Axially loaded 5 pile group and single pile in sand

Un groupe de cinq pieux et un pieu isolé dans le sable avec chargement axial

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SYNOPSIS: A group of five piles was loaded to failure in a medium dense sand together with a control single pile as a reference. The piles were heavily instrumented; this allowed to obtain all usual data on the pile behavior during loading including residual stresses after driving, load versus depth, load transfer curves at various depth and so on.

THE PROGRAM
The Federal Highway Administration recently sponsored a research project on piles in Sand. Phase 1 has already been published (Briaud,Tucker, 1984). Phase 2 consisted of load testing 5 driven single piles and a group of 5 driven piles. The results of the program are documented in 4 reports: 2 on the field work (Ng 1988 (a),Ng 1988 (b)) and 2 on the results and their analysis (Tucker, Briaud,1988,Kon,Briaud, 1988). Only the group of 5 driven piles and the corresponding control single pile are dealt with in this article.

THE SOIL
The site is in San Francisco. The first 1.37 m are made of sandy gravel with particles up to 10 cm in size (Figure 1). From 1.37 m to 12.20 m is a hydraulic fill made of clean sand (SP). Below 12.20 m layers of medium stiff to stiff silty clay (CH) are interbedded with the sand down to the bedrock found around 14.33 m . The water table is 2.4 m deep. The sand of the hydraulic fill had the following average properties: 80% of the particles by weight smaller than 1mm, 2% smaller than 0.075 mm, dry unit weight 15.7kN/m^3 water content 22.6%, friction angle 35.4° from direct shear tests on Sprague-Henwood samples, SPT blow count 15 blows per 30 cm , CPT point resistance 6240 kPa, shear wave velocity shear modulus 38320 kPa. Selected profiles are shown on figures 2, 3 and 4.

THE PILES
The piles are closed end steel pipe piles: 27.3 cm outside diameter, 0.93 cm wall thickness, cross section area A including instrumentation channels 99.42 cm . If E is the steel modulus, the AE value was measured to be 1.584 x 10^6 kN during the calibration procedure. The instrumentation on the piles consisted of strain gages, top and toe load cells, and toe tell tales. The instrumentation of the soil consisted of extensometers and piezometers. Before driving each pile, a 1.37 m deep, 0.3 m diameter hole was drilled in the sandy gravel layer. Then each pile was driven down to a depth of 9.15 m below the ground surface.

THE PREDICTIONS
The pile driving predictions included the use of the TTI wave equation program, the WEAP 86 wave equation program, the CASE method (performed by Dr. Holloway of InSitu Tech) and the CAPWAP analysis (performed by Dr. Goble of the University of Colorado). The quantities predicted were the maximum dynamic force at the pile top during driving,the residual force at the pile bottom after driving and the ultimate load for the pile at the end of the load test. The single pile behavior during the load test was predicted with regard to the residual point load before the load test, the ultimate load of the pile and the load settlement curve. The pile group behavior during the load test was predicted with regard to the residual point load before the load test, the ultimate load of the group and the load settlement curve.

PILE DRIVING
The piles were driven with a Delmag D22 diesel hammer. After driving the reaction piles and the test piles, the ground surface had settled 12.8 cm. The order of driving and the configuration of the pile group is shown on Figure 1. Once in place the group of 5 piles was fitted with a pile cap weighing approximately 120kN. After the load test, the pile cap was removed and the piles were restruck including the single pile. During driving a pile driving monitoring system was used: this work was done by Dr. Holloway of In-Situ Tech. The pile driving analyzer results are presented in Table 1 and 2 together with the blow count. As can be seen on the tables, driving was easy but the blow count on the final 30 cm of penetration increased as more piles were driven in the group. This reflects the compacting of the sand. After the test and upon restrike, the blow count had increased by 17% on the average; it varied less from 1 pile to another than in the initial driving and was no longer tied to the order of driving.

LOAD TESTS
The single pile was tested 24 days after driving. The loading sequence consisted of increasing the load in 45kN increments, holding the load for 30 minutes and recording the displacement as well as all the instrumentation every 5 minutes. The time to reach the final load step was 6 hours at which time the settlement had reached 8.46 cm. The pile group was tested 38 days after driving. The loading sequence consisted of increasing the load in 267kN increments, holding each load for 30 minutes and recording the displacement as well as all the instrumentation

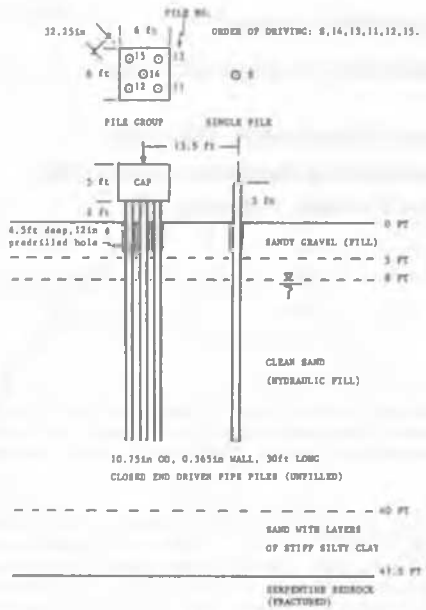

Figure 1. General test condition.(1ft=12in=0.305m)

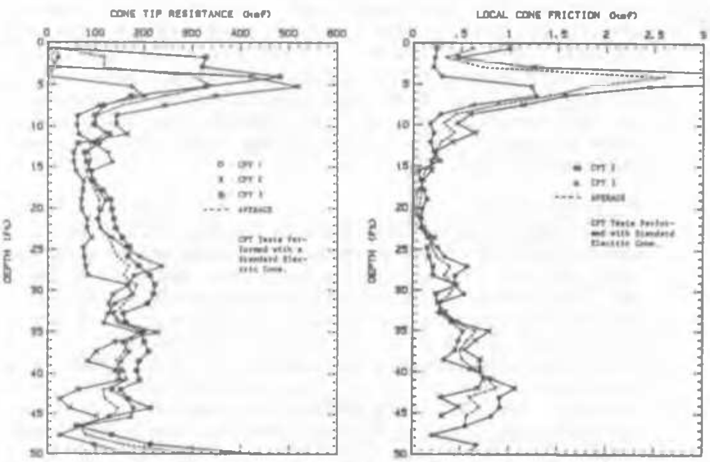

Figure 2. CPT profiles.(1ft=0.305m;1ksf=47.8kPa)

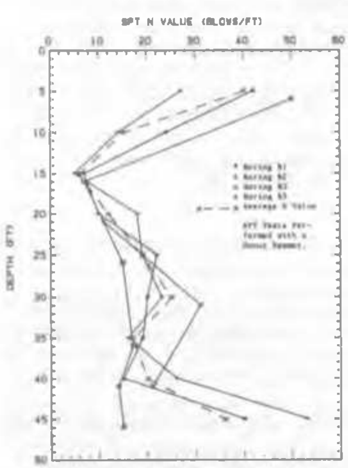

Figure 3. SPT profiles.

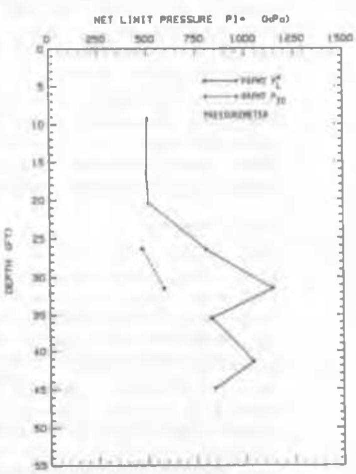

Figure 4. PMT profile.

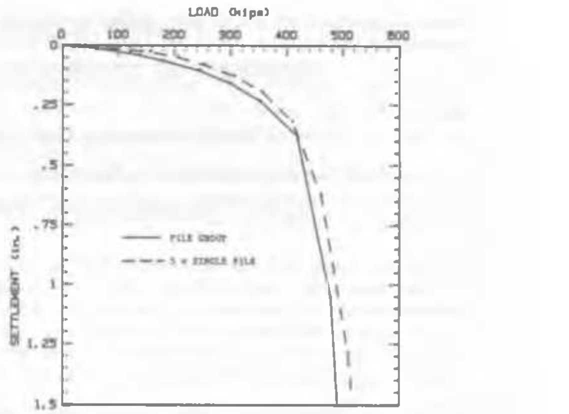

Figure 5. Load-settlement curves.(1in=2.54cm;1k=4.45kN)

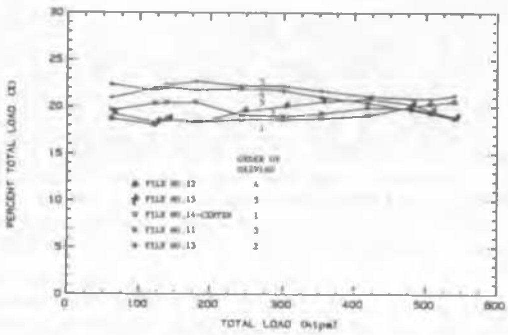

Figure 6. Load carried by each pile.(1kip=4.45kN)

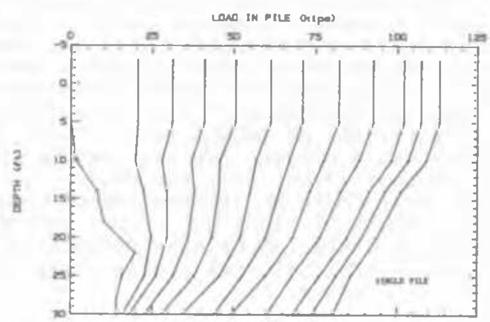

Figure 7. Load versus depth: single pile.

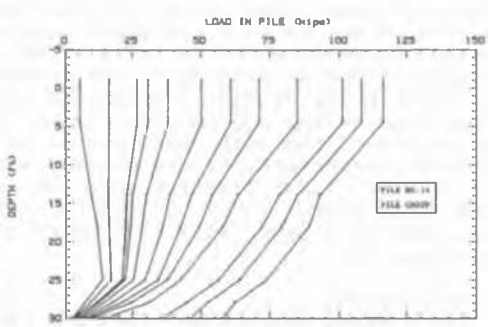

Figure 8. Load versus depth: group pile no.14.

every 5 minutes. The time to reach the end of the final load step was 9.7 hours at which time the settlement was 18 cm.

RESULTS

The results consist of the residual load distribution in the piles after driving, the load settlement curve of the single pile, of the group and of each pile in the group, the load versus depth profiles for each pile, the load transfer curves and the maximum friction versus depth profiles. The load settlement curves are shown on Figure 5. The load carried by each pile in the group is shown on Figure 6. The residual load distribution in the piles after driving was obtained experimentally by zeroing the instrumentation while the piles were laying on the ground and by reading it again after pile driving. The profiles obtained are the first profiles on the load versus depth plots of Figure 7 for the single pile and Figure 8 for pile 14 in the group. The load transfer curves are shown on Figure 9 and 10.

ANALYSIS AND CONCLUSIONS

1. Residual loads must be accounted for when analyzing instrumented load tests on single piles or pile groups. The single pile had a residual point load of 61kN or 11% of the ultimate point resistance. The piles in the group had significant residual friction stresses but had residual point loads which were much smaller than for the single pile (average 10kN). This is probably because near full penetration the driving of a pile loosens the prestressing existing under the points of neighboring piles.

2. The plunging load for the single pile was 505kN while the plunging load for the 5 pile group was 2499kN. The efficiency was 0.99. This is consistent with common practice but not consistent with previously published data. These previously published data show higher efficiency for this type of sand but were obtained on small scale models for the most part.

3. At the plunging load the best estimate of friction load carried by the single pile was 147 kN while the average friction load per pile in the group was 269kN. The efficiency on the friction load was 1.83. This is probably because driving piles in a group increases horizontal effective stresses at the soil pile interface.

4. The unit side friction at the plunging load is shown in Table 3 for all the piles, along with the ratio of the friction to the average SPT blowcount. CPT q_c, CPT f_s and PMT p_l values. These ratios can also be compared to the ratios recommended by various methods.

5. At the plunging load the best estimate of point load for the single pile was 359kN while the average point load per pile in the group was 242kN. The efficiency on the point load was 0.67. This is consistent with the observation made on the residual point loads and is again explained by the loosenning of the prestressing under driven piles due to the driving of a new pile. The efficiencies of 0.67 for the point and 1.83 for the friction lead to think that groups of end bearing piles may have efficiencies lower than 1 while groups of friction piles may have efficiencies higher than 1. In other words the efficiency depends not only on the type of sand and on the pile spacing but also on the load distribution in the piles. and on the pile length.

6. The unit point resistance of the plunging load is shown in Table 4 for all the piles along with the ratio of the point resistance to the SPT blowcount, CPT q_c and PMT p_l. These ratios can also be compared to the ratios recommended by various methods.

7. If the residual stresses are not considered the profile of maximum side friction f_{max} versus depth does indicate a clear break beyond which f_{max} does not increase versus depth; this break was at a depth of 13.4 pile diameters. This is consistent with the concepts of critical depth and of limiting friction value. If, however, the residual stresses are properly included then the break is not as clear and the value of f_{max} continues to increase with depth.

8. The coefficient of horizontal pressure, K, was calculated from f_{max} by using a soil-pile friction angle equal to 2/3 of the soil friction angle. The K values averaged 0.82 for the single pile and 1.72 for the piles in the group. The average k_o value measured by preboring and selfboring pressuremeters was 0.96.

9. The settlement of the single pile at half the plunging load (245kN) was 2.7 mm while the settlement of the group at 5 times the load on the single pile(1225kN) was 3.5 mm. The settlements at working loads are very small and the settlement ratio is 1.29. Calculating the settlement of the group by assuming that the problem is equivalent to a square spread footing having a width equal to the width of the group and located at a depth equal to 2/3 of the pile depth leads to a settlement of 10.2 mm when using the elasticity formula with the reload pressuremeter modulus.

10. The methods which predicted the single pile capacity the best as well as the point/side friction distribution were 2 methods based on cone penetrometer data(deRuiter-Beringen,1979 and Schmertmann, 1978). Among the SPT methods Coyle-Castello (1981) and Nordlund(1963)performed best. The program PILGP2(O'Neill.et al,1981)allowed to obtain the complete load settlement curve for the group. In PILGP2 the use of the reload pressuremeter modulus for the elastic interaction between piles lead to a conservative estimate of the group settlement at working loads.

11. The TTI program(Hirsch et al,1976) used with the recommended values for the various parameters predicted well plunging loadsof the single pile based on the initial blowcount but overpredicted it by 44% when using the restrike blowcount. The TTI program overpredicted significantly the static capacity of the piles in the group. The TTI program overpredicted significantly the maximum force in the piles during driving.

12. The WEAP 86 program (Goble and Rausche, 1986)used with the recommended values for the various parameters predicted well the capacity of the single pile(defined by Davisson's criterion) when using the restrike blowcount but underpredicted that load by 27% when using the initial blowcount. The WEAP 86 program predicted relatively well the capacity of the piles in the group (Davisson's criterion). The WEAP 86 program predicted a maximum force in the piles during driving which corresponded generally to the high end of the measured range.

13. The average Case method prediction of the capacity of the piles was 35% higher than the measured capacity of the pile (Davisson's criterion)but only 10% higher than the plunging load. The better agreement with the plunging load may be due to the low blowcount at full penetration in which case the pile is brought near the soil-failure under each blow. The CAPWAP analysis gave site specific values of the parameters for the

soil model and lead to a predicted pile capacity within 7% of the measured pile capacity(Davisson's criterion).

14. At low blowcounts the consideration of reresidual stresses in the pile driving analysis did not affect significantly the predicted pile capacity or the predicted driveability. The influence of residual stresses increases as the pile length increases and as the relative soil-pile stiffness increases. At high blowcounts ignoring residual stresses leads to smaller predicted capacity for a given blowcount or to a much higher blowcount for a given capacity.

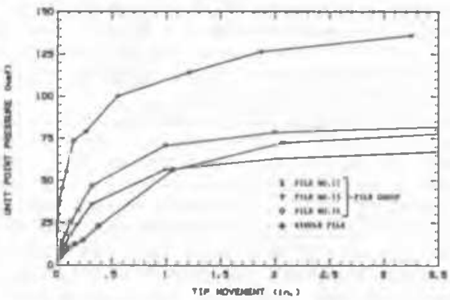

Figure 9.Point resistance curve.(1in=2.54cm;1ksf=47.8kPa)

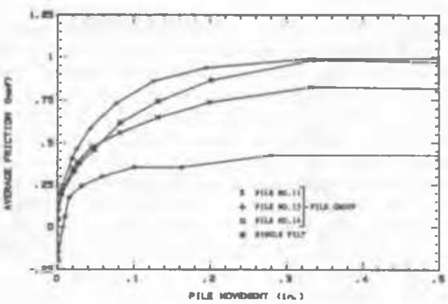

Figure 10.Friction resistance curve.(1in=2.54cm;1ksf=47.8kPa)

Table 1. Pile Driving Analyzer Results for Initial Driving*

Pile No.		Order of Driving	Total No. of blows	Blow Count Final Foot	E_{max} (kip-ft)	F_{max} (kips)	R_s (kips)
Single			62	6	9-16	215-300	80-120
Group	14	1	62	7	8-12	185-220	100-105
	13	2	62	8	10-13	245-270	140-165
	11	3	100	9	14-16	255-275	150-160
	12	4	129	9	9-11	230-250	90-100
	15	5	122	11	8-12	235-265	135-140

* furnished by Dr. D.M. Holloway of InSitu Tech

Table 2. Pile Driving Analyzer Results for Restrike*

Pile No.		Restrike Blow Count	Equi. Blow Count per ft	E_{max} (Kip)	F_{max} (kips)	R_s (kips)
Single		4/6 in	8	9-24	180-365	105-165
Group	14	8/10 in	9.6	5-27	135-340	75-120
	13	6/6 in	12	7-19	235-370	100-130
	11	8/10 in	9.6	9-18	225-310	80-135
	12	8/9 in	10.7	8-23	190-390	85-145
	15	8/10 in	9.6	10-18	245-400	90-120

* furnished by Dr. D.M. Holloway of InSitu Tech

Table 3. Comparison of friction values and soil parameters

Pile		Plunging Friction (tsf)	N/f_{max} (bpf/tsf)	q_c/f_{max}	f_s/f_{max}	p_l/f_{max}
Single		0.23	74	283	0.65	26
Group	11	0.35	48	186	0.43	17
	13	0.50	34	131	0.30	12
	14	0.42	40	157	0.36	14
Group Average		0.42	40	158	0.36	14

Table 4. Comparison of point resistance with soil parameters

Pile		Plunging resistance (tsf)	q_{max}/N (tsf/bpf)	q_{max}/q_c	q_{max}/p_l
Single		60.1	2.4	0.75	5.7
Group	11	42.2	1.7	0.53	4.0
	13	36.6	1.45	0.46	3.5
	14	42.9	1.7	0.54	4.1
Group Average		40.7	1.6	0.51	3.9

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