.	Location	Maximum load applied	Gross settlement		Net settlement	
	RD	Tons	inch	mm	inch	mm
1	13+662	220	0.741	18.81	0.657	16.68
2	44+850	244	0.644	16.36	0.582	14.78
3	126+388	220	0.653	16.58	0.569	14.45
4	171+900	195	0.614	15.61	0.536	13.63
5	210+870	220	0.66	16.77	0.520	13.21

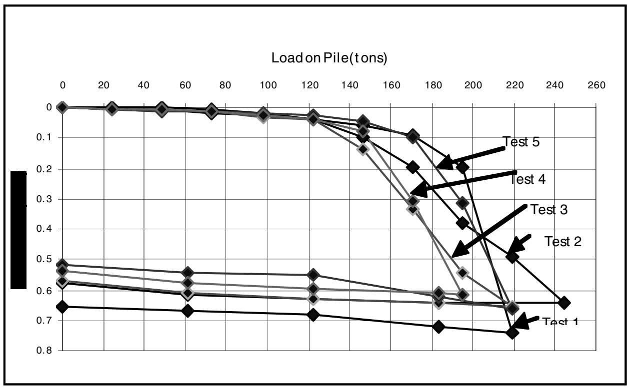

Figure 1 Load settlement plots for five pile load tests

Table 3 Ultimate loads using different methods

Method of estimating ultimate load capacity from load test		Ultimate load capacity (tons)				
		Test 1	Test 2	Test 3	Test 4	Test 5
i.	From intercept of two tangents	190	147	131	152	178
ii.	From point of change of slope	195	170	146	170	195
iii.	From 6 mm net settlement	200	184	168	170	198
iv.	90% (Hansen 1963)	170	120	120	136	158
v.	80% (Hansen 1963)	236	194	184	183	207
vi.	Limit value (Davisson 1972)	209	226	187	183	207
vii.	Chin (1970)	232	268	258	229	231
viii	12 mm settlement	210	220	190	185	210

Figure 2 shows the variation of ultimate load determined using different methods at each pile location. Table 4 gives a comparison of ultimate loads from various methods to the theoretically predicted values. An out come to this comparison is that the change of slope and 6 mm net settlement methods predict average ultimate load close to the theoretically predicted values.

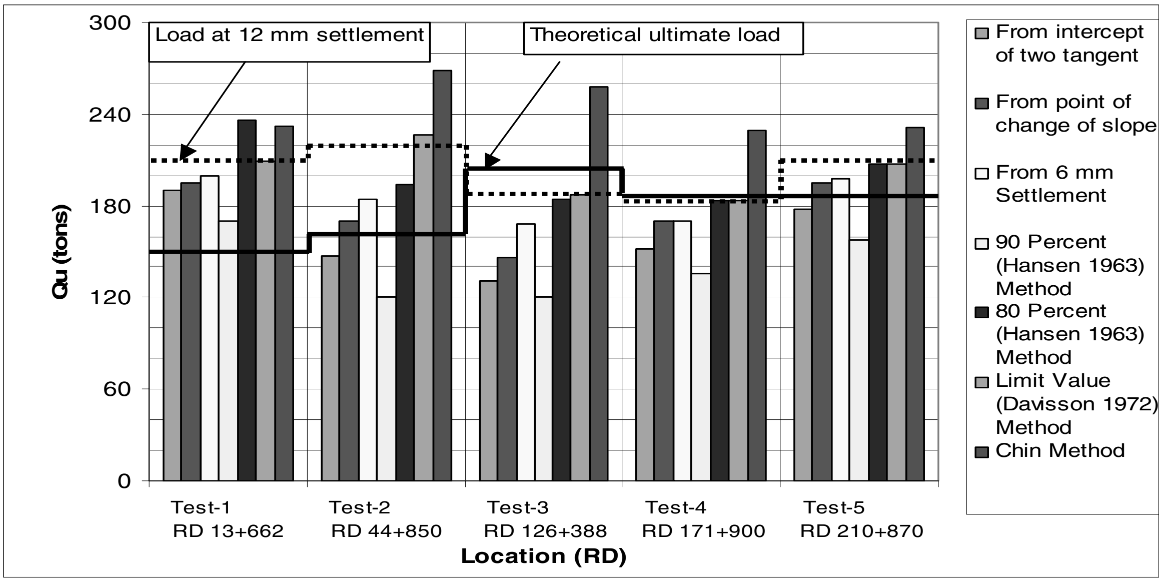

Figure 2 Variation of ultimate load for each pile using different methods

Table 4 Percentage of ultimate load from various methods to the theoretical ultimate load

Methods of estimating ultimate load capacity from load test		%age of ultimate load to theoretical ultimate load					
		Test 1	Test 2	Test 3	Test 4	Test 5	Average
i)	From intercept of two tangents	124	90	64	82	96	91
ii)	From point of change of slope	127	104	71	91	105	100
iii)	From 6 mm net settlement (AASHTO)	131	113	82	91	106	105
iv)	90 Percent (Hansen 1963)	111	74	59	73	85	80
v)	80 Percent (Hansen 1963)	154	119	90	98	111	115
vi)	Limit Value (Davisson 1972)	137	139	91	98	111	115
vii)	Chin (1970)	152	164	126	123	124	138

Table 5 Percentage of ultimate load to the failure load against 12 mm settlement

Methods of Estimating Ultimate Load Capacity from Load Test		%age of Ultimate load to maximum applied load					
		Test 1	Test 2	Test 3	Test 4	Test 5	Average
i)	From intercept of two tangents	90	67	69	82	85	79
ii)	From point of change of slope	93	77	77	92	93	86
iii)	From 6 mm net settlement (AASHTO)	95	84	88	92	94	91
iv)	90 % (Hansen 1963)	81	55	63	74	75	69
v)	80 % (Hansen 1963)	112	88	97	99	99	99
vi)	Limit Value (Davisson 1972)	100	103	98	99	99	100
vii)	Chin (1970)	110	122	136	124	110	120

Table 5 presents the percentage of ultimate load from the various methods with respect to the ultimate load against 12 mm settlement.

Table 5 shows that 80 % Hansen and Limit Value Davisson methods predict ultimate loads which agree very closely to the ultimate loads against 12 mm gross settlement

Tip bearing (Q_b) and shaft resistance (Q_s) components interpreted by Van Wheele (1957) method illustrate that the shaft friction is fully mobilized between 3 to 8 mm gross settlement and base resistance increases continuously up to failure. Table 6 shows the settlement for complete mobilization of shaft resistance and the percentage of total load carried by shaft and base. On average, at failure up to 30 % of load was taken by the piles at the base, and about 70 % of load was resisted along the shaft.

Table 6 Settlement for complete mobilization of shaft and %age of total load carried by shaft and base

Test No.	Location	Settlement at full mobilization of skin friction	Ultimate skin friction	Ultimate base resistance
	RD	mm	(%)	(%)
1	13+662	5	85	15
2	44+850	5	57	43
3	126+388	3.5	58	42
4	171+900	7.5	75	25
5	210+870	8	78	22
Average			70	30

4 CONCLUSIONS:

The following conclusions are drawn from this study:

- The change of slope and 6 mm net settlement methods provide ultimate loads on average close to the theoretically predicted values using the parameters assumed.
- For 12 mm gross settlement, the best methods of determining ultimate load from pile load test results for the local sand are the 80% Hansen and Limit value (Davisson 1972) methods.
- For bored piled in sand, at an average, about 30 % of load is taken by the piles at the base, and that up to 70 % of load is taken by the shaft friction at failure.
- The slip to develop maximum skin resistance is on the order of 3 to 8 mm

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